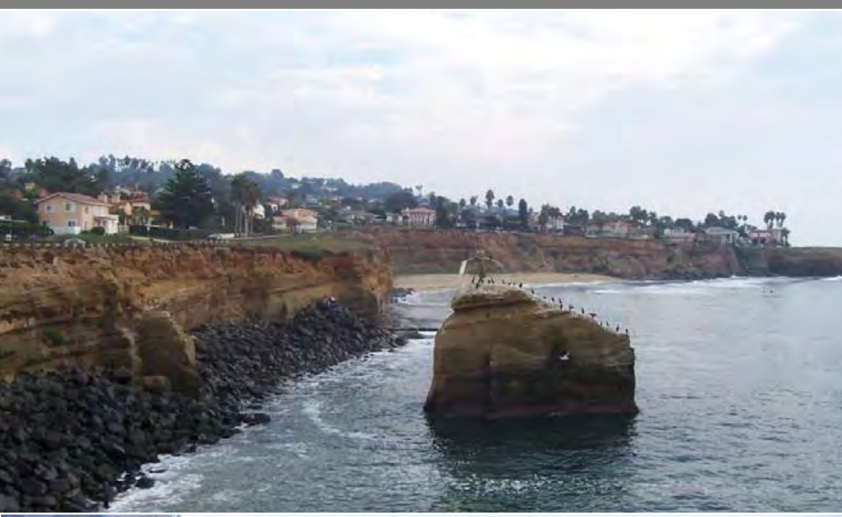
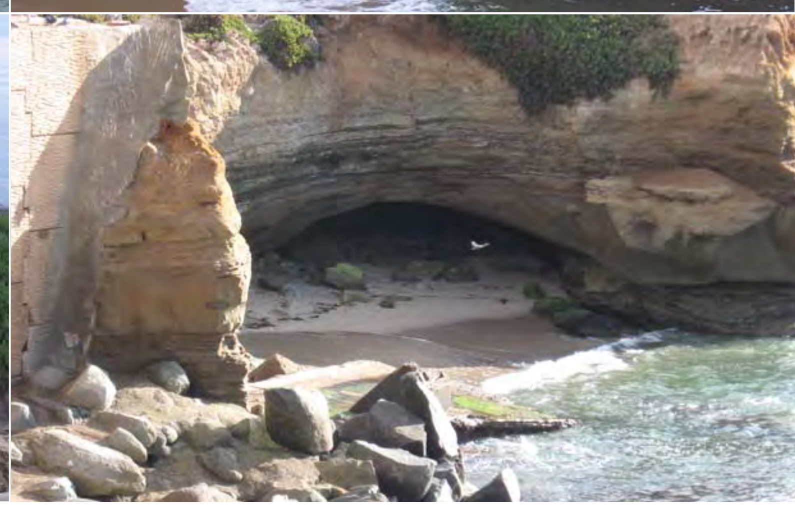
DUDEK

SUNSET CLIFFS NATURAL PARK

Final Drainage Study



April 2012

PREPARED FOR
Park Planning and Development Division
Park and Recreation Department
City of San Diego
202 'C' Street
San Diego, CA 92101

PREPARED BY
Dudek
605 Third Street
Encinitas, CA 92024
800.450.1818
www.dudek.com

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FINAL

SUNSET CLIFFS NATURAL PARK DRAINAGE STUDY

Prepared for:

City of San Diego
Park and Recreation Department
202 'C' Street, MS 35
San Diego, CA 92101
Contact: Paul Jacob, RCE, Associate Civil Engineer

Prepared by:

DUDEK
605 Third Street
Encinitas, California 92024
Contact: Steve Jepsen, Senior Project Manager

MAY 2012

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EXECUTIVE SUMMARY

Project Description

This Drainage Study for Sunset Cliffs Natural Park was completed to provide a drainage improvement plan and pipeline alignments suitable for scoping the detailed construction design activities. The challenge was to conceive a system that will convey large storm water runoff flows from up slope hardscaped developed areas across the natural parks while eliminating the severe erosion problems. The Drainage Study began with a blank canvas, enthusiastic volunteer drainage committee, engineering team and a genuine need for permanent drainage improvements to preserve the parks natural coastal resources. The final edition of the Drainage Study provides a layout, drainage inlet locations, pipeline route, pipeline sizes, and outlet locations for a complete drainage system designed to prevent damaging erosion from storm water flows traveling from upland improved areas across the park to the Pacific Ocean.

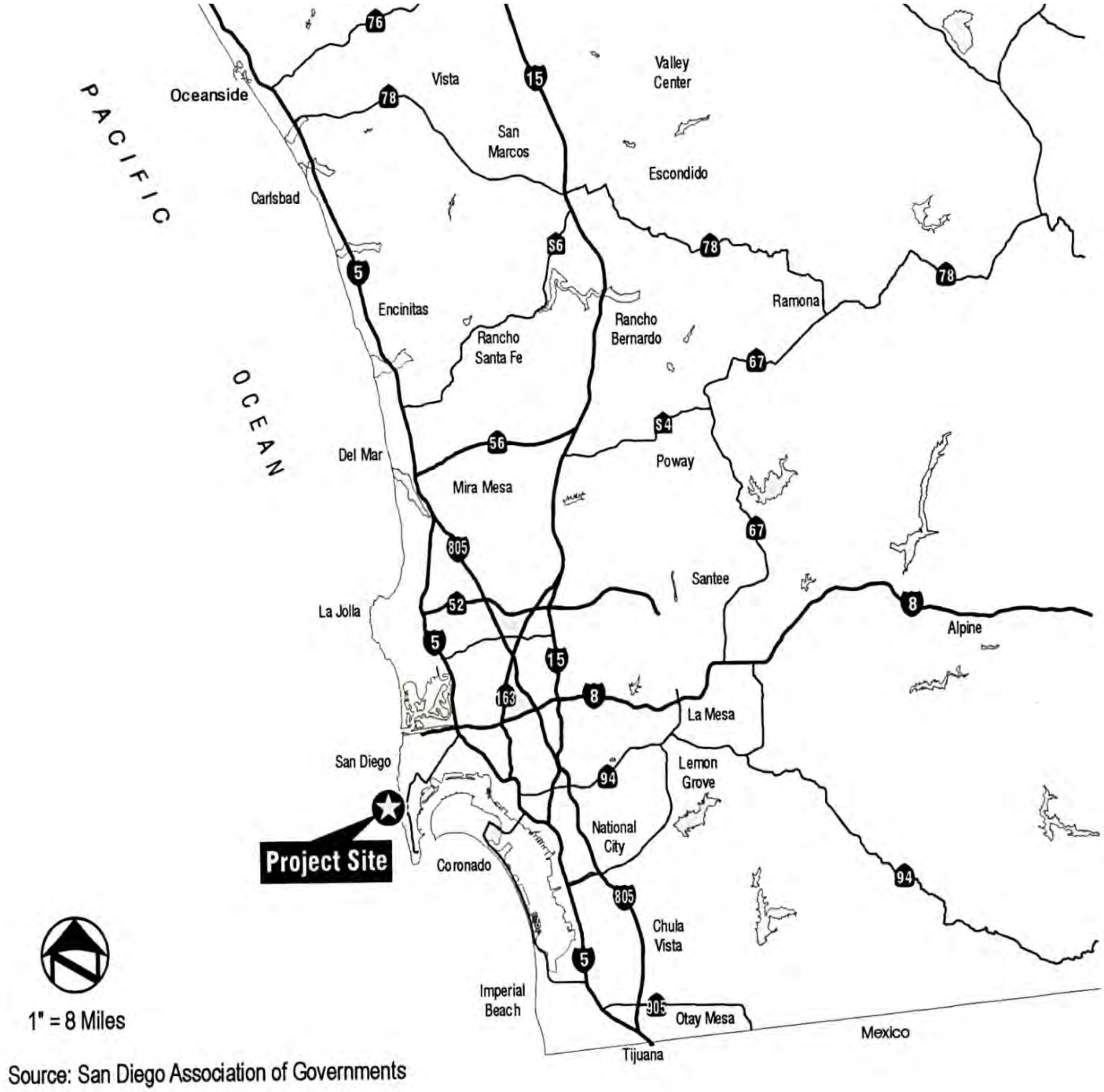
Site Description

Sunset Cliffs Natural Park (SCNP) is located approximately five miles west of downtown San Diego along the western shoreline of the Point Loma Peninsula. The Park is bordered to the north by the Adair Street/Sunset Cliffs Boulevard intersection. The site is bordered to the west by the Pacific Ocean and to the east by Sunset Cliffs Boulevard, single-family residential uses, and the Point Loma Nazarene University (PLNU). The site is bordered to the south by the Fort Rosecrans Military Reservation.

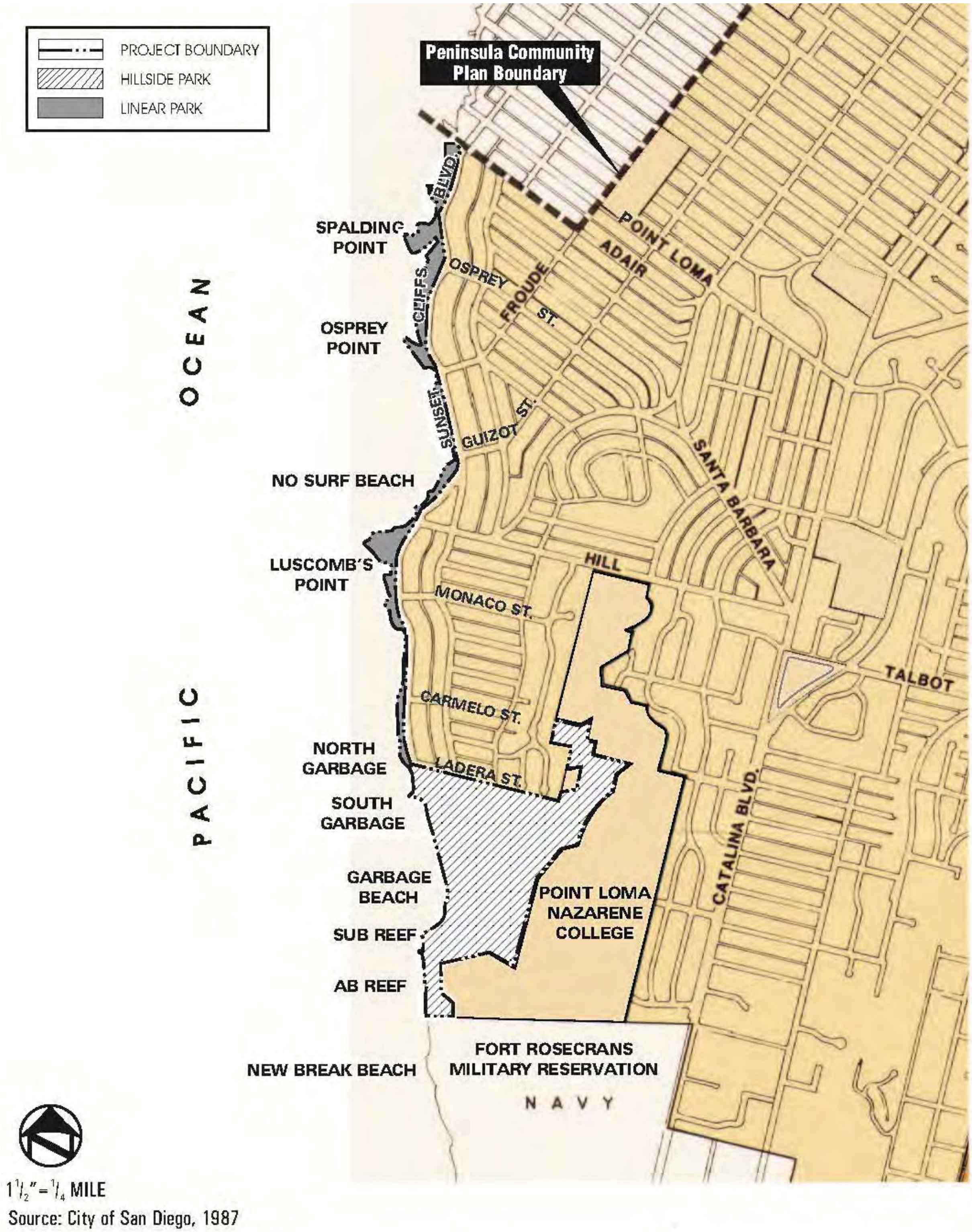
The Master Plan divides the Park into two sections. The 18-acre Linear Park section includes the natural cliff and street parking areas that extend approximately 1.25 miles south to the Sunset Cliffs Boulevard/Ladera Street intersection. The 50-acre Hillside Park includes the natural cliff and hillside area that extends from the Sunset Cliffs Boulevard/Ladera Street intersection approximately 0.5 mile south to the northern border of the military reservation. The location of the project on a regional and local context are illustrated in Figures ES-1, ES-2 and ES-3.

Land uses within the Linear Park consist of parking areas and pedestrian trails with recreational uses generally consisting of jogging, surfing, fishing, tide pooling, and bicycling. The Hillside Park supports a combination of passive and active recreation uses as well as private structures. The Hillside Park is primarily used by Park visitors for passive recreation such as surfing, hiking, and jogging. The 1.4-acre athletic field in Hillside Park has been vacated from active sports use but supports other active recreation such as dog walking, Frisbee games, and unorganized neighborhood Park use.

Figure ES-1 Regional Map



ES-2 Vicinity Map



ES-3 Aerial Photograph

The SCNP Master Plan provides recommendations and guidelines for land uses proposed within the Park with a primary goal to:

> *"Create a park where people can enjoy San Diego's natural coastal environment as it once was, free from the effects of man and intended to inspire the user to reflect on the grandeur of the sea, and beauty of the cliffs that are Point Loma," Sunset Cliffs Natural Park Council (SCNPC).*

To accomplish this goal, the following objectives and/or planning principles were forwarded as guidelines to direct Park planning decisions regarding development preservation:

- Do no harm; protect, conserve and enhance.
- Maintain focus on the unique coastal resources.
- Allow public access with minimal environmental impacts.
- Maintain planning integrity/strategy for resource preservation.
- Restore areas of neglect and damage to their previous condition and visual quality.

The Master Plan land use recommendations and guidelines generally consist of project elements that stop the current erosion problems in the park, restore the site to a more natural state, and allow the public to safely enjoy the natural resources in the Park. Some of the major project elements in the Master Plan include: a comprehensive drainage plan; a native plant preservation and revegetation program; a continuous system of marked pedestrian trails with observation points, signage, and railings in selected places; construction of access to Garbage Beach; restoration of the existing Ladera Street stairway; and demolition of the Life Estates.

This drainage study is the initial step towards advancement of the SCNP Master Plan. The drainage study included geotechnical investigations, a shoreline and bluff erosion report, a hydrology analysis, hydraulic analysis for drainage inlet and pipeline sizes, extensive alignment alternatives analysis, and biological constraints. The primary focus was the hydrology and hydraulic analysis and alignment alternatives analysis.

Hillside Park

The final selected drainage improvement recommendations for Hillside Park are shown on Figure ES-4 and are described as follows:

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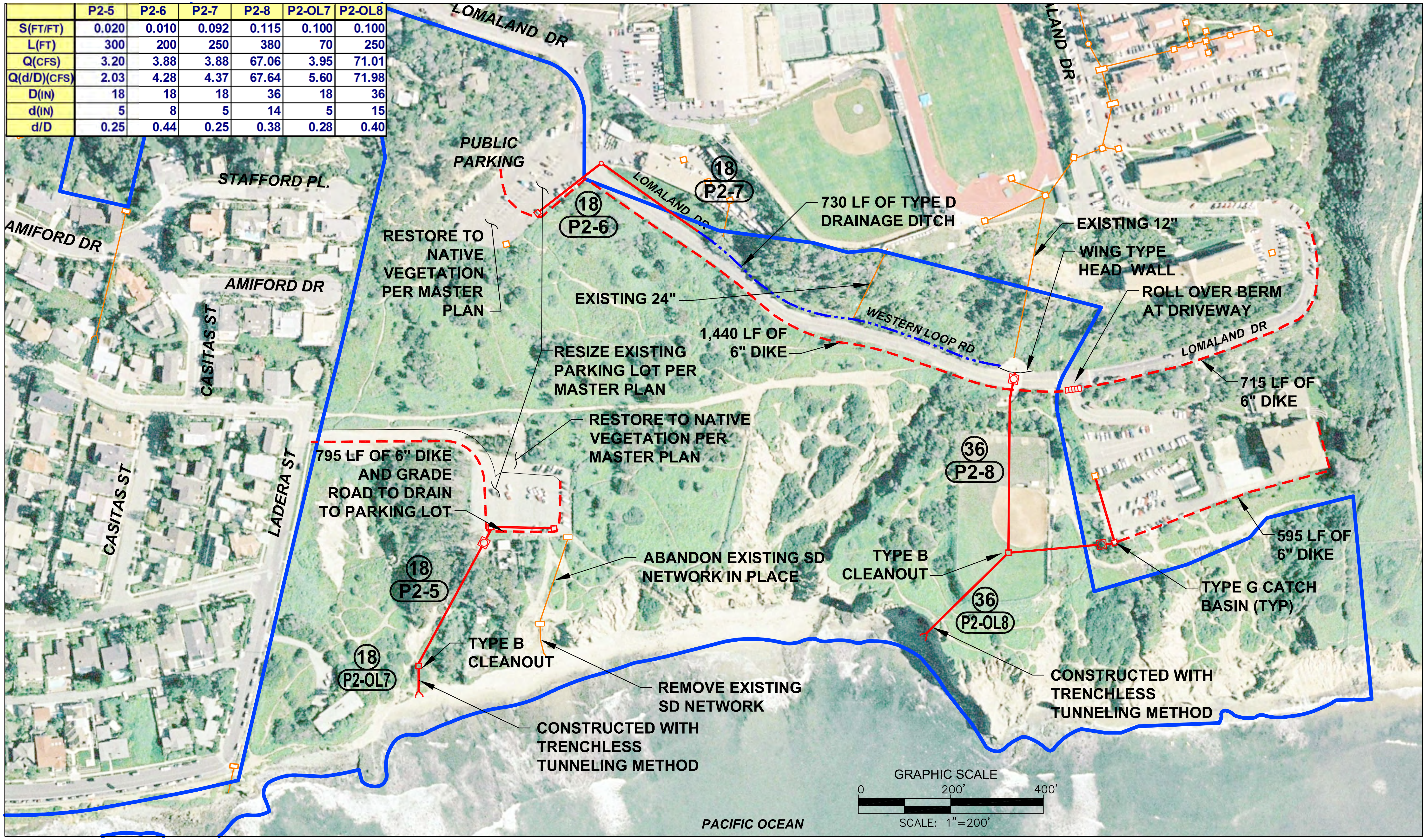
	P2-5	P2-6	P2-7	P2-8	P2-OL7	P2-OL8
S(FT/FT)	0.020	0.010	0.092	0.115	0.100	0.100
L(FT)	300	200	250	380	70	250
Q(CFS)	3.20	3.88	3.88	67.06	3.95	71.01
Q(d/D)(CFS)	2.03	4.28	4.37	67.64	5.60	71.98
D(IN)	18	18	18	36	18	36
d(IN)	5	8	5	14	5	15
d/D	0.25	0.44	0.25	0.38	0.28	0.40



LEGEND:

	BASIN BOUNDARY		PROPOSED STORM DRAIN		PROPOSED STORM DRAIN TREATMENT		PROPOSED CURB INLET
	RIDGE LINE		EXISTING STORM DRAIN		EXISTING CATCH BASIN	48	PROPOSED PIPE SIZE
	PARK BOUNDARY		DIRECTION OF FLOW		EXISTING STORM DRAIN CLEANOUT	P2-1	ALTERNATIVE NO. - PIPE NUMBER(TYP)

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760.942.5147 Fax 760.632.0164

Master Drainage Exhibit
Sunset Cliffs
Selected Alternative: Hillside Park

FIGURE
ES-4

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The Hillside Park Selected Alternative consists of an 18-inch drainage pipeline and outfall conveying storm water from the lower parking lot at the north end of the park and a second 36-inch drainage pipeline network collecting storm water from Lomaland Drive/Western Loop Road, portions of the PLNU campus and the upper university parking lot and discharging through an outfall at the south end of the park. In addition to these primary pipelines the Hillside Park Selected Alternative includes a curb and brow ditch on Lomaland Drive/Western Loop Road and improvements to and a drain line from the PLNU Young Hall parking area. The curb and brow ditch project element on Lomaland Drive/Western Loop Road should be considered high priority due to the relatively low cost of construction, ease of permitting and effectiveness of erosion reduction. It has been proposed as an erosion prevention project to be constructed by PLNU.

Table ES-1 Selected Hillside Park Alternative

ITEM NO.	DESCRIPTION	UNIT	QUANTITY	UNIT COST	ITEM TOTAL
1	6" AC Dike (Type A)	LF	3545	$ 12	$ 42,540
2	Catch Basin (Type G)	EA	5	$ 7,900	$ 39,500
3	18" Storm Drain (Water Tight Joints)	LF	1310	$ 130	$ 170,300
4	36" Storm Drain (Directional Bore)	LF	630	$ 1,300	$ 819,000
5	Cleanout (Type B)	EA	2	$ 6,968	$ 13,936
6	Outfall with Energy Dissipater	EA	2	$ 50,000	$ 100,000
7	Permanent Water Quality BMP	EA	3	$ 7,000	$ 21,000
8	Remove Existing Storm Drain	LF	90	$ 60	$ 5,400
8	Pavement Restoration	SF	2500	$ 7	$ 17,500
9	Concrete Drainage Ditch (Type D)	LF	730	$ 26	$ 18,980
10	Clear & Grub	SF	72000	$ 2	$ 108,000
11	Grading	CY	2052	$ 36	$ 73,872
12	Mobilization, BMPs, Bonds & Cleanup - 10%	LS	1	$ 143,003	$ 143,003
CONSTRUCTION Sub-total Selected Hillside Park Alternative:					$ 1,573,031
Contingency - 20 %					$ 314,606
CONSTRUCTION TOTAL Selected Hillside Park Alternative:					$ 1,887,637
	Mitigation:	AC	2	$ 200,000	$ 400,000
	Soft Costs (Design, Permitting, CM, Admin)				$ 915,055
	PROGRAM COST Selected Hillside Park Alternative:				$ 3,202,692

Linear Park

The final selected drainage improvement recommendations for Linear Park are shown on Figures ES-5 and ES-6 and are described as follows:

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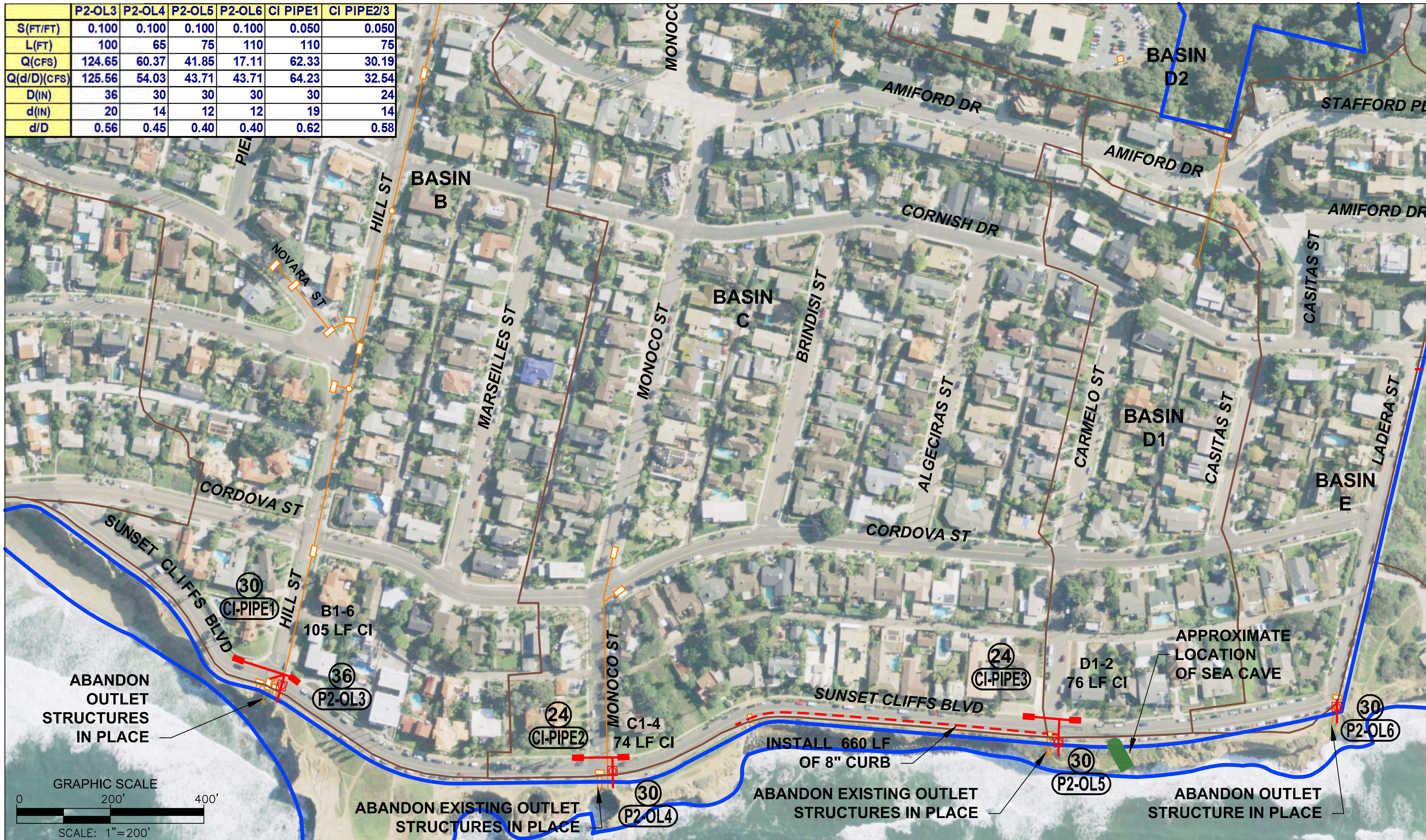
	P2-OL3	P2-OL4	P2-OL5	P2-OL6	CI PIPE1	CI PIPE2/3
S(FT/FT)	0.100	0.100	0.100	0.100	0.050	0.050
L(FT)	100	65	75	110	110	75
Q(CFS)	124.65	60.37	41.85	17.11	62.33	30.19
Q(d/D)(CFS)	125.56	54.03	43.71	43.71	64.23	32.54
D(IN)	36	30	30	30	30	24
d(IN)	20	14	12	12	19	14
d/D	0.56	0.45	0.40	0.40	0.62	0.58

SEE ALT 4 FIGURE 5



LEGEND:

BASIN BOUNDARY	PROPOSED STORM DRAIN	PROPOSED STORM DRAIN TREATMENT
RIDGE LINE	EXISTING STORM DRAIN	EXISTING CATCH BASIN
PARK BOUNDARY	DIRECTION OF FLOW	EXISTING STORM DRAIN CLEANOUT
PROPOSED CURB INLET	48 PROPOSED PIPE SIZE	P2-1 ALTERNATIVE NO. - PIPE NUMBER(TYP)

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Master Drainage Exhibit
Sunset Cliffs - Linear Park
Selected Alternative: BASIN B - E

FIGURE ES-5

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	P2-1	P2-2	P2-3	P2-4	P2-OL1	P2-OL2
S(FT/FT)	0.014	0.005	0.023	0.011	0.100	0.100
L(FT)	95	450	510	95	110	65
Q(CFS)	30.95	11.45	21.38	20.61	42.40	41.99
Q(d/D)(CFS)	32.25	12.08	21.84	21.38	43.71	43.71
D(IN)	30	30	24	30	30	30
d(IN)	18.00	13.50	14.00	15.00	12.00	12.00
d/D	0.60	0.45	0.58	0.50	0.40	0.40

LEGEND:

- BASIN BOUNDARY
- RIDGE LINE
- PARK BOUNDARY
- PROPOSED STORM DRAIN
- EXISTING STORM DRAIN
- DIRECTION OF FLOW
- PROPOSED STORM DRAIN TREATMENT
- EXISTING CATCH BASIN
- EXISTING STORM DRAIN CLEANOUT
- PROPOSED CURB INLET
- 48 PROPOSED PIPE SIZE
- P2-1 ALTERNATIVE NO. - PIPE NUMBER(TYP)

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Master Drainage Exhibit
Sunset Cliffs - Linear Park
Selected Alternative: BASIN X & A

FIGURE ES-6

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The Linear Park Selected Alternative consists of six separate small drainage network elements feeding six outfalls. With the exception of the proposed outfall at the foot of Froude Street, all proposed outfalls in the linear park are located to replace existing outfalls. The proposed outfall at the foot of Froude Street will reduce flow at the at the Osprey Street outfall. The Froude Street outfall was selected as a preferred location since there is no public access at this location and the beach is already covered with large rip rap. Each of these six elements could be constructed independently as funding allows. However, it is recommended that the improvements are bundled to the largest extent possible due to the permitting considerations of working in a coastal environment, specialized nature of the construction methods and the six drainage networks are small. It would be significantly less efficient and more costly to bid, award and construct each one independently.

Table ES-2 Linear Park Alternatives

ITEM NO.	DESCRIPTION	UNIT	QUANTITY	UNIT COST	ITEM TOTAL
Selected Alternative: Linear Park Basin X & A					
1	Curb Inlet (Type C)	EA	9	$ 7,900	$ 71,100
2	8" Curb & Gutter (Type H)	LF	230	$ 33	$ 7,590
3	18" Storm Drain (Water Tight Joints)	LF	250	$ 130	$ 32,500
4	24" Storm Drain (Water Tight Joints)	LF	510	$ 150	$ 76,500
5	30" Storm Drain (Water Tight Joints)	LF	640	$ 164	$ 104,960
6	30" Storm Drain (Directional Bore)	LF	175	$ 1,300	$ 227,500
7	Cleanout (Type A)	EA	2	$ 6,968	$ 13,936
8	Outfall with Energy Dissipater	EA	2	$ 40,000	$ 80,000
9	Permanent Water Quality BMP	EA	2	$ 7,000	$ 14,000
10	Pavement Restoration	SF	9500	$ 7	$ 66,500
11	Mobilization, BMPs, Bonds & Cleanup - 10%	LS	1	$ 69,459	$ 69,459
CONSTRUCTION Sub-total Selected Linear Park Basin X & A:					$ 764,045
Contingency - 20 %					$ 152,809
CONSTRUCTION TOTAL Selected Linear Park Basin X & A:					$ 916,854
Selected Alternative: Linear Park Basin B - E					
1	Curb Inlet (Type C)	EA	6	$ 7,900	$ 47,400
2	8" Curb & Gutter (Type H)	LF	660	$ 33	$ 21,780
3	18" Storm Drain (Water Tight Joints)	LF	0	$ 130	$ ---
4	24" Storm Drain (Water Tight Joints)	LF	150	$ 150	$ 22,500
5	30" Storm Drain (Water Tight Joints)	LF	110	$ 164	$ 18,040
6	30" Storm Drain (Directional Bore)	LF	250	$ 1,200	$ 300,000
7	36" Storm Drain (Directional Bore)	LF	100	$ 1,300	$ 130,000
8	Outfall with Energy Dissipater	EA	4	$ 40,000	$ 160,000
9	Permanent Water Quality BMP	EA	4	$ 7,000	$ 28,000
10	Pavement Restoration	SF	3610	$ 7	$ 25,270
11	Mobilization, BMPs, Bonds & Cleanup - 10%	LS	1	$ 75,299	$ 75,299
	CONSTRUCTION Sub-total Selected Linear Park Basin B & E:				$ 828,289
	Contingency - 20 %				$ 165,658
	CONSTRUCTION TOTAL Selected Linear Park Basin B & E:				$ 993,947
	CONSTRUCTION TOTAL Selected Linear Park Alternative:				$ 1,910,800
	Soft Costs (Design, Permitting, CM, Admin)				$ 764,320
	PROGRAM COST Selected Linear Park Alternative:				$ 2,675,120

The drainage improvements for Hillside and Linear Park can be constructed separately or together as one project. Combining both Hillside and Linear Park drainage improvements into one project will provide several benefits including:

- A probable savings in construction from an economy of scale;

- Attraction of more and larger contractor's with greater trenchless capabilities and challenging coastal bluff construction experience potentially reducing construction duration and construction change orders;
- A single permitting program instead of separate permitting programs in a challenging coastal permitting environment.

One of the advantages of the selected alternative for the Hillside Park is that it can easily be constructed in multiple stand-alone phases as funding allows. The phases can be described as follows:

1. Brow ditch and curb on Lomaland Drive/Western Loop Road. Potentially to be constructed by PLNU.
2. Lower parking lot improvements, curb, drain and outfall.
3. Main storm drain pipeline from head of Culvert Canyon.
4. Young Hall parking lot curb, improvements and drain.

Other Erosion Considerations

There are several other significant sources of erosion in the Park's. Construction of a new drainage system is an important step towards reducing damaging erosion in the park. Other sources of erosion include:

- Pedestrian, bicycle and canine traffic erosion
- Lack of native vegetation to protect and anchor soil
- Perforation of the bluffs from burrowing rodents
- Concentrated runoff from parking lots

These erosion sources and potential remedies are described in greater detail in Section 2 of the report.

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1 INTRODUCTION

1.1 Project Description

Sunset Cliffs Natural Park (SCNP) constitutes a unique coastal environment in San Diego County. People have gathered at this special location over the years to seek relief from urban living, enjoy the coastal bluff environment and reflect on evening sunsets. SCNP is located approximately five miles west of downtown San Diego along the western shoreline of the Point Loma Peninsula. The Park is bordered to the north by the community of Ocean Beach delineated by the Adair Street/Sunset Cliffs Boulevard intersection. The site is bordered to the west by the Pacific Ocean and to the east by Sunset Cliffs Boulevard, single-family residential uses, and Point Loma Nazarene University (PLNU). The site is bordered to the south by the Fort Rosecrans Military Reservation.

The SCNP Master Plan divides the Park into two sections. The 18-acre Linear Park section includes the natural cliff and street parking areas that extend approximately 1.25 miles south from the northern border to the Sunset Cliffs Boulevard/Ladera Street intersection. The 50-acre Hillside Park includes the natural cliff and hillside area that extends from the Sunset Cliffs Boulevard/Ladera Street intersection approximately 0.5 mile south to the northern border of the military reservation. The location of the project on a regional and local context are illustrated in Figures ES-1, ES-2 and ES-3.

Land uses within the Linear Park consist of parking areas and pedestrian trails with recreational uses generally consisting of jogging, surfing, fishing, tide pooling, and bicycling. This Hillside Park supports a combination of passive and active recreation uses as well as private structures. The Hillside Park is primarily used by Park visitors for passive recreation such as surfing, hiking, and jogging. The SCNP Master Plan provides recommendations and guidelines for land uses proposed within the Park with a primary goal to:

> "*Create a park where people can enjoy San Diego's natural coastal environment as it once was, free from the effects of man and intended to inspire the user to reflect on the grandeur of the sea, and beauty of the cliffs that are Point Loma,*" *Sunset Cliffs Natural Park Council (SCNPC).*

To accomplish this goal, the following objectives and/or planning principles were forwarded as guidelines to direct Park planning decisions regarding development preservation:

- Do no harm; protect, conserve and enhance.
- Maintain focus on the unique coastal resources.
- Allow public access with minimal environmental impacts.
- Maintain planning integrity/strategy for resource preservation.
- Restore areas of neglect and damage to their previous condition and visual quality.

The SCNP Master Plan land use recommendations and guidelines generally consist of project elements that stop the current erosion problems in the park, restore the site to a more natural state, and allow the public to safely enjoy the natural resources in the Park. Some of the major project elements in the Master Plan include: a comprehensive drainage plan; a native plant preservation and revegetation program; a continuous system of marked pedestrian trails with observation points, signage, and railings in selected places; construction of a new public access to Garbage Beach; restoration of the existing Ladera Street stairway; and demolition of the Life Estates.

The first step towards execution of the SCMP Master Plan is a drainage study. The purpose of the drainage study is to provide engineering recommendations for conveying urban runoff and rainwater flows from upper hardscaped developed areas across the natural parks to the Pacific Ocean while eliminating harm to the SCNP from erosion caused by non-existent, under capacity or defective drainage facilities. To accomplish this goal, the construction of culverts, pipelines and outfalls is required.

1.2 Acknowledgements

Several entities should be acknowledged for their dedication to preservation and protection of the SCNP and their assistance with the completion of the Drainage Study. These entities include:

- The Sunset Cliffs Natural Park Recreation Council (SCNPRC), future generations will benefit for the determination and dedication of the SCNPRC to protecting, restoring and preserving the park for all to enjoy.
- The SCNPRC Drainage Subcommittee, the members of the SCNPRC Drainage Subcommittee deserve special recognition for providing valuable local knowledge and insight as well as guidance throughout the study preparation process. The members include:
  - Dedi Ridenour
  - Ann Swanson
  - Barbara Keiler
  - Gene Berger
- The Sunset Cliffs Association (SCA) particularly Camilla Ingram and Craig Barilotti who provided detailed technical input and astute drainage/erosion observations and documentation.
- The residents of Sunset Cliffs and Ocean Beach whose attendance and contributions at public meetings helped guide a plan of drainage solutions that take into consideration the needs of the community as a whole.
- The City of San Diego Parks and Recreation Department in particular the Project Manager Paul Jacob whose expertise in civil engineering and practical approach was invaluable to the challenging task of managing the multiple party interests and opinions.

Without Mr. Jacob's strong management, completion of a drainage study that met all parties' expectations and needs would have been challenging.

- The City of San Diego Parks and recreation Department – ***"We enrich lives through quality parks and programs"***

1.3 How to use this document

The executive summary describes the selected drainage solutions including sizes and locations ready to be incorporated into final construction documents by a civil engineer. The executive summary also mentions other considerations for reducing erosion in the park.

For the individual who wishes to understand how the recommended facilities were developed and selected there is a Drainage Alternatives & Constraints section that describes the facilities is greater detail including: calculation methods, construction methods, construction schedule and estimated construction cost opinions.

Other Sections in the Drainage Study include:

- Biological Resources and Constraints
- CEQA and Regulatory Requirements
- Construction Issues
- Monitoring Program

For detailed information and analysis created for and referred to during the drainage solution study process the following appendices are provided:

- Appendix A - Geotechnical Report
- Appendix B - Hydrology and Hydraulic Analysis
- Appendix C - Shoreline and Bluff Protection Report
- Appendix D - Sunset Cliffs Association Drainage Conditions and Recommendations
- Appendix E - Recent Erosion and Mass Wasting Observed in Sunset Cliffs Natural Park

1.4 Final Design Scope of Work

Completion of the Sunset Cliffs Natural Park Drainage Study provides the basis for the final design scope of work of the drainage infrastructure improvements needed to protect the Linear and Hillside Parks from future erosion due to urban runoff from upslope developed areas during storm events. The completed Drainage Study represents a critical step in the park protection process by identifying preferred drainage infrastructure routes and configurations that can be provided to the design engineer for a specific project scope of work and fee estimate.

Specific scope of work items that should be included in the drainage infrastructure final design include:

- Aerial and detailed topographical ground surveying
- Park boundary survey
- Geotechnical investigation and report including soil borings along the pipeline alignment and mapping of existing sea caves
- Mapping and potholing of existing utilities
- Plan and profile drawings of pipeline alignments
- Hydraulic Grade Line (HGL) calculations for verification of pipeline diameter
- Final design of energy dissipating outfall structures
- Bluff stabilization design
- Final design of permanent water quality Best Management Practices (BMPs)
- Preparation of Storm Water Pollution Protection Plan (SWPPP)
- Grading plans
- Construction equipment access plan
- Project specifications and bidding documents
- Detailed cultural resource survey and impact analysis
- Detailed biological survey and impact analysis
- Resource agency permitting and mitigation plans
- Coastal Development permitting
- California Environmental Quality Act (CEQA) documentation tiered off of the Sunset Cliffs Natural Park Master Plan Environmental Impact Report (EIR)
- Public Outreach support

2 DRAINAGE ALTERNATIVES & CONSTRAINTS

To refine the drainage alternatives down to the two alternatives presented in this section the design team took into account constraints related to public acceptance, source location and intensity of rain water flow, construction cost, SCNP park boundary, constructability issues, biological constraints and geotechnical constraints. Preferred alternatives for the Hillside Park area are shown on Figures 2-1 and 2-2. Preferred alternatives for the Linear Park area are shown on Figures 2-3, 2-4, 2-5 and 2-6. These figures will be found at the end of this section. The selected alternatives are presented in the Executive Summary on Figures ES–4, ES–5 and ES-6.

2.1 Geotechnical Analysis and Testing

The geotechnical investigation was primarily focused on identify locations and significance of perched water areas. Significant perched water areas may affect bluff stability. As a part of the geotechnical investigations, ground water monitoring wells were constructed. Long term monitoring of these wells will provide an indication of a rise or fall in the perched water table areas. Construction of the recommended drainage improvements along with reducing other groundwater sources such as excess landscape irrigation and water pipeline leaks will potentially reduce perched water tables and lower the perched water table threat to bluff stability. At this stage in the SCNP drainage improvement program there are no plans for the invasive construction of a perched water table sub-drain system. The complete Geotechnical Report is provided in Appendix A of this report.

2.2 Hydrology Study

To determine the peak runoff rates of storm water (Q) and points of concentration, a hydrology analysis was completed in accordance with the 2003 San Diego County Hydrology Manual using rational and modified rational methods. The results of the hydrology analysis were used to size the pipeline improvements and locate and size drainage inlets. The complete Hydrology and Hydraulic Analysis report is provided in Appendix B of this report.

2.3 Drainage Analysis and Design Criteria

2.3.1 Drainage Analysis

Significant drainage from upslope hardscaped impervious areas enters the parks at several locations with erosive velocity. Erosion gullies have formed in these locations. Sediment transportation during rain events further scours the gullies. Some of the gullies are large and represent a danger to park visitors. The locations where run off enters the parks or originates from impervious areas of the park were identified using several methods including evaluation of topographical mapping, historical photographs, visual evidence of erosion, multiple site visits (dry and rainy conditions) and information provided by residents. Two detailed reports providing and inventory of the drainage sources and areas of erosion were prepared by the Sunset Cliffs Association (SCA). 1) Sunset Cliffs Association Drainage Conditions and

Recommendations Sunset Cliffs Natural Park, March 22, 2007. 2) Recent Erosion and Mass Wasting Observed in Sunset Cliffs Natural Park, February 15, 2011. These reports are provided in Appendix D and E of this report. The most significant drainage courses causing erosion for the Hillside and Linear Park are summarized below:

Hillside Park from north to south:

- Concentrated runoff from the upper parking lot
- Concentrated runoff from the lower parking lot
- Overflow from Western Loop Road/Lomaland Drive near the 18-inch pipe from PLNU
- Overflow from Western Loop Road/Lomaland Drive near the 24-inch pipe from PLNU
- Concentrated runoff from the concrete storm water convergence structure and Arizona Crossing at the low point of Western Loop Road/Lomaland Drive near the head of Culvert Canyon
- Concentrated runoff from the Young Hall parking lot

Linear Park from north to south:

- Denuded unvegetated soil south of Adair Street
- Concentrated runoff from the parking lot south of Adair Street
- Concentrated runoff from the parking lot north of Osprey Street
- Concentrated runoff from the parking lot south of Osprey Street
- Insufficient drainage facilities at the foot of Osprey Street
- Concentrated runoff from the parking lot north of Froude Street
- Insufficient drainage facilities at the foot of Hill Street
- Denuded unvegetated soil at Luscomb's Point
- Insufficient drainage facilities at the foot of Carmelo Street

2.3.1.1 Other Erosion Considerations

There are several other significant sources of erosion in the Park's. Construction of a new drainage system is an important step towards reducing damaging erosion in the park. Other sources of erosion and potential remedies include:

- Pedestrian, Bicycle and Canine erosion is pervasive throughout the park due to the lack of a site specific designed trail system. A new trail system is currently being planned. The trail system should be well defined with barrier systems that discourage off trail activity. The trails should attempt to follow existing ground contours. Steeply sloped trails will create erosion issues. Signs should be posted to educate park users of the importance of staying on the trails.

- Loss of native vegetation due to pedestrian traffic or concentrated water runoff erosion removes the natural plant barrier protecting the soil from rainfall and reduces the soil permeability increasing runoff and eventually erosion. A native plant restoration planting program should be implemented. The newly planted areas should be fenced off to protect the plants and irrigated for the first three to five years until the native plants are established. Native plant restoration may be used as a source of mitigation for the disturbance associated with drainage facility pipeline construction.
- Burrowing rodents/mammals have perforated the bluffs particularly near the Young Hall area. These burrows weaken the soil and provide new pathways for water to erode the bluffs. Rodent resistant self-closing trash containers should be used at the PLNU facilities and throughout the park to minimize the rodent food supply. Raptor perches can also be installed in strategic locations to help control the rodent population. Signs should be posted to educate park users of the importance of hauling off food waste or placing it in appropriate receptacles.
- Excessive irrigation or water pipe leaks can adversely affect bluff stability by lubricating the contact surface between the weaker erodible Bay Point Formation at the surface of the park and the lower less pervious Point Loma Foundation. This can lead to mass wasting and block falls. Public education and incentives for careful irrigation management and xeriscape gardens along with testing of key distribution pipelines will help minimize the water lubrication of the contact areas.
- Parking lots should be designed to minimize run off during rain events. Some Low Impact Design (LID) features encouraging water percolation may or may not be feasible due to the soil type, slope and bluff proximity. Location specific analysis will be needed to comply with current stormwater policies and practices in effect at the time projects are implemented. Parking lots in the linear park area should be design to drain towards Sunset Cliffs Boulevard not towards the ocean and over the bluffs.

2.3.2 Design Criteria

Design Criteria for the inlet sizing and pipeline sizing is based on the 2003 San Diego County Hydrology Manual using a 50-year storm event. A detailed explanation of the inlet sizing is provided in the Hydrology and Hydraulic report in Appendix B of this report. Capacity for the proposed pipelines is based on a slope consistent with the slope of the ground surface over the pipeline. During final design the actual slope of the pipelines will be determined based on depth of existing connecting drainage facilities, actual surveyed ground profile, existing utilities and other factors.

As noted on the Selected Alternative figures, water pollution control devices should be installed on all inlet and/or outlet structures to comply with the latest urban stormwater permit requirements consistent with the Regional Water Quality Control Board.

Low Impact Design (LID) practices should be followed for the design of the new parking lots, drainage improvements and other park improvements. A common theme of low impact design is infiltration. Infiltration as a Best Management Practice (BMP) is not recommended by the City of San Diego Storm Water Standards under the following conditions:

- High groundwater
- Proximity to contaminated soil
- Engineered Fill
- Low infiltration rate
- Clay soils
- Impermeable Bedrock
- Slopes steeper that 25% (4 to 1)
- Slopes prone to instability

Many of these conditions are present in various locations of the Linear and Hillside Parks. As such, a detailed analysis on a location specific basis will be required before infiltration solutions can be implemented.

2.4 Shoreline Solutions

Part of the SCNP drainage study included an investigation into the SCNP bluff stability and recommendations for long term bluff protection measures. There is no community support for non-natural bluff stability structures. Therefore, at this stage in the SCNP drainage improvement program there are no plans for moving forward with bluff stability improvements identified in the Shoreline Bluff Erosion Protection report. The complete Shoreline Bluff Erosion Protection report is provided in Appendix C of this report.

2.5 Biological Resources Constraints Analysis

2.5.1 Relationship to MSCP

The proposed project is located within the City of San Diego Multiple Species Conservation Program (MSCP) Subarea and the Subarea's Multiple Habitat Planning Area (MHPA). Section 1.4, *Land Use Considerations*, of the Subarea Plan states that utility lines are an allowable use with the City's MHPA. Section 1.4.2, *Roads and Utilities–Construction and Maintenance Policies*, provides further directives regarding utilities located within the MHPA, which are relevant to the proposed drainage project. No habitat linkage areas are identified in the Subarea Plan that connect to Sunset Cliffs Natural Park. Additionally, the City Subarea Plan does not identify any specific MHPA guidelines that relate to Sunset Cliffs Natural Park.

Section 1.5.7, *Specific Management Policies and Directives for Urban habitat Lands*, of the Subarea Plan states that utility activities and controlling urban runoff and protecting water quality are major issues in the City's Urban Habitat Lands. To address these issues, the City Park and Recreation Department has prepared or is preparing a Natural Resources Management Plan.

The City's Land Use Adjacency Guidelines provide a list of issues to be addressed for projects within or adjacent to the MHPA. The proposed projects should be consistent with the guidelines, as summarized below:

- Removal and reconfiguration of the parking areas will be within the footprint of the existing parking lots. Drainage catchment areas will not be changed significantly from existing conditions.
- No toxic chemicals are planned for use during project implementation
- Lighting proposed in the park is for the parking area only. All lighting will face inward to parking areas only and will not effect habitat.
- The only noise impacts will be temporary, occurring during construction or restoration of park facilities.
- Appropriate barriers will be installed to direct public access from sensitive resources. These barriers will not restrict or impeded wildlife movement.
- Plant species used for revegetation will be native species appropriate to the area.

2.5.2 Vegetation Communities

Vegetation communities considered sensitive by the *City of San Diego MSCP Subarea Plan* include those listed as Tier I through Tier III in the MSCP. Based upon the *Biological Resource Report, Sunset Cliff's Natural Park*, *LDR 91-0644* (Dudek 2003), the vegetation communities that could potentially be impacted by the proposed drainage improvements are provided in Table 1 by each proposed project alternative and designated tier. The proposed project is located within the Multiple Habitat Planning Area (MHPA) and, for purposes of this analysis, it assumed that the proposed mitigation will be within the Sunset Cliff's Natural Park in MHPA. Based upon these assumptions, the mitigation required by the City of San Diego also is shown in Table 2-1.

Table 2-1 Potential Vegetation Community Impacts for Each Proposed Project Alternative

Vegetation Type	Tier	Alternative I: Hillside Park	Alternative 2: Hillside Park SELECTED ALTERNATIVE	Alternative 1: Linear Park	Alternative 2: Linear Park SELECTED ALTERNATIVE
Tier I (Mitigation Required at a 2:1 Ratio)					
Disturbed Southern Coastal Bluff Scrub	I	—	—	X	X
Disturbed Southern Maritime Chaparral	I	—	—	—	—
Cactus Scrub	I	X	X	X	X
Unvegetated Sandstone	I	X	—	X	X
Cliff Faces, Beach and Rocky Shore	I	X	—	X	X
Tier II (Mitigation Required at a 1:1 Ratio)					
Coastal Sage Scrub	II	X	—	—	—
Disturbed Coastal Sage Scrub	II	X	—	—	—
Restored/Coastal Sage Scrub	II	X	—	—	—
Tier IV (No Mitigation Required)					
Developed Land	IV	X	X	—	—
Ruderal Habitat	IV	X	X	X	X
Giant Reed-Dominated Habitat	IV	—	X	—	—
Eucalyptus Revegetated Area	IV	—	X	—	—
Disturbed Habitat	IV	—	—	—	X

2.5.3 Special-Status Plants

According to the *Biological Resource Report, Sunset Cliff's Natural Park, LDR 91-0644* (Dudek 2003), no plant species listed or proposed for listing as threatened or endangered by the USFWS or CDFG were identified on site, however, additional spring surveys are necessary in order to confirm the absence of rare plants because the rare plant surveys were conducted outside the appropriate season. Five plant species recognized as special-status by the California Native Plant Society (CNPS) were observed on the project site and will be avoided by the proposed alternatives evaluated.

A focused spring survey is required for special-status plants. Species to be surveyed for during the appropriate time of year include Shaw's agave, aphanisma, Del Mar manzanita, Coulter's saltbush, south coast saltbush, Nevin's barberry, golden-spined cereus, seaside calandrinia, Lewis' evening-primrose, wart-stemmed ceanothus, Orcutt's spine flower, sea dahlia, San Diego sand aster, short-leaved live-forever, variegated dudleya, coast wallflower, cliff spurge, San Diego barrel cactus, spiny rush, Nuttall's lotus, snake cholla, short-lobed broomrape, Torrey pine, Nuttall's scrub oak, ashy spike moss, narrow-leaved nightshade, and San Diego County viguera, before any construction activity. If Orcutt's spineflower would be affected by construction activities a project redesign and/or mitigation would likely be required. It should be noted that this species is not a MSCP covered species, therefore a separate and subsequent authorization would be required from the USFWS pursuant to the federal Endangered Species

Act. If impacts may occur to wart-stemmed ceanothus or snake cholla, translocation or revegetation are required as a MSCP condition of coverage.

2.5.4 Special-Status Wildlife

According to the *Biological Resource Report, Sunset Cliff's Natural Park, LDR 91-0644* (Dudek 2003), the only special-status bird species recorded during the current survey was the state and federally listed endangered California brown pelican. This species was recorded flying over the ocean adjacent to the park, is not expected to use lands associated with the park (Dudek 2003), and is not expected to be impacted by the proposed drainage improvements. Two listed species, coastal California gnatcatcher and Pacific pocket mouse, were identified as potentially-occurring on the site. Focused surveys for the gnatcatcher were negative and the trapping program for the pocket mouse was negative (Dudek 2003); therefore, no impacts to California gnatcatcher or Pacific Pocket mouse as a result of the proposed drainages improvement project are expected to occur.

Two California species of special concern—the northwestern San Diego pocket mouse and San Diego desert woodrat—were observed on the park site. There is a moderate to high potential that the proposed project may impact the San Diego desert woodrat and northwestern San Diego pocket mouse. However, these impacts, if determined to be significant, can be mitigated to a level below significance.

Due to the potential for nesting of a number of raptor species, a preconstruction nesting survey for raptors should be conducted prior to removal of potential nest trees. Raptors that potentially nest within the project area include Cooper's hawk, red-shouldered hawk, and white-tailed kite. In addition, prior to construction, a survey should be conducted within 300 feet of impact areas to identify if there are potential nesting burrows for the burrowing owl.

2.5.5 Intertidal Resources

It is not anticipated that the proposed drainage improvements would significantly impact intertidal resources. Indirect impacts to the beach and intertidal area may occur during construction from erosion and sedimentation, but should be controlled to a below significant level by construction BMPs. These impacts can be avoided by employing BMPs identified in the water quality mitigation measures to keep sediments from entering the intertidal area. Sediments, rock, debris, and eroded soils as a result of project construction should be kept on site and not allowed to move into either the intertidal zone or the beach areas.

2.5.6 Jurisdictional Waters

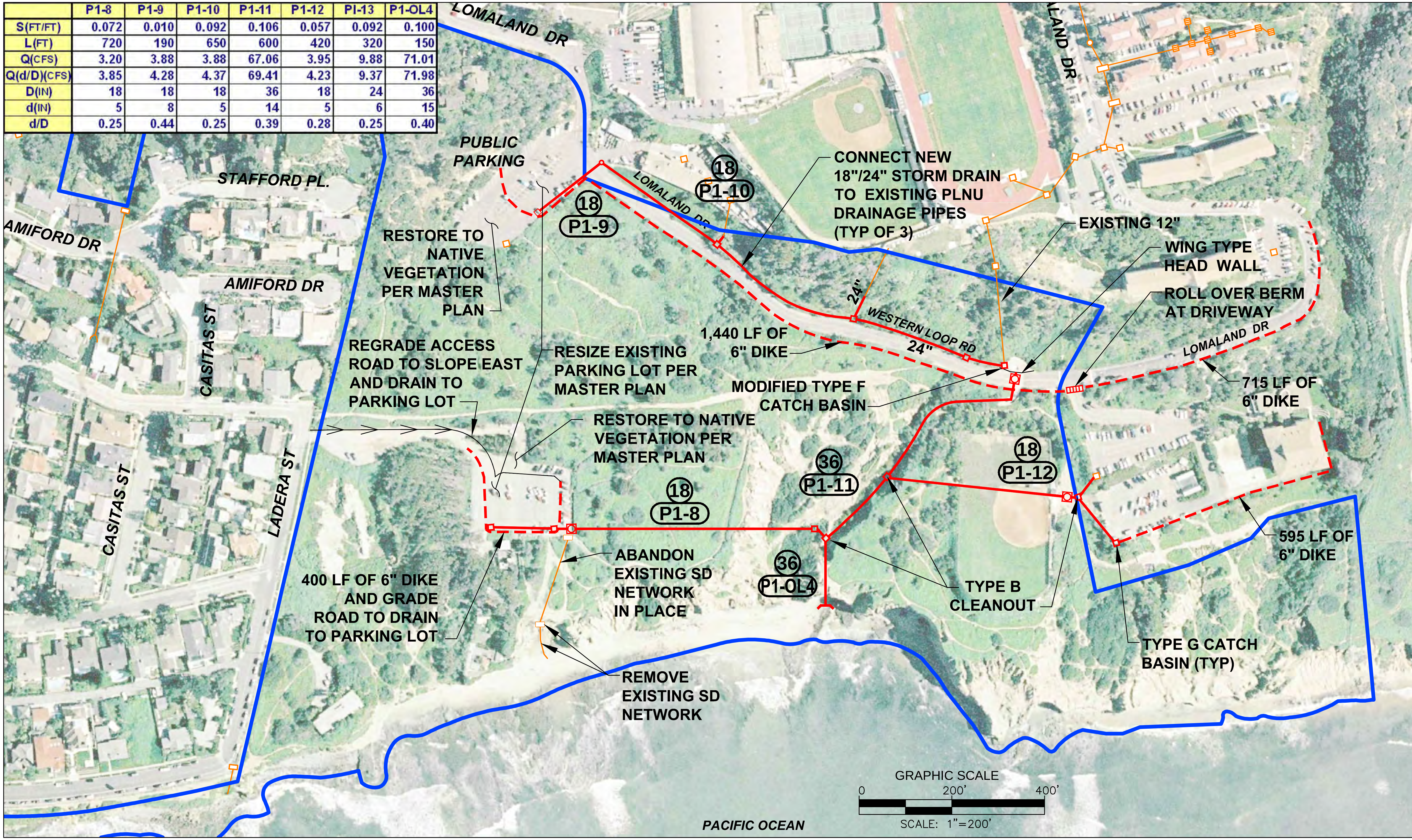
According to the *Biological Resource Report, Sunset Cliff's Natural Park, LDR 91-0644* (Dudek 2003), no wetlands occur on site. A drainage located north of the athletic field, referred to herein as the "Culvert Canyon", has been an ephemeral drainage (Dudek 2003) and is likely under the jurisdiction of the U.S. Army Corps of Engineers (ACOE), California Department of Fish and Game (CDFG), California Regional Water Quality Control Board (RWQCB), and California Coastal Commission (CCC). With respect to wetlands permitting, Alternative 1:

Hillside Park may qualify for a Nationwide Permit 43–Stormwater Management Facilities. However, it is important to note that the current Nationwide Permits expire in March 2012 and will be reissued.

2.5.7 Wildlife Corridors and Habitat Linkages

The site is isolated from other native habitats to the north and east by urban development and to the west by the Pacific Ocean. To the south, the site is connected to the adjacent Point Loma Ecological Reserve (Reserve) which is managed by the U.S. Navy. However, a chain-link fence separates the park from the Reserve on Navy property. The site does not function as, nor appear to be part of, a larger movement corridor or linkage (Dudek 2003).

	P1-8	P1-9	P1-10	P1-11	P1-12	PI-13	P1-OL4
S(FT/FT)	0.072	0.010	0.092	0.106	0.057	0.092	0.100
L(FT)	720	190	650	600	420	320	150
Q(CFS)	3.20	3.88	3.88	67.06	3.95	9.88	71.01
Q(d/D)(CFS)	3.85	4.28	4.37	69.41	4.23	9.37	71.98
D(IN)	18	18	18	36	18	24	36
d(IN)	5	8	5	14	5	6	15
d/D	0.25	0.44	0.25	0.39	0.28	0.25	0.40



LEGEND:

BASIN BOUNDARY	PROPOSED STORM DRAIN	PROPOSED STORM DRAIN TREATMENT
RIDGE LINE	EXISTING STORM DRAIN	EXISTING CATCH BASIN
PARK BOUNDARY	DIRECTION OF FLOW	EXISTING STORM DRAIN CLEANOUT
PROPOSED CURB INLET	48 PROPOSED PIPE SIZE	P1-2 ALTERNATIVE NO. - PIPE NUMBER(TYP)

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Master Drainage Exhibit
Sunset Cliffs
Preferred Alternative 1: Hillside Park

FIGURE 2-1

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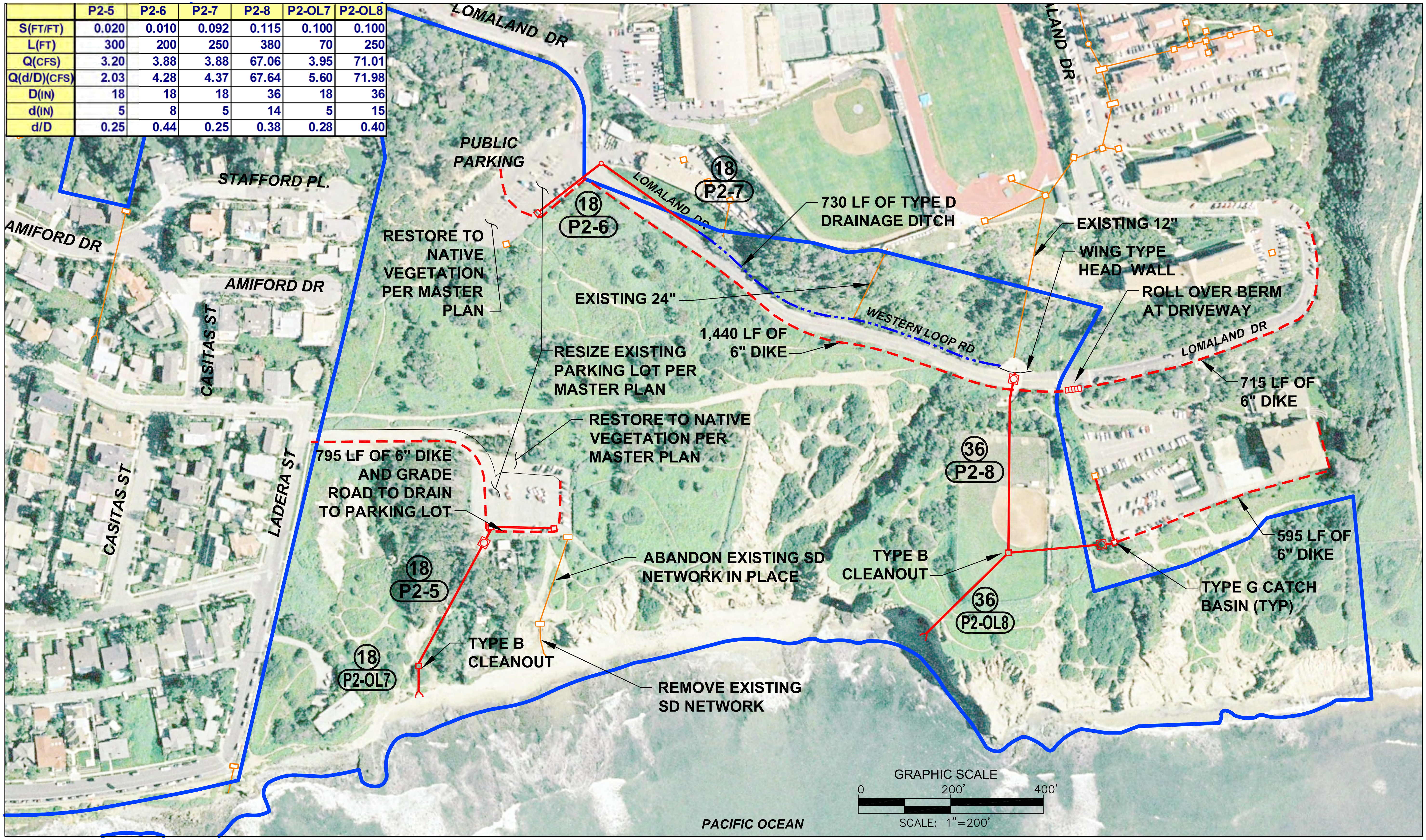
	P2-5	P2-6	P2-7	P2-8	P2-OL7	P2-OL8
S(FT/FT)	0.020	0.010	0.092	0.115	0.100	0.100
L(FT)	300	200	250	380	70	250
Q(CFS)	3.20	3.88	3.88	67.06	3.95	71.01
Q(d/D)(CFS)	2.03	4.28	4.37	67.64	5.60	71.98
D(IN)	18	18	18	36	18	36
d(IN)	5	8	5	14	5	15
d/D	0.25	0.44	0.25	0.38	0.28	0.40



LEGEND:

BASIN BOUNDARY	PROPOSED STORM DRAIN
RIDGE LINE	EXISTING STORM DRAIN
PARK BOUNDARY	DIRECTION OF FLOW
PROPOSED STORM DRAIN TREATMENT	PROPOSED CURB INLET
EXISTING CATCH BASIN	48 PROPOSED PIPE SIZE
EXISTING STORM DRAIN CLEANOUT	P2-1 ALTERNATIVE NO. - PIPE NUMBER(TYP)

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Master Drainage Exhibit
Sunset Cliffs
Preferred Alternative 2: Hillside Park

FIGURE 2-2

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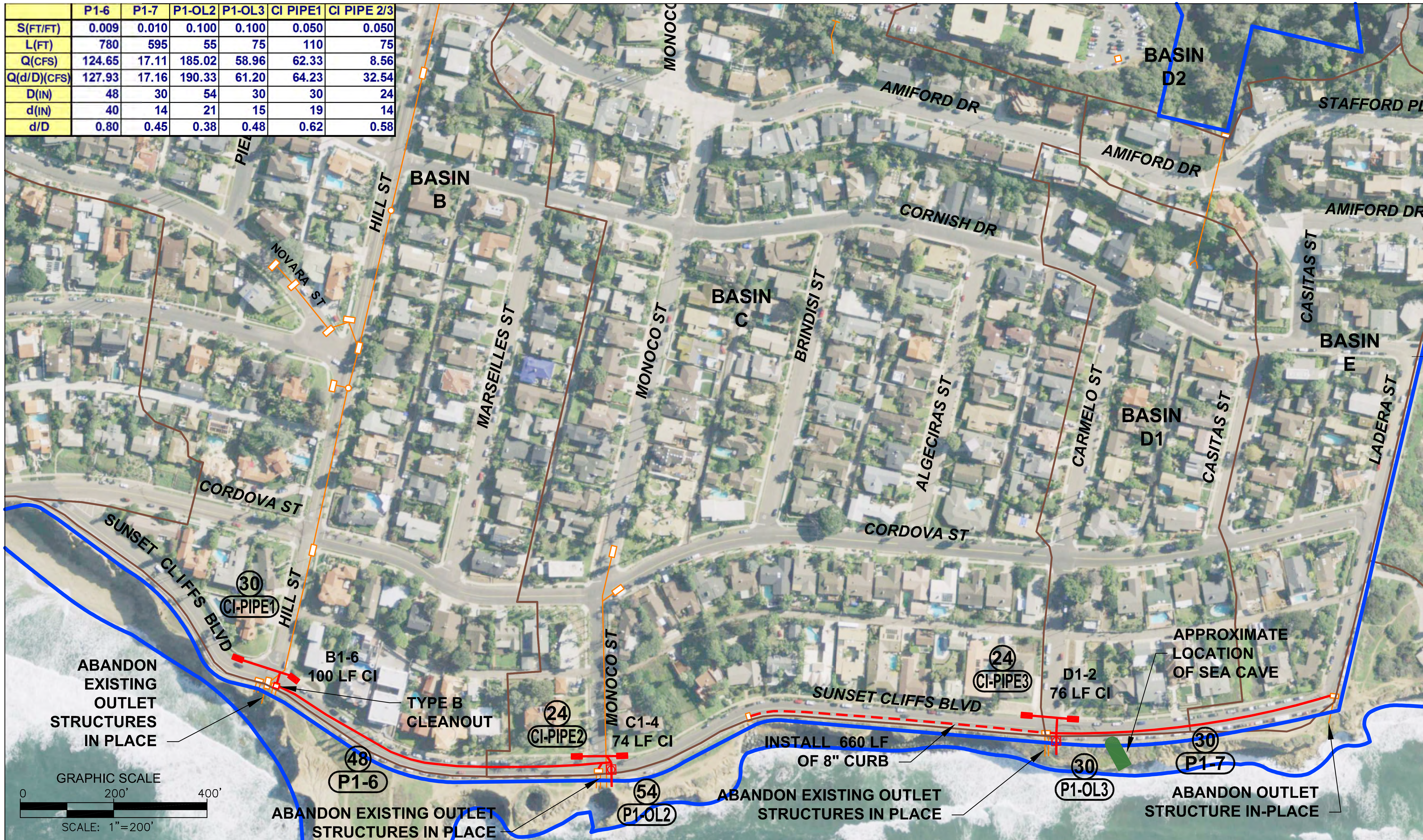
	P1-6	P1-7	P1-OL2	P1-OL3	CI PIPE1	CI PIPE 2/3
S(FT/FT)	0.009	0.010	0.100	0.100	0.050	0.050
L(FT)	780	595	55	75	110	75
Q(CFS)	124.65	17.11	185.02	58.96	62.33	8.56
Q(d/D)(CFS)	127.93	17.16	190.33	61.20	64.23	32.54
D(IN)	48	30	54	30	30	24
d(IN)	40	14	21	15	19	14
d/D	0.80	0.45	0.38	0.48	0.62	0.58



LEGEND:

- BASIN BOUNDARY
- RIDGE LINE
- PARK BOUNDARY
- PROPOSED STORM DRAIN
- EXISTING STORM DRAIN
- DIRECTION OF FLOW
- PROPOSED STORM DRAIN TREATMENT
- EXISTING CATCH BASIN
- EXISTING STORM DRAIN CLEANOUT
- PROPOSED CURB INLET
- 48 PROPOSED PIPE SIZE
- P1-2 ALTERNATIVE NO. - PIPE NUMBER(TYP)

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Master Drainage Exhibit
Sunset Cliffs - Linear Park
Preferred Alternative 1: BASIN B - E

FIGURE 2-3

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	P1-1	P1-4	P1-5	P1-9	P1-OL1
S(FT/FT)	0.014	0.005	0.011	0.023	0.100
L(FT)	95	1,080	95	500	110
Q(CFS)	30.95	50.54	20.61	21.61	84.37
Q(d/D)(CFS)	32.25	52.42	21.38	21.84	90.15
D(IN)	30	48	30	24	48
d(IN)	18.00	25.00	15.00	14.00	14.50
d/D	0.60	0.52	0.50	0.58	0.30

LEGEND:

- BASIN BOUNDARY
- RIDGE LINE
- PARK BOUNDARY
- PROPOSED STORM DRAIN
- EXISTING STORM DRAIN
- DIRECTION OF FLOW
- PROPOSED STORM DRAIN TREATMENT
- EXISTING CATCH BASIN
- EXISTING STORM DRAIN CLEANOUT
- PROPOSED CURB INLET
- 48 PROPOSED PIPE SIZE
- P1-2 ALTERNATIVE NO. - PIPE NUMBER(TYP)

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Master Drainage Exhibit
Sunset Cliffs - Linear Park
Preferred Alternative 1: BASIN X & A

FIGURE 2-4

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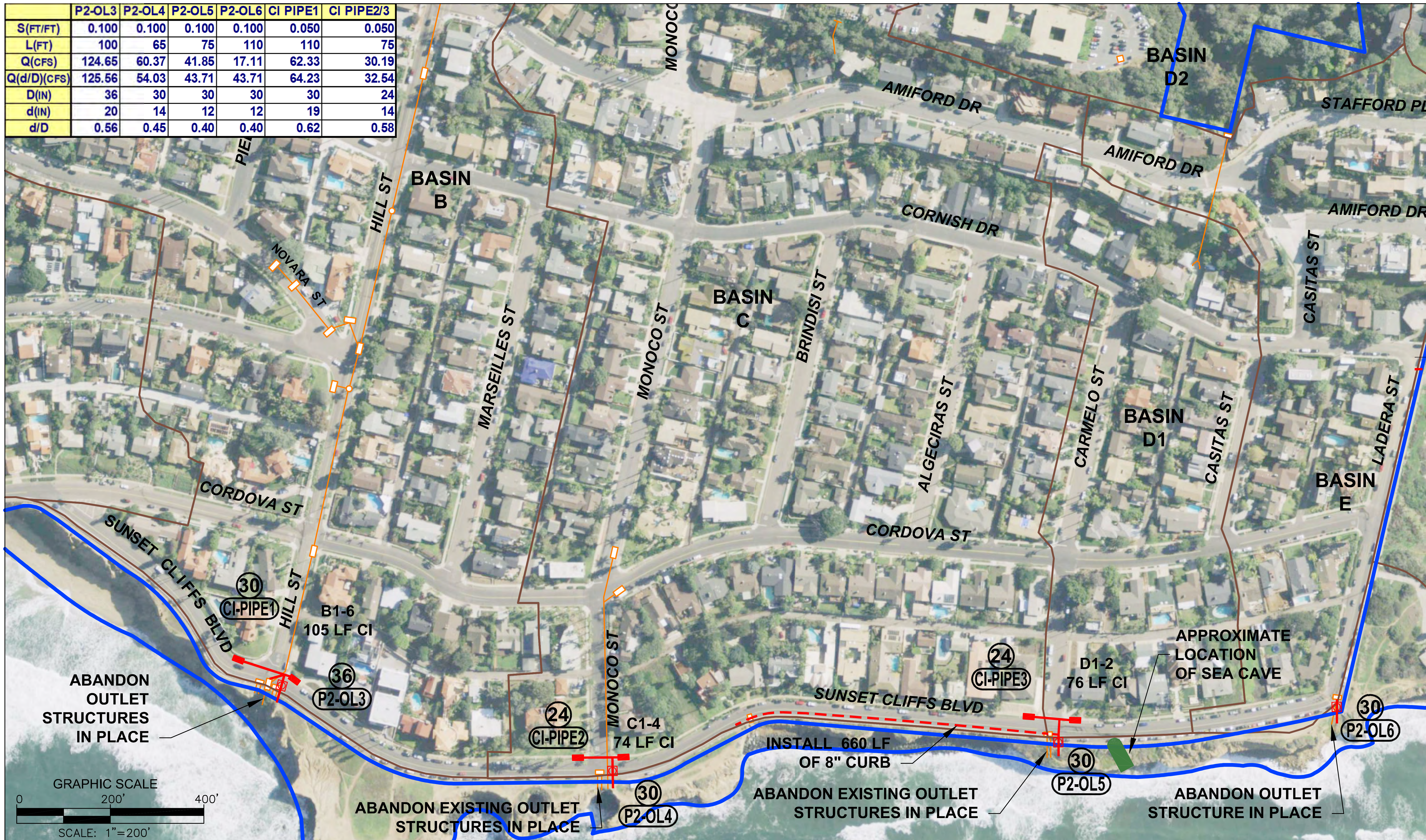
	P2-OL3	P2-OL4	P2-OL5	P2-OL6	CI PIPE1	CI PIPE2/3
S(FT/FT)	0.100	0.100	0.100	0.100	0.050	0.050
L(FT)	100	65	75	110	110	75
Q(CFS)	124.65	60.37	41.85	17.11	62.33	30.19
Q(d/D)(CFS)	125.56	54.03	43.71	43.71	64.23	32.54
D(IN)	36	30	30	30	30	24
d(IN)	20	14	12	12	19	14
d/D	0.56	0.45	0.40	0.40	0.62	0.58



LEGEND:

BASIN BOUNDARY	PROPOSED STORM DRAIN	PROPOSED STORM DRAIN TREATMENT	PROPOSED CURB INLET		
RIDGE LINE	EXISTING STORM DRAIN	EXISTING CATCH BASIN	48	PROPOSED PIPE SIZE	
PARK BOUNDARY	DIRECTION OF FLOW	EXISTING STORM DRAIN CLEANOUT	P2-1	ALTERNATIVE NO. - PIPE NUMBER(TYP)	

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Master Drainage Exhibit
Sunset Cliffs - Linear Park
Preferred Alternative 2: BASIN B - E

FIGURE 2-5

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	P2-1	P2-2	P2-3	P2-4	P2-OL1	P2-OL2
S(FT/FT)	0.014	0.005	0.023	0.011	0.100	0.100
L(FT)	95	450	510	95	110	65
Q(CFS)	30.95	11.45	21.38	20.61	42.40	41.99
Q(d/D)(CFS)	32.25	12.08	21.84	21.38	43.71	43.71
D(IN)	30	30	24	30	30	30
d(IN)	18.00	13.50	14.00	15.00	12.00	12.00
d/D	0.60	0.45	0.58	0.50	0.40	0.40

LEGEND:

- BASIN BOUNDARY
- RIDGE LINE
- PARK BOUNDARY
- PROPOSED STORM DRAIN
- EXISTING STORM DRAIN
- DIRECTION OF FLOW
- PROPOSED STORM DRAIN TREATMENT
- EXISTING CATCH BASIN
- EXISTING STORM DRAIN CLEANOUT
- PROPOSED CURB INLET
- 48 PROPOSED PIPE SIZE
- P2-1 ALTERNATIVE NO. - PIPE NUMBER(TYP)

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Master Drainage Exhibit
Sunset Cliffs - Linear Park
Preferred Alternative 2: BASIN X & A

FIGURE 2-6

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3 CONSTRUCTION ISSUES

Construction of the Sunset Cliffs Natural Park drainage improvements will consist of three primary construction activities.

1. Conventional cut and cover pipeline construction methods
2. Trenchless directional boring pipeline construction methods
3. Reinforced decorative concrete energy dissipating outfall structures

Ancillary construction activities include:

1. Construction of pre-cast or cast-in-place concrete drainage inlets and pipeline junction structures commonly referred to as cleanouts.
2. Asphaltic Concrete (AC) and concrete pavement restoration as needed after new drainage pipelines have been installed in hard paved areas.
3. Construction of concrete or AC dike or curb to control sheet flow of surface water.

Specific construction methods and issues associated with each drainage improvement are as follows:

Hillside Park Alternative 1 – Construction activities for Hillside Park Alternative 1 generally consists of construction of the following components:

- Asphaltic Concrete (AC) dike or curb to control the surface flow of storm water keeping the flow on hard paved surfaces to prevent erosion resulting from concentrated sheet flow leaving a hard paved surface and eroding an un-paved surface. Construction of AC dikes for this alternative can be completed with conventional equipment by a local contractor. There are no special access requirements or restrictions associated with the proposed locations of the AC dikes.
- 18-inch to 36-inch diameter drainage pipelines to convey storm water from hard paved areas and tributary drainage features to the energy dissipating outfall structure. For this alternative the most practical and economical construction method will likely be conventional cut and cover. Installation of the 36-inch drainage pipeline in the Culvert Canyon alignment will require heavy grading and excavation to lay back the Culvert Canyon slopes to provide a safe construction corridor at the bottom of the canyon and pipe trench. The site can be accessed from the west side of Western Loop Road.
- One energy dissipating outlet structure constructed of concrete with a specially textured and colored finish to blend into the surrounding sandstone and rock. Construction of the outlet structures will require a General Engineering contractor with a specialized decorative concrete subcontractor. Access will be challenging due to the location of the structures and the base of the fragile bluffs. Use of a barge brought in from the ocean is a potential approach, but there are some issues. Wave action may upset or move the barge and a heavy barge may damage the reef and sensitive marine

environment. A long reach crane can deliver materials and workers from the top off the bluff. The crane will have to be located far enough back from the bluff edge to prevent bluff failure.

Hillside Park Alternative 2 – Construction activities for Hillside Park Alternative 2 generally consists of construction of the following components:

- Asphaltic Concrete (AC) dike or curb to control the surface flow of stormwater keeping the flow on hard paved surfaces to prevent erosion resulting from concentrated sheet flow leaving a hard paved surface and eroding an un-paved surface. Construction of AC dikes for this alternative can be completed with conventional equipment by a local contractor. There are no special access requirements or restrictions associated with the proposed locations of the AC dikes.
- 18-inch to 36-inch diameter drainage pipelines to convey storm water from hard paved areas and tributary drainage features to the energy dissipating outfall structures. For this alternative the most practical and economical construction method will likely be conventional cut and cover construction methods combined with Horizontal Directional Drilling (HDD) or directed jack micro-tunneling. A drilling or tunneling rig can be set up in the lower parking lot and in the old baseball field. Due to the undocumented fill under the ball field consisting of concrete rubble and construction debris, the drilling site will have to be excavated beyond the undocumented fill to avoid damage and refusal of the drilling/tunneling equipment. The site can be accessed from the PLNU parking lot west of Lomaland Drive.
- Two energy dissipating outlet structures constructed of concrete with a specially textured and colored finish to blend into the surrounding sandstone and rock. Construction of the outlet structures will require a General Engineering contractor with a specialized decorative concrete subcontractor. Access will be challenging due to the location of the structures and the base of the fragile bluffs. Use of a barge brought in from the ocean is a potential approach, but there are some issues. Wave action may upset or move the barge and a heavy barge may damage the reef and sensitive marine environment. A long reach crane can deliver materials and workers from the top off the bluff. The crane will have to be located far enough back from the bluff edge to prevent bluff failure.

Linear Park Alternative 1 and 2 - Construction activities for Linear Park Alternative 1 and Alternative 2 are similar and generally consist of construction of the following components:

- Asphaltic Concrete (AC) dike or concrete curb to control the surface flow of storm water keeping the flow on hard paved surfaces to prevent erosion resulting from concentrated sheet flow leaving a hard paved surface and eroding an un-paved bluff surface. Construction of dikes or curbs for this alternative can be completed with conventional equipment by a local contractor. There are no special access requirements or restrictions associated with the proposed locations of the AC dikes.

- 18-inch to 54-inch diameter drainage pipelines to convey storm water from hard paved areas and tributary drainage features to the energy dissipating outfall structure. For this alternative the most practical and economical construction method will likely be conventional cut and cover. Special attention will be required for construction of drainage pipelines in Sunset Cliffs Boulevard over the sea caves near the foot of Carmelo Street and Froude Street. For the Linear Park Alternative 1, a detailed survey should be completed to determine the extents and distance from the cave ceiling to the road surface. With this information engineers can determine the feasibility of trenching and installing drainage pipeline in the road over the sea caves. At a minimum work over the sea caves should be conducted with light construction equipment and light weight pipe materials such as ABS truss pipe. The large diameter pipelines are outfall pipelines and should be installed with directional drilling or boring construction methods.
- Energy dissipating outlet structures constructed of concrete with a specially textured and colored finish to blend into the surrounding sandstone and rock. Construction of the outlet structures will require a General Engineering contractor with a specialized decorative concrete subcontractor. Access will be challenging due to the location of the structures and the base of the fragile cliffs. Use of a barge brought in from the ocean is not a potential approach due to the sheer cliff lack of beach for the barge to land on. A long reach crane can deliver materials and workers from the top off the cliff. The crane will have to be located far enough back from the cliff edge to prevent bluff failure. Figure 3-1 on the following page shows a potential outfall construction approach utilizing trenchless construction methods to minimize impacts to the coastal bluffs.

3.1 Resource Protection/Community Concerns

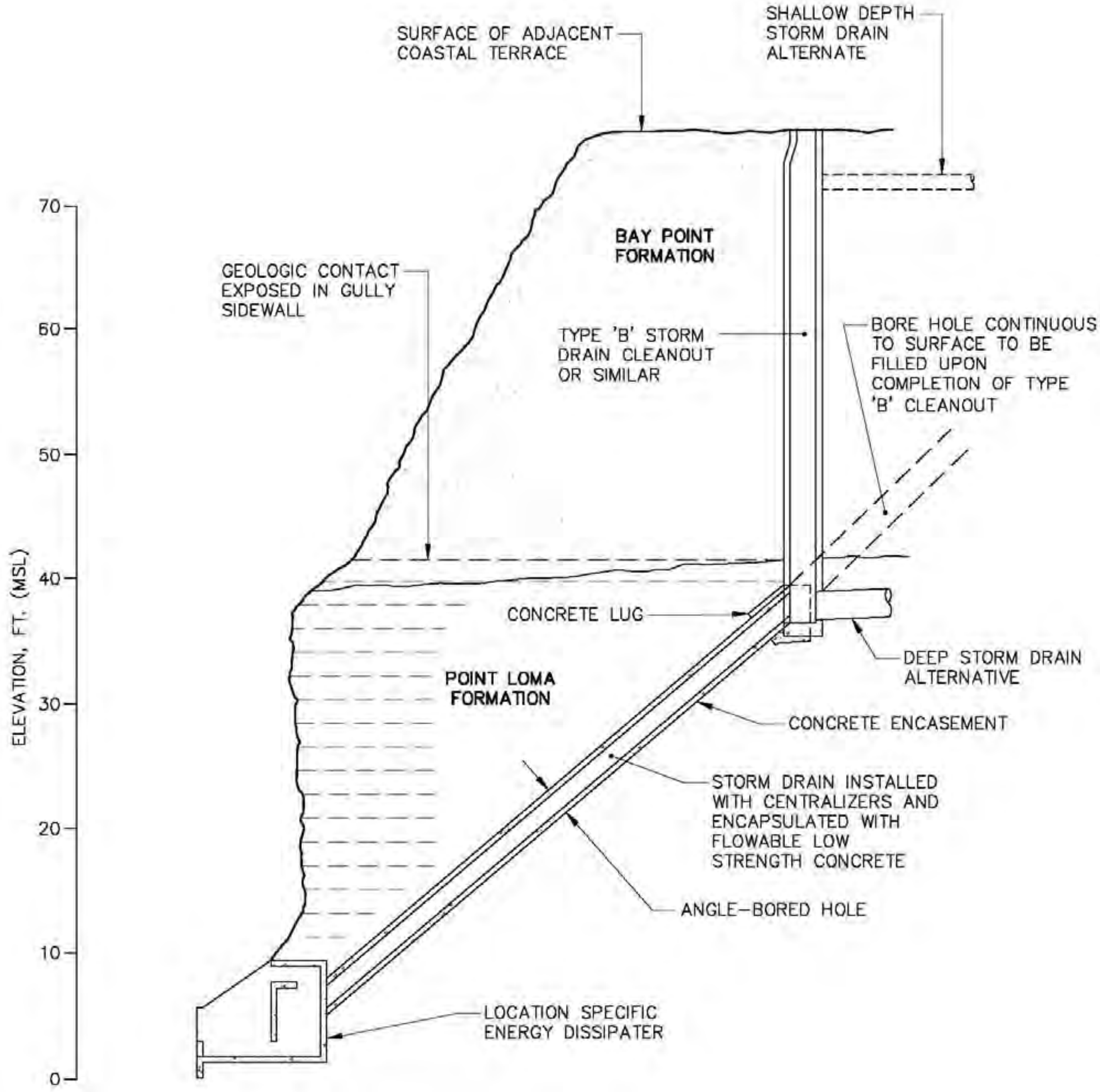
Final drainage pipeline alignments and allowable construction methods should take into consideration protection of the Sunset Cliffs Natural Park unique resources. Since the drainage improvements are linear in nature impacts to park resources can be minimized without compromising the effectiveness of the drainage improvements. Trenchless construction methods are recommended where practical to minimize excavation and ground surface disturbance.

The final result of the drainage improvements will be resource protection in the form of erosion protection from concentrated run off from paved urban areas up slope from the park. Regulatory permit requirements and resource protection measures outlined in the Master Environmental Impact Report when followed will protect the parks resources. A good example of a well preserved and protected coastal slope are Coastal Sage Scrub, Southern Coastal Bluff Scrub and Southern Maritime Chaparral found to the south of the Hillside Park in the Point Loma Ecological Reserve (Reserve) which is managed by the U.S. Navy. A critical component of restoring the SCNP to its natural state is the elimination of excess storm water runoff and pedestrian erosion.

During public meetings conducted to present the proposed drainage solutions to concerned residents and community group, a number of varying opinions were shared. The most consistent message was a desire for “soft” and “green” solutions. Some of the public proposed solutions included constructed wetlands, bio-swales, percolation basins or underground tanks. The drainage solutions proposed in this report are design to convey un-naturally large quantities of storm water runoff from upland impervious developed areas under the natural park to the Pacific Ocean through pipelines. Due to the un-naturally large quantities of storm water generated from the upland impervious areas and the steep slope of the natural parks, it is impractical to construct wetlands, bio-swales, percolation basins or underground tanks while still maintaining the essential natural characteristics of the park terrain.

Another community concern was the known presence of archeological and paleontological features located in Culvert Canyon that would be disturbed or damaged if a new pipeline is constructed in Culvert Canyon as shown in Hillside Preferred Alternative 1. For this and other reasons the Culvert Canyon alignment was not selected. The selected alternative for the Hillside Park drain features a pipeline alignment through the abandoned ball field on the southern edge of Hillside Park. This alignment is consistent with the alignment shown in the SCNP Master Plan.

Figure 3-1 Typical Storm Drain Outlet Detail



3.2 Regulatory Requirements

Regulatory requirements will be dictated by the permitting documents required. The permitting required will depend on the focused biological surveys, focused wetland delineation and prescribed construction methods as derived during final design. The following list represents regulatory agencies that would potentially issue a permit or dictate regulatory requirements for the final project configuration.

- A Construction General Permit with State Water Resources Control Board will require a site specific Storm Water Pollution Prevention Plan (SWPPP)
- A Coastal Development Permit with final approval by the California Coastal Commission.
- An individual 404 Permit or Nationwide Permit from the Army Corps Of Engineers in compliance with Section 404 of the federal Clean Water Act.
- A Section 401 Water Quality Certification from Regional Water Quality Control Board in compliance with Section 401 of the federal Clean Water Act and/or a Waste Discharge Requirement (WDR) in compliance with the Porter-Cologne Act
- A Section 1602 Streambed Alteration Agreement from California Department of Fish Game
- Incidental take permit from the United States Fish and Wildlife Service under the federal Endangered Species Act
- Incidental take permit from the California Department of Fish and Game under the California Endangered Species Act

Other regulatory requirements would include the Mitigation, Monitoring and Reporting Plan requirements as written in the project specific CEQA document.

3.3 Public Safety

The hazards of pipeline and outfall construction in and around the Sunset Cliff bluffs will be significant. Robust barriers and generous buffer zones around the construction areas will need to be maintained at all times. A project of this magnitude in a popular public destination with an active residential community, should have a full time public liaison trained to interact with the public. A project brochure handout is a useful tool to satiate the curiosity of most interested citizens. A night time and weekend watchman should be considered when heavy equipment and open deep excavations have the potential of creating an attractive nuisance to the public.

Hillside Park Alternative 2 locates significant drainage pipeline improvements under the athletic field constructed with undocumented fill material that should be tested for hazardous materials prior to excavation.

After construction, the open pipeline ends at the outlet structure should be protected from unauthorized entrance by utilizing a corrosion resistant gate over the end of an open pipeline.

The spacing of the gate bars should prevent passage of anything larger than a 4-inch diameter sphere. Another method on protecting the open pipeline ends from unauthorized intrusion is the installation of rubber duckbill check valves. These check valves are made in a variety of configurations to meet various installation configurations and are not subject to corrosion.

3.4 Estimated Costs

Unit prices for this planning phase estimate are based on recent competitively bid contractor unit prices for similar construction activities as well as the most recent City of San Diego Development Services Department Unit Price List dated January 2009. A detailed breakdown of the engineer's estimate of probable construction costs for each alternative is provided on the following pages; the total construction cost for each alternative is listed below. The cost listed below are construction costs only. Additional costs such as design, project management, permitting, etc. are listed separately on the following detail pages.

- Hillside Park Alternative No. 1 - $1,996,162
- Hillside Park Alternative No. 2 - $1,887,637
- Linear Park Alternative No. 1 - $2,054,117
- Linear Park Alternative No. 2 – $1,910,800

Detailed Cost Opinions can be found on the following pages.

Table 3-1 Hillside Park Alternative No. 1 Cost Opinion

ITEM NO.	DESCRIPTION	UNIT	QUANTITY	UNIT COST	ITEM TOTAL
Hillside Park Alternative 1					
1	6" AC Dike (Type A)	LF	3150	$ 12	$ 37,800
2	Catch Basin (Type G)	EA	6	$ 7,900	$ 47,400
3	18" Storm Drain (Water Tight Joints)	LF	2280	$ 130	$ 296,400
4	24" Storm Drain (Water Tight Joints)	LF	330	$ 150	$ 49,500
5	36" Storm Drain (Water Tight Joints)	LF	600	$ 500	$ 300,000
6	36" Storm Drain (Directional Bore)	LF	150	$ 1,500	$ 225,000
7	Cleanout (Type B)	EA	8	$ 6,968	$ 55,744
8	Outfall with Energy Dissipater	EA	1	$ 60,000	$ 60,000
9	Permanent Water Quality BMP	EA	3	$ 7,000	$ 21,000
10	Remove Existing Storm Drain	LF	90	$ 60	$ 5,400
11	Pavement Restoration	SF	6000	$ 7	$ 42,000
12	Clear & Grub	SF	120000	$ 2	$ 180,000
13	Grading	CY	9500	$ 36	$ 342,000
14	Mobilization, BMPs, Bonds & Cleanup - 10%	LS	1	$ 166,224	$ 166,224
CONSTRUCTION Sub-total Hillside Park Alternative 1:					$ 1,828,468
Contingency - 20 %					$ 365,694
CONSTRUCTION TOTAL Hillside Park Alternative 1:					$ 2,194,162
Mitigation:		AC	3	$ 200,000	$ 600,000
Soft Costs (Design, Permitting, CM, Admin)					$ 1,117,665
PROGRAM COST Hillside Park Alternative 1:					$ 3,911,827

Table 3-2 Hillside Park Alternative No. 2 Cost Opinion

ITEM NO.	DESCRIPTION	UNIT	QUANTITY	UNIT COST	ITEM TOTAL
Hillside Park Alternative 2					
1	6" AC Dike (Type A)	LF	3545	$ 12	$ 42,540
2	Catch Basin (Type G)	EA	5	$ 7,900	$ 39,500
3	18" Storm Drain (Water Tight Joints)	LF	1310	$ 130	$ 170,300
4	36" Storm Drain (Directional Bore)	LF	630	$ 1,300	$ 819,000
5	Cleanout (Type B)	EA	2	$ 6,968	$ 13,936
6	Outfall with Energy Dissipater	EA	2	$ 50,000	$ 100,000
7	Permanent Water Quality BMP	EA	3	$ 7,000	$ 21,000
8	Remove Existing Storm Drain	LF	90	$ 60	$ 5,400
8	Pavement Restoration	SF	2500	$ 7	$ 17,500
9	Concrete Drainage Ditch (Type D)	LF	730	$ 26	$ 18,980
10	Clear & Grub	SF	72000	$ 2	$ 108,000
11	Grading	CY	2052	$ 36	$ 73,872
12	Mobilization, BMPs, Bonds & Cleanup - 10%	LS	1	$ 143,003	$ 143,003
	CONSTRUCTION Sub-total Hillside Park Alternative 2:				$ 1,573,031
	Contingency - 20 %				$ 314,606
	CONSTRUCTION TOTAL Hillside Park Alternative 2:				$ 1,887,637
	Mitigation:	AC	2	$ 200,000	$ 400,000
	Soft Costs (Design, Permitting, CM, Admin)				$ 915,055
	PROGRAM COST Hillside Park Alternative 2:				$ 3,202,692

Table 3-3 Linear Park Alternative No. 1 Cost Opinion

ITEM NO.	DESCRIPTION	UNIT	QUANTITY	UNIT COST	ITEM TOTAL
Alternative 1: Linear Park Basin X & A					
1	Curb Inlet (Type C)	EA	9	$ 7,900	$ 71,100
2	8" Curb & Gutter (Type H)	LF	230	$ 33	$ 7,590
3	18" Storm Drain (Water Tight Joints)	LF	300	$ 130	$ 39,000
4	24" Storm Drain (Water Tight Joints)	LF	500	$ 150	$ 75,000
5	30" Storm Drain (Water Tight Joints)	LF	190	$ 164	$ 31,160
6	48" Storm Drain (Water Tight Joints)	LF	1080	$ 239	$ 258,120
7	48" Storm Drain (Directional Bore)	LF	110	$ 1,200	$ 132,000
8	Cleanout (Type A)	EA	3	$ 6,968	$ 20,904
9	Outfall with Energy Dissipater	EA	1	$ 50,000	$ 50,000
10	Permanent Water Quality BMP	EA	1	$ 7,000	$ 7,000
11	Pavement Restoration	SF	13220	$ 7	$ 92,540
12	Mobilization, BMPs, Bonds & Cleanup - 10%	LS	1	$ 78,441	$ 78,441
	CONSTRUCTION Sub-total Linear Park Basin X & A:				$ 862,855
	Contingency - 20 %				$ 172,571
	CONSTRUCTION TOTAL Linear Park Basin X & A:				$ 1,035,426
Alternative 1: Linear Park Basin B & E					
1	Curb Inlet (Type C)	EA	6	$ 7,900	$ 47,400
2	8" Curb & Gutter (Type H)	LF	660	$ 33	$ 21,780
3	18" Storm Drain (Water Tight Joints)	LF	0	$ 130	$ ---
4	24" Storm Drain (Water Tight Joints)	LF	150	$ 150	$ 22,500
5	30" Storm Drain (Water Tight Joints)	LF	670	$ 164	$ 109,880
6	48" Storm Drain (Water Tight Joints)	LF	780	$ 239	$ 186,420
7	30" Storm Drain (Directional Bore)	LF	75	$ 1,200	$ 90,000
8	54" Storm Drain (Directional Bore)	LF	55	$ 1,400	$ 77,000
9	Cleanout (Type A)	EA	3	$ 6,968	$ 20,904
10	Outfall with Energy Dissipater	EA	2	$ 50,000	$ 100,000
11	Permanent Water Quality BMP	EA	2	$ 7,000	$ 14,000
12	Pavement Restoration	SF	11693	$ 7	$ 81,851
13	Mobilization, BMPs, Bonds & Cleanup - 10%	LS	1	$ 77,174	$ 77,174
	CONSTRUCTION Sub-total Linear Park Basin B & E:				$ 848,909
	Contingency - 20 %				$ 169,782
	CONSTRUCTION TOTAL Linear Park Basin B & E:				$ 1,018,690
	CONSTRUCTION TOTAL Linear Park Alternative 1:				$ 2,054,117
	Soft Costs (Design, Permitting, CM, Admin)				$ 821,647
	PROGRAM COST Linear Park Alternative 1:				$ 2,875,763

Table 3-4 Linear Park Alternative No. 2 Cost Opinion

ITEM NO.	DESCRIPTION	UNIT	QUANTITY	UNIT COST	ITEM TOTAL
Alternative 2: Linear Park Basin X & A					
1	Curb Inlet (Type C)	EA	9	$ 7,900	$ 71,100
2	8" Curb & Gutter (Type H)	LF	230	$ 33	$ 7,590
3	18" Storm Drain (Water Tight Joints)	LF	250	$ 130	$ 32,500
4	24" Storm Drain (Water Tight Joints)	LF	510	$ 150	$ 76,500
5	30" Storm Drain (Water Tight Joints)	LF	640	$ 164	$ 104,960
6	30" Storm Drain (Directional Bore)	LF	175	$ 1,300	$ 227,500
7	Cleanout (Type A)	EA	2	$ 6,968	$ 13,936
8	Outfall with Energy Dissipater	EA	2	$ 40,000	$ 80,000
9	Permanent Water Quality BMP	EA	2	$ 7,000	$ 14,000
10	Pavement Restoration	SF	9500	$ 7	$ 66,500
11	Mobilization, BMPs, Bonds & Cleanup - 10%	LS	1	$ 69,459	$ 69,459
CONSTRUCTION Sub-total Linear Park Basin X & A:					$ 764,045
Contingency - 20 %					$ 152,809
CONSTRUCTION TOTAL Linear Park Basin X & A:					$ 916,854
Alternative 2: Linear Park Basin B & E					
1	Curb Inlet (Type C)	EA	6	$ 7,900	$ 47,400
2	8" Curb & Gutter (Type H)	LF	660	$ 33	$ 21,780
3	18" Storm Drain (Water Tight Joints)	LF	0	$ 130	$ ---
4	24" Storm Drain (Water Tight Joints)	LF	150	$ 150	$ 22,500
5	30" Storm Drain (Water Tight Joints)	LF	110	$ 164	$ 18,040
6	30" Storm Drain (Directional Bore)	LF	250	$ 1,200	$ 300,000
7	36" Storm Drain (Directional Bore)	LF	100	$ 1,300	$ 130,000
8	Outfall with Energy Dissipater	EA	4	$ 40,000	$ 160,000
9	Permanent Water Quality BMP	EA	4	$ 7,000	$ 28,000
10	Pavement Restoration	SF	3610	$ 7	$ 25,270
11	Mobilization, BMPs, Bonds & Cleanup - 10%	LS	1	$ 75,299	$ 75,299
CONSTRUCTION Sub-total Linear Park Basin B & E:					$ 828,289
Contingency - 20 %					$ 165,658
CONSTRUCTION TOTAL Linear Park Basin B & E:					$ 993,947
CONSTRUCTION TOTAL Linear Park Alternative 2:					$ 1,910,800
Soft Costs (Design, Permitting, CM, Admin)					$ 764,320
PROGRAM COST Linear Park Alternative 2:					$ 2,675,120

3.5 Estimated Schedule

3.5.1 Phased Implementation Plan

Implementation of drainage improvements can be divided into two areas, the Hillside Park section and the Linear Park Section. These two park sections do not share drainage improvements and can be constructed independently or concurrently depending on funding available.

Hillside Park Alternative 1 consists of a drainage pipeline network feeding into one single outfall and must be constructed in a single phase to achieve functionality.

Hillside Park Alternative 2 consists of a small drainage pipeline conveying storm water from the lower parking lot at the north end of the park and a second large drainage pipeline network collecting storm water from Lomaland Drive/Western Loop Road and the university parking lot and discharging through an outfall at the south end of the park. In addition to these primary pipelines Alternative 2 includes a curb and brow ditch on Lomaland Drive/Western Loop Road and improvements to and a drain line from the Young Hall parking area. These drainage systems could be constructed independently as funding allows although the permitting process to construct these facilities will be extensive and it would be crucial for the project success to construct the drainage facilities within the time period allowed by the permits. For a project of this magnitude a two to three year permit window should be possible to negotiate.

Linear Park Alternative 1 consists of three separate drainage network elements feeding three outfalls. Each of these three elements could be constructed independently as funding allows although the permitting process to construct these facilities will be extensive and it would be crucial for the project success to construct the drainage facilities within the time period allowed by the permits. For a project of this magnitude a two to three year permit window should be possible to negotiate.

Linear Park Alternative 2 consists of six separate small drainage network elements feeding six outfalls. Each of these six elements could be constructed independently as funding allows although the permitting consideration are identical to the Linear Park Alternative 1 projects and the six drainage networks are so small that it would be impractical to bid, award and construct each one independently.

In summary, it is recommended that the Sunset Cliffs drainage improvements are constructed as one single phase including Hillside Park and Linear Park improvements or two separate phases with Hillside Park and Linear Park improvements each being a separate phase. Priority of the drainage improvements depends on which resources are to be protected. In general, the Linear Park drainage improvements will protect public infrastructure resources such as the public walkway along the bluff, Sunset Cliffs Blvd. and utilities in Sunset Cliffs Blvd. The Hillside Park drainage improvements will protect natural resources of coastal sage scrub and coastal bluffs.

4 MONITORING PROGRAM

During construction monitoring plans will be dictated by the final CEQA document, the SWPPP, resource agency permit conditions and the Coastal Development Permit conditions. At a minimum, monitoring activities will include the following:

- Delineation and monitoring of construction site limits
- Monitoring of the construction site best management practices as dictated by the SWPPP
- Delineation and monitoring of biological resources that are not to be disturbed. A monitor may need to be present for construction activities during the avian nesting periods depending on the results of the focused surveys. Depending on the species, the avian nesting period range is generally from February to September.
- Delineation and monitoring of historical resources that are not to be disturbed. A monitor may need to be present for construction activities near historical resources. Hillside Park Alternative 1 will likely require monitoring in the upper reaches of Culvert Canyon.
- Delineation of any known Paleontological sites and monitoring for indications of Paleontological resources that may be discovered during excavation in the Point Loma and Bay Point Formations
- In addition to resource monitoring, a project of this magnitude in a popular public destination with an active residential community, should have a full time public liaison trained to interact with the public. A project brochure handout is a useful tool to satiate the curiosity of most interested citizens.

Sunset Cliffs Natural Park benefits from an active community volunteer group with a strong interest in restoring and preserving the natural features of the Sunset Cliffs Natural Park. Drainage systems generally do not require much maintenance however looking for signs of potential drainage feature failure during dry weather or after rains and identifying areas for repair or maintenance can prevent erosion damage from drainage system malfunctions during rain events. Post Construction monitoring carried out by community volunteers could consist of the following elements:

- Long term monitoring of the ground water monitoring wells to document a rise or fall in the perched water table areas
- Look for signs of curb over topping
- Look for pavement cracking or settling around drainage inlets and pipeline alignments
- Look for ground settlement of sink holes around drainage structures and pipeline alignments in unpaved areas

- Prior to a predicted rain event and immediately after a rain event, look for blocked storm drain inlets, trash or plant debris in gutters or brow ditches

Minor maintenance tasks such as trash removal, weed control and minor sediment removal can be carried out by a community volunteer group. Any monitoring observations revealing indications of developing sink holes or structural failures should be documented with photographs and sent to a designated City of San Diego Park and Recreation Department representative. Commonly found items such as coins or ball point pens can be used to photo document the size of cracks and fractures.

APPENDIX A

Geotechnical Report

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GEOTECHNICAL
PROFESSIONALS INC.

August 4, 2006
(Revised October 9, 2006)

Dudek and Associates, Inc.
605 Third Street
Encinitas, California 92024

Attention: Mr. Steve Jepsen

Subject: Revised Geotechnical Data Report
Sunset Cliffs Natural Park
San Diego, California
GPI Project No. 2081.I

Dear Mr. Jepsen:

This letter report presents the results of a geotechnical data collection investigation performed by Geotechnical Professionals Inc. (GPI) to supplement the Drainage Study being performed by Dudek & Associates for the Sunset Cliffs Natural Park in San Diego, California. Specifically, this letter report presents the geotechnical and geologic data collected during our field and laboratory investigation, which included drilling and logging borings, groundwater monitoring well installation, geologic observation, groundwater measurements, moisture/density testing, and permeability.

The Sunset Cliffs Natural Park Recreation Council provided comments on GPI's geotechnical data report dated August 4, 2006. The comments have been incorporated into this revised report or discussed in a response letter included as Appendix C.

PROJECT DESCRIPTION

Sunset Cliffs Natural Park is located along the western shoreline of the Point Loma Peninsula as shown on Figure 1, Site Location Map. The park is divided in two distinct areas named Linear Park and Hillside Park, as shown in Figure 2, Site Plan

Linear Park spans a distance of approximately one mile directly along the ocean bluffs from Adair Street to Ladera Street. This narrow area of land bordered by Sunset Cliffs Boulevard to the east and the bluffs and ocean to the west covers approximately 18 acres. Linear Park contains parking lots, paths and undeveloped land above the bluffs, steep ocean bluffs/cliffs, sea caves, sea walls, rip rap placed at the base of cliffs in a few areas, small intermittent beaches at the base of the bluffs, a sewer pump station, storm drain outlets near the top of the bluffs, and a stairway at its south end. The landward area to the east of Linear Park contains single family residential homes to the top of the ridge of Point Loma.

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Hillside Park begins at the south end of Linear Park. Hillside Park is a natural open space along a west facing slope covering approximately 50 acres. The Hillside Park is bordered to the north by Ladera Street, to the west by the ocean, to the east by Point Loma Nazarene University (PNLU), and to the south by the Point Loma Naval Military Complex. Hillside Park contains a paved parking lot along Lomaland Drive, a paved parking lot at the east end of Ladera Street, undeveloped land with many hiking trails, a baseball field, intermittent erosion gullies, drainage canyons to the ocean, a concrete drainage swale down the bluffs, ocean bluffs, steep ocean cliffs, sea caves, beaches at the base of the cliffs, and City-owned life estates (rental homes) near the bluffs near the north end of Hillside Park and above the Upper Parking Lot. Hillside Park contains additional infrastructure such as Western Loop Road, the City's pressurized sludge line, PLNU's sewage facilities, a private drive near the northeast canyon, and the culvert on Western Loop Road. Single-family residential homes are north of Hillside Park along Ladera Street and at the northeast border of the park east of the terminus of Ladera Street.

Figure 2 shows an aerial photograph and site plan of the park.

In general, the majority of drainage east of Linear Park is conveyed down slope in storm drain systems and on east/west-trending streets. At the bluff face along Sunset Cliffs Boulevard, there exists a curb/gutter system and a series of cantilevered culverts at street-ends, discharging through the face of the coastal bluff. A portion of the upland drainage at the Hillside Park, south of Ladera Street, is conveyed as sheet flow over the coastal terrace and over the coastal bluff as sheet flow. The sheet flow has created rills, gullies, and three relatively mature drainages within Hillside Park. Upland drainage at Hillside Park is also conveyed in storm drain pipes, which collect, convey and concentrate the runoff, and along Western Loop Road, which carries runoff to a culvert emptying into a deep ravine.

The first project as part of the Master Plan improvements to Sunset Cliffs Natural Park called for a comprehensive drainage study and drainage plan for the park. The drainage study will present an environmentally responsible plan to restore areas damaged by past erosion, preserve the unique geological formations within the park, minimize urban runoff onto and across the park land, conduct an engineering study of the existing drainage patterns, address erosion control by implementing native plant preservation and re-vegetation, and develop recommendations regarding drainage solution. Any new on-site drainage system will capture, collect, treat, and convey surface water to minimize surface/subsurface erosion and groundwater seepage.

A technical report addressing the geotechnical, hydrology, and coastal processes was prepared in support of the Master Plan (Reference1).

There are two main objectives of this geotechnical data collection study. First by drilling borings throughout the park, the in-situ moisture content of the soils can be determined at various depths, the geologic stratification between the terrace deposits and underlying sedimentary bedrock can be better understood, and permeability tests of relatively undisturbed samples can be performed. This information will allow the designers of the drainage improvements to better understand the amount of seepage from the ground surface into underlying layers. Second, by installing groundwater monitoring wells, the perched groundwater layers can be measured and monitored over time to determine the seasonal changes of perched groundwater levels and to measure the changes in perched groundwater after implementation of a comprehensive drainage plan.

SCOPE OF WORK

Our scope of work for this investigation consisted of review and use of existing geotechnical data, field exploration, monitoring well installation, laboratory testing, groundwater measurements, and the preparation of this letter report.

This report was prepared in general accordance with the scope of work presented in our Proposal No. SP01120 dated March 4, 2002, to Dudek & Associates, the City of San Diego's consultant for the subject project.

The field exploration for this geotechnical data collection study consisted of eight exploratory borings. The exploratory borings were drilled using truck-mounted, air percussion and air rotary equipment to depths of 20 to 40 feet below existing grades. At seven of the boring locations, groundwater monitoring wells were installed to measure groundwater perched on the less permeable layers of the sedimentary bedrock. Details of the drilling, well installation, and Logs of Borings are presented in Appendix A. The locations of the borings and monitoring wells were selected with the input of representatives of the City of San Diego and the Sunset Cliffs Recreational Council. The locations were selected where perched water was anticipated based upon seepages observed along the cliffs, phreatophytic vegetation, or proximity relative to highly irrigated athletic field. The locations of the subsurface explorations are shown on the Site Plan, Figure 2.

Laboratory soil tests were performed on selected representative samples as an aid in soil classification and to evaluate the engineering properties of the soils. The geotechnical laboratory testing program included determinations of moisture content and dry density, and coefficient of permeability. Laboratory testing procedures and results are summarized in Appendix B.

GEOLOGIC SETTING

In general, Point Loma is a peninsula extending from the San Diego River southward to the lighthouse near the entrance of San Diego Bay to the Pacific Ocean. Point Loma has a width, between the Pacific Ocean to the west and San Diego Bay to the east, ranging from approximately 2 miles in the north and approximately ½ mile at its southern end.

Point Loma has a western shoreline consisting predominantly of irregular bluffs due to differences in geologic structure and in rock hardness. The majority of the bluffs are sea cliffs, subject to marine erosion at the base, with relatively shallow caves and coves between rocky headlands, formed by wave erosion of less resistant rock masses. Where sufficient sand is available and trapped between the headlands, relatively small pocket beaches have been formed sporadically along Point Loma. In other areas, a shore platform of bedrock surface extends into the ocean.

The continental margin at San Diego is underlain by a moderately thick, nearly flat-lying succession of Upper Cretaceous, Tertiary and Quaternary sedimentary deposits and a Mesozoic plutonic and metamorphic basement complex (Reference 2).

The geology of Sunset Cliffs Natural Park, as well as data on the stability of the bluffs and the rates and causes of bluff retreat, have been summarized in several recent publications and is only briefly summarized here.

The Linear Park consists of the bluff tops west of and adjacent to Sunset Cliffs Boulevard, and sea cliffs within the bluffs that drop down to ocean. The Hillside Park consists of the bluffs and sea cliffs with a coastal terrace with natural drainages extending upslope several hundred feet to Point Loma Nazarene University. Areas of the coastal terrace within the Hillside Park have been graded during the construction of parking areas, homes, roads, a dump site, garden areas, PLNU's sewage infrastructure, City's sludge line, and the athletic field. In general, the bluffs are very steep to vertical. The sea cliffs are generally exposed to the breaking surf at high tide with intermittent sea caves at the base and with exposed shore platform or sand beaches at low tide.

The geology of Sunset Cliffs Natural Park consists of a homoclinally, northeast-dipping sequence of Cretaceous-age, marine sedimentary rock that forms the sea cliff, capped by Pleistocene age, marine and non-marine terrace deposits at the top of the bluff that represent an angular unconformity with an age gap of approximately 60-70 Million years. The terrace deposits underlie Sunset Cliffs Boulevard and the adjacent land of Hillside Park to the east. The marine sedimentary rocks are a portion of the Point Loma Formation and the terrace deposits are named the Bay Point Formation (Reference 3).

As described in the referenced publications (References 4, 5, and 6), erosion and retreat of the bluffs are caused by wave abrasion in the lower bluff and by subaerial processes in the upper bluff. Sea caves and other signs of more rapid erosion along the cliffs are due to localized weaknesses in the bedrock including faults, joints, lithologic differences (siltstone vs. sandstone) and by weakening of the bedrock by localized seepage. In addition, subterranean seepage and piping are also major sources of erosion and bluff retreat as well as the development of sea caves (Reference 1, 6, and 7). Erosion of the bluff top, within the weaker Bay Point Formation, is largely due to uncontrolled runoff, foot traffic, and at times, perched groundwater following the formation contact between the terrace deposits and underlying bedrock. The stability of the seaside bluffs have been examined by others near the life estates (Reference 4) and along Linear Park (Reference 5).

Groundwater flow is controlled largely by seepage, fracture porosity and by differences in the permeability of shale, siltstone and sandstone.

SURFACE CONDITIONS

Along Linear Park, the top of the bluffs at the ocean ranges from approximately +25 feet (mean sea level) near Adair Street on the north, upward to approximately +65 feet near Cordova Street, downward to approximately to +44 feet near Monaco Street, and back upward to approximately +65 feet at Ladera Street on the south. The bluffs are extremely steep with near vertical lower sections with the exception of a promontory area just south of Hill Street. Along Hillside Park, the top of the bluffs at the ocean ranges from approximately +65 feet at Ladera Street to approximately +80 feet at the southerly park boundary. Hillside Park contains three major westerly-trending and relatively mature drainages through the coastal terrace, that extend upslope beyond Western Loop Road on the southern border of the adjacent property of Point Loma Nazarene College. The elevation along the upper portion of Hillside Park adjacent to Point Loma Nazarene College ranges from approximately 140 to 310 feet. Within Hillside Park, the bluffs range from extremely steep near Ladera Street to the north, and extremely steep on the south end near the Navy property. In the middle of Hillside Park just south of the life estates, the bluffs are not as steep with a relatively large sand beach area (Garbage Beach) at the base of the bluffs.

Along Linear Park, our borings/monitoring wells were installed in pavement areas consisting of a parking lot (south of Adair Street), and City street (Froude Street and Sunset Cliffs Boulevard). The borings/monitoring wells were installed in pavement areas ranging from approximately 40 to 80 feet from the bluff face. In Hillside Park, the borings/monitoring wells near the bluff face were installed in park area with sparse vegetation on the terrace deposits. Figure 3, Boring Location Plan, shows the locations of the borings at the bluff faces.

The boring/monitoring wells installed along the eastern side of Hillside Park were installed in Western Loop Road. Western Loop Road is paved with asphalt concrete over base material. Western Loop Road slopes downward from the City parking lot at the northeast corner of Hillside Park to the entrance to the college dormitories at the southeast corner of Hillside Park. In the slope above the borings/monitoring wells on Western Loop Road, there is an athletic field in PLNU property. The athletic field is approximately 60 to 100 feet higher than Western Loop Road. Near one boring/monitoring well on Western Loop Road, there exists dense vegetation of phreatophytic plants indicating an upslope water source. In addition, seeps of water were observed in the rather distressed asphalt concrete pavement surface along Western Loop Road indicating water flowing within the base course.

GEOLOGIC UNITS

Two main geologic formations exist within Sunset Cliffs Natural Park as wells as minor amounts of fill, alluvial/colluvial deposits within the drainages, residual soils, and beach sands.

<u>Bay Point Formation</u>: The Bay Point Formation unconformably caps the underlying bedrock and consists of marine sands with shells and rounded gravel and cobbles, grading

upward into medium dense to dense, silty sands, silts, clayey sands, and hard, sandy clays that are probably non-marine in origin. The Bay Point Formation is relatively erodible and porous resulting in surface and subaerial erosion with water seeping through the formation to underlying bedrock. The Bay Point Formation forms relatively moderate to steep slopes at the bluff tops.

Point Loma Formation: The Point Loma Formation crops out along the entire coastal bluffs and consist of interbedded fine grained, moderately well cemented, thin to thick bedded sandstone and dark gray shaly to massive siltstone. The most resistant forming sea cliffs are composed largely of sandstone, while the less steep cliffs are composed of less erosion resistant siltstone. The Point Loma Formation forms vertical slopes with faults, fractures, and joints locally within the slope face.

The geologic formations at the bluff faces are shown in photographs shown in Figure 4. Generalized geologic cross-sections at the bluff face at the wells installed along Linear Park and at the north end of Hillside Park are shown on Figure 5. The alignment of the cross-sections at the bluff faces are shown on the boring locations plans (Figure 3).

GROUNDWATER CONDITIONS

Recent past studies (References 1 and 4) indicated groundwater seepage may contribute to the instability of geologic units within the face of the bluffs in the sedimentary rock and terrace deposits by weakening their soil strength as well as supporting vegetation in the bluff face, which weakens the bluff face. Based upon observations of water seepage in the face of the bluffs, well above the ocean level, it has been conjectured that significant perched groundwater conditions exist throughout Sunset Cliffs Natural Park.

Perched groundwater results from water infiltrating downward to a confining layer of low permeability. The water reaches a depth where water fills all of the openings in soil and cracks in rock while not infiltrating the relatively impermeable zone. Perched groundwater within bedrock is often influenced by fractures and faults, which are more permeable paths for groundwater movement, rather than the permeability of the intact bedrock. The perched groundwater level rises and saturates the overlying soil or bedrock. The depth to the perched water table depends on the nature of the geological materials and the slope of the land surface. The perched groundwater zone remains as long as the overlying infiltration flow is greater than the infiltration flow downward in the less permeable zone or outward towards areas of greater permeability (bluff face at Sunset Cliffs Park).

A perched groundwater condition in the park may result from surface water (from sheet flow, rainfall, or irrigation) seeping through the relatively permeable terrace deposits of the Bay Point Formation downward to the less permeable sedimentary bedrock of the Point Loma Formation. Perched groundwater conditions in the park likely results from contribution of water infiltration occurring within the park and from off-site sources outside the park flowing downstream along bedrock contacts towards the ocean. Urbanization of the neighborhoods east of Linear Park and the college east of Hillside Park likely has increased the amount of irrigation and surface water potentially contributing to any perched water zones. Perched groundwater levels may also be affected by rainy seasons in

Southern California, where most rainfall occurs between December and March followed by a dry season with very little rainfall, as well as extended seasons of above or below normal precipitation.

We installed seven groundwater monitoring wells in the eight borings drilled throughout Sunset Cliffs Natural Park. The locations were selected where perched water may have been anticipated based upon seepages along the bluffs, phreatophytic vegetation, or proximity relative to highly irrigated athletic fields.

A groundwater monitoring well (LP-1) was installed in a 31-foot boring in the parking lot in Linear Park just south of Adair Street. The monitoring well was installed approximately 40 feet from the bluff face. The monitoring well was installed at this location because of the seeps observed at the bluff face some 10 to 15 feet above the bottom of the bluff and the past bluff failures just north of this site, which resulted in a large mechanically stabilized earth retaining wall being built for bluff stabilization. Directly after drilling and well installation on May 16, 2006, groundwater was measured at a depth of 27 feet from the ground surface at Elevation +8 feet. The groundwater within the well at that time was confirmed by tasting to be fresh water. On June 11, 2006, the groundwater level was measured 21 feet from the ground surface at Elev. +14 feet. On July 8, 2006, the groundwater depth increased slightly by 0.7 feet. The groundwater appears to be perched on a gray siltstone bed encountered at approximately Elev. +10 feet.

A groundwater monitoring well (LP-2) was installed in a 26-foot boring in Froude Street directly across Sunset Cliffs Boulevard from Linear Park. The monitoring well was installed approximately 70 feet from the bluff face. The monitoring well was installed at this location because of the seeps observed at the cliff face and the apparent bluff retreat to the western edge of Sunset Cliffs Boulevard, which resulted in significant rip rap being placed at the bluff base. In addition, during the installation of the sewer in the eastern half of Sunset Cliffs Boulevard in the 1990's, a representative of the Sunset Cliffs Recreational Council observed seepage in the sewer trench excavation. Directly after drilling and well installation on May 16, 2006, groundwater was measured 18 feet from the ground surface at Elevation +25 feet. On June 11, 2006, the groundwater level was measured 14 feet from the ground surface at Elev. +29 feet. On July 8, 2006, the groundwater depth decreased slightly by 0.1 feet. The groundwater appears to be perched on a gray cemented sandstone bed encountered at approximately Elev. +20½ feet.

A groundwater monitoring well (LP-3) was installed in a 31-foot boring in a paint delineated traffic island at the intersection of Cordova Street and Sunset Cliffs Boulevard. The monitoring well was installed at a distance of approximately 80 feet from the bluff face. The monitoring well was installed at this location because of the seeps observed at the bluff face above the relatively large beach in this area. The geologic observation at this boring site indicated much thicker terrace deposits than anticipated with a thickness of 25 feet. No groundwater was observed to be resting on the contact between the terrace deposits and sandstone directly after drilling and well installation on May 17, 2006. In groundwater readings taken on June 11, 2006 and July 8, 2006, no groundwater was observed in the groundwater monitoring well. If any infiltration of water does occur through the terrace deposits, it does not become perched on the sandstone at a depth of 25 feet or the darker gray, siltstone and claystone at a depth of 30 feet. The location of this monitoring well is relatively high and the bedrock contacts may slope to the north. Any perched groundwater may be at deeper depths or along geologic contacts likely to be sloping downward to the north.

A groundwater monitoring well (LP-4) was installed in a 40-foot boring near the western curb of Sunset Cliffs Boulevard just north of Monaco Street and Pump Station No. 35. The monitoring well was installed approximately 40 feet from the bluff face. The monitoring well was installed at this location because of the seeps observed at the bluff face and the storm conveyance pipes running south on Monaco Street out letting in the bluff face. Directly after drilling and well installation on May 15, 2006, no groundwater was observed to the bottom of well at Elevation +4 feet. On June 11, 2006, the groundwater level was measured 37 feet from the ground surface at Elev. +7 feet. On July 8, 2006, the groundwater depth decreased slightly by 0.3 feet.

A groundwater monitoring well (HP-1) was installed in a 40-foot boring in the park area just south of Ladera Street and east of the stairway at the south end of Linear Park. The monitoring well was installed approximately 40 feet from the bluff face. The monitoring well was installed at this location because of the seeps observed at the bluff face just south of the bottom of the stairway. Directly after drilling and well installation on May 17, 2006, no groundwater was observed to the bottom of well at Elevation +28 feet. On June 11, 2006, the groundwater level was measured 33 feet from the ground surface at Elev. +35 feet. On July 8, 2006, the groundwater depth increased slightly by 0.5 feet. The groundwater appears to be perched on a dark gray siltstone bed encountered at approximately Elevation +39½ feet.

A groundwater monitoring well (HP-2) was installed in a 30-foot boring in area just down slope of the baseball field constructed at the southern end of the Hillside Park near the top of the coastal bluff. The monitoring well was installed approximately 300 feet from the bottom of the bluff and approximately 40 feet from the baseball field. The monitoring well was installed at this location because of the irrigation used for the ball field and its potential to flow through the fill and upper terrace deposits to the sedimentary bedrock of the Point Loma Formation. The geologic observation at this boring site indicated fill, residual soils, and terrace deposits totaling 28 feet thick overlying siltstone of the Point Loma Formation. No groundwater was observed to be resting on the contact between the terrace deposits and siltstone directly after drilling and well installation on May 18, 2006. In groundwater readings taken on June 11, 2006 and July 8, 2006, no groundwater was observed in the groundwater monitoring well. If any infiltration of water does occur through the terrace deposits, it does not become perched on the siltstone at a depth of 28 feet. The bottom sand layer of the terrace deposits indicated very moist conditions but the geologic contacts may slope downward allowing any water to flow to the face of the bluffs without creating a perched water zone.

A groundwater monitoring well (HP-4) was installed in a 25-foot boring in an area just down slope of the PLNU's ball field on Western Loop Road. The monitoring well was installed approximately 650 feet from the ocean on the eastern side of Hillside Park. The monitoring well was installed at this location because of the irrigation used for the athletic field and its potential to flow through the fill and upper terrace deposits to the sedimentary bedrock of the Point Loma Formation. In addition, a dense growth of phreatophytic vegetation existed directly west of the monitoring well location on the slope between the athletic field and Western Loop Road. The geologic observation at this boring site indicated fill and residual soils of 6 feet thick overlying siltstone and claystone of the Point Loma Formation. No groundwater was observed to be resting on the contact between the residual soils and siltstone directly after drilling and well installation on May 15, 2006. In groundwater readings taken on June 11, 2006 and July 8, 2006, no groundwater was observed in the groundwater monitoring well. If any infiltration of water does occur through the fill and residual soils, it does not become perched on the upper siltstones of the Point Loma Formation to a depth of 25 feet. The presence of the phreatophytic vegetation indicates very moist conditions however the less permeable geologic contacts may slope downward allowing any water to flow into the erosion gullies within Hillside Park without creating a perched water zone. Seeps through the surface of the distressed pavement at this location indicate water may likely be flowing downhill in the base course of the pavement of Western Loop Road.

We drilled the Boring HP-3 down slope on Western Loop Road and directly after Boring HP-4. This location was selected because it was directly down slope from the athletic field of Point Loma Nazarene University. The conditions at HP-4 were observed to be pavement placed directly over siltstone of Point Loma Formation. The conditions in the siltstone and sandstone of the Point Loma Formation were similar to the previous boring but with less moisture within the formation. Due to the unlikelihood that perched groundwater would be encountered at this location with the observed geology and moisture characteristics of the formation, no groundwater monitoring well was installed.

Table 1 summarizes the perched groundwater monitoring readings taken for this study.

The elevations of the ground surface of each monitoring wells is based upon aerial topographic plans provided by Dudek and Associates with contours at every foot. During drilling, the depth of groundwater was estimated to the nearest foot. In subsequent readings, the depth of groundwater was measured to the nearest 0.1 foot from the top of the groundwater monitoring well. The top of the monitoring well covers should be surveyed if readings with a greater accuracy are desired.

The elevation of the perched groundwater may fluctuate between seasons and years of relatively wet or dry weather. We recommend the groundwater wells be monitored at least bi-monthly by the City of San Diego prior to implementation of drainage improvements and after implementation of drainage improvements. The groundwater monitoring wells should be periodically maintained at the surface to prevent any leaks into the wells and should be abandoned in accordance with the regulations of the County of San Diego, Department of Environmental Health (Reference 8) after completion of the monitoring program.

PERMEABILITY OF SITE SOILS

The rate of flow of water through soil or bedrock is controlled by the material's permeability in accordance with Darcy's Law. The average flow velocity through a material is proportional to the hydraulic gradient by the coefficient of permeability (k) (or hydraulic conductivity). Water flows through soils in the interconnected voids between the soil particles. The coefficient of permeability for soil depends on the soil type, particle size distribution, void ratio, and homogeneity of the soil mass. In bedrock, fractures and the connectivity of the factures can increase the permeability significantly.

A material with a high coefficient of permeability allows water to drain through it easily. In general, clays have a low permeability coefficient with water draining through it very slowly. Sands have a high permeability coefficient with water draining through it relatively quickly. Intact sedimentary bedrock will have a relatively low permeability coefficient but the overall permeability of the formation will be dependent on the amount of fractures. Typical permeability coefficients for soil types (Reference 9) have been summarized below:

Soil Type	Coefficient of Permeability – k (cm/sec)	Drainage Characteristic
Uniform Coarse Sand	0.4	Good
Uniform Medium Sand	0.1	Good
Clean, well graded sand and gravel	0.01	Good
Uniform, fine sand	4×10^{-3}	Good
Silty Sand	10^{-4}	Good/Poor
Uniform Silt	5×10^{-5}	Poor
Sandy Clay	5×10^{-6}	Poor
Silty Clay	1×10^{-6}	Poor/Practically Impervious
Clay	1×10^{-7}	Practically Impervious

The coefficient of permeability of representative soil samples of the terrace deposits of the Bay Point Formation and sedimentary bedrock of the Point Loma Formation was estimated by laboratory tests. The tests were performed by a constant-head method in accordance with ASTM D 2434. Relatively undisturbed soils samples collected during the drilling program were extruded from ring samples and placed in a constant-head permeameter and saturated. The flow of water through the saturated samples at three constant head conditions under laminar flow was measured to estimate the coefficient of permeability.

The coefficient of permeability for four samples of the terrace deposits ranged from 1.1×10^{-4} to 2.0×10^{-7} cm/sec. This indicates poor to practically impervious drainage characteristics of the soils tested in the terrace deposits. The coefficient of permeability for two samples of the sedimentary bedrock ranged from 1.3×10^{-4} to 2.6×10^{-6} cm/sec. This indicates poor drainage characteristics of the samples tested in the sedimentary bedrock. The results of the permeability testing are presented in Appendix B.

LIMITATIONS

The report, exploration logs, and other materials resulting from GPI's efforts were prepared exclusively for use by Dudek and Associates and their consultants for their drainage study. The report is not intended to be suitable for reuse on extensions or modifications of the project or for use on any project other than the currently proposed development as it may not contain sufficient or appropriate information for such uses. If this report or portions of this report are provided to contractors or included in specifications, it should be understood that they are provided for information only. This report cannot be utilized by another entity without the express written permission of GPI. This report is an instrument of our services and remains the property of GPI.

Soil deposits may vary in type, strength, and many other important properties between points of exploration due to non-uniformity of the geologic formations or to man-made cut and fill operations. While we cannot evaluate the consistency of the properties of materials in areas not explored, the conclusions drawn in this report are based on the assumption that the data obtained in the field and laboratory are reasonably representative of field conditions and are conducive to interpolation and extrapolation.

Our investigation and evaluations were performed using generally accepted engineering approaches and principles available at this time and the degree of care and skill ordinarily exercised under similar circumstances by reputable Geotechnical Engineers practicing in this area. No other representation, either expressed or implied, is included or intended in our report.

Respectfully submitted,
Geotechnical Professionals Inc.

REGISTERED PROFESSIONAL ENGINEER
DONALD A. CORDS
No. GE 2529
Exp. 6/30/07
GEOTECHNICAL
STATE OF CALIFORNIA

Donald A. Cords, G.E.
Associate

REGISTERED GEOLOGIST
THOMAS G. HILL
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Thomas G. Hill, C.E.G.
Consulting Engineering Geologist

Exp 9/30/08

DAC/TGH:sph

Enclosures: References
Table 1 - Perched Groundwater Measurements
Figure 1 - Site Location Plan
Figure 2(a to b) - Site Plan
Figure 3 (a to e) - Boring Location Plan
Figure 4 - Site Photographs
Figure 5 (a to e) - Geologic Cross-Sections
Appendix A - Exploratory Borings and Well Installation
Appendix B - Laboratory Tests
Appendix C - Response to Comments

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3. Kennedy, M.P., and Peterson, G.L., 1975, "Geology of San Diego Metropolitan Area, California," California Division of Mines and Geology Bulletin 200.

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TABLE 1

PERCHED GROUNDWATER MEASUREMENTS

Boring No./ Location	Surface Elevation, feet	Well Depth, feet	Measurement Date	Perched Groundwater Depth, feet	Perched Groundwater Elevation
LP-1 Adair Parking	+35	31	5/16/06 6/11/06 7/8/06	27 21.3 20.6	+ 8 +14 +14
LP-2 Froude Street	+43	26	5/16/06 6/11/06 7/8/06	18 13.6 13.7	+25 +29 +29
LP-3 Cordova Street	+64	31	5/17/06 6/11/06 7/8/06	Not Encountered Not Encountered Not Encountered	--- --- ---
LP-4 Monaco Street	+44	40	5/15/06 6/11/06 7/8/06	Not Encountered 37.2 36.9	--- +7 +7
HP-1 Ladera Street	+68	40	5/17/06 6/11/06 7/8/06	Not Encountered 32.9 33.4	--- +35 +35
HP-2 South Coastal Terrace	+83	30	5/18/06 6/11/06 7/8/06	Not Encountered Not Encountered Not Encountered	--- --- ---
HP-3 Western Loop South	+165	20	No well installed.		
HP-4 Western Loop North	+187	25	5/15/06 6/11/06 7/8/06	Not Encountered Not Encountered Not Encountered	--- --- ---

Notes:

Elevations estimated to nearest foot from topographic plan provided by Dudek & Associates (dated 11/14/06).

First groundwater measurements taken directly after drilling and installing monitoring wells to the nearest foot.

Groundwater depth measurements from elevation of well cover.

Groundwater depth to 0.1 foot during subsequent readings.

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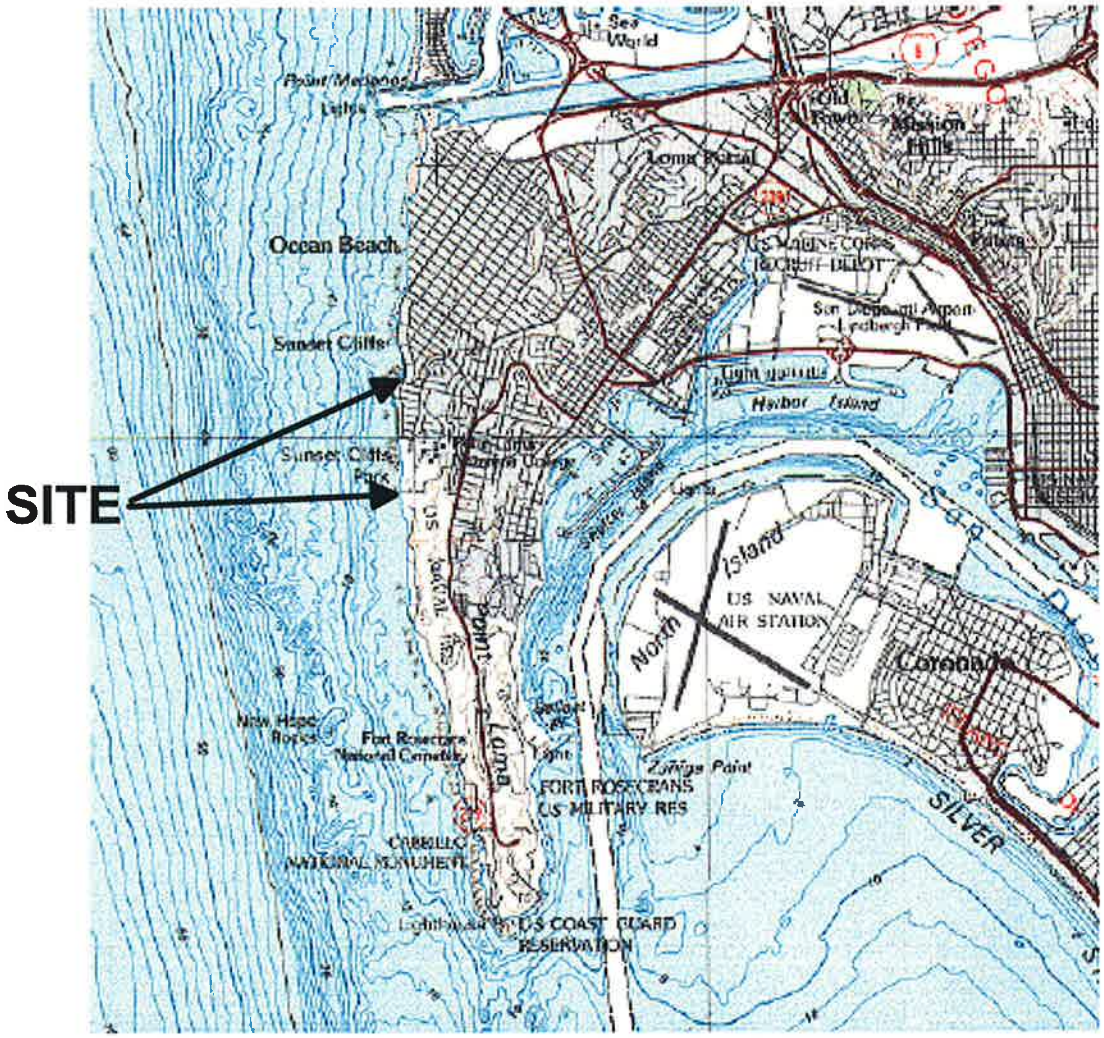

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SITE LOCATION MAP REPRODUCED FROM TOPO! 1997

GEOTECHNICAL PROFESSIONALS, INC.

SUNSET CLIFFS NATURAL PARK

GPI PROJECT NO. 2081.I	NO SCALE

SITE LOCATION MAP

FIGURE 1

0 500 1000 FEET

EXPLANATION

LP-1 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING AND GRONDWATER MONITORING WELL

HP-3 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING ONLY

APPROXIMATE BOUNDARY OF LINEAR PARK

BASE PLAN REPRODUCED FROM GOOGLE EARTH

GPI GEOTECHNICAL PROFESSIONALS, INC.

SUNSET CLIFFS NATURAL PARK

GPI PROJECT NO.:2081.I | SCALE: 1"=500'

SITE PLAN
Linear Park

FIGURE 2(a)

EXPLANATION

LP-1 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING AND GRONDWATER MONITORING WELL

HP-3 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING ONLY

APPROXIMATE BOUNDARY OF HILLSIDE PARK

0 300 600 FEET

BASE PLAN REPRODUCED FROM GOOGLE EARTH

GPI GEOTECHNICAL PROFESSIONALS, INC.

SUNSET CLIFFS NATURAL PARK

GPI PROJECT NO.:2081.I | SCALE: 1"=300'

Revision 9/06

SITE PLAN
Hillside Park

FIGURE 2(b)

EXPLANATION

APPROXIMATE LOCATION OF EXPLORATORY BORING AND GROUNDWATER MONITORING WELL

LOCATION OF GENERALIZED GEOLOGIC CROSS-SECTION

BASE PLAN REPRODUCED FROM GOOGLE EARTH

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SUNSET CLIFFS NATURAL PARK

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BORING LOCATION PLAN
Adair Street Parking

FIGURE 3(a)

EXPLANATION

APPROXIMATE LOCATION OF EXPLORATORY BORING AND GROUNDWATER MONITORING WELL

LOCATION OF GENERALIZED GEOLOGIC CROSS-SECTION

BASE PLAN REPRODUCED FROM GOOGLE EARTH

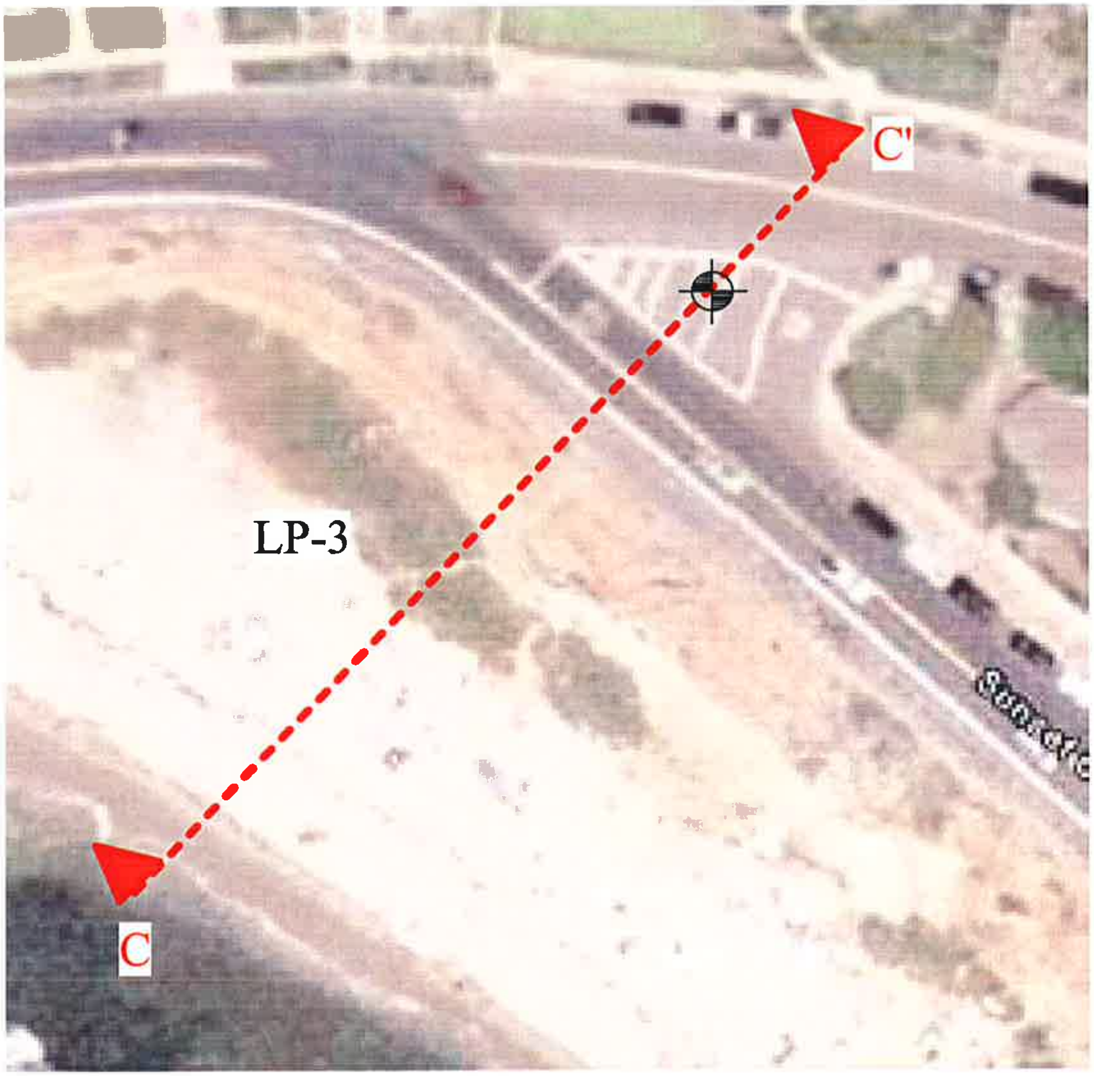
GPI GEOTECHNICAL PROFESSIONALS, INC.

SUNSET CLIFFS NATURAL PARK

GPI PROJECT NO. 2081.I | SCALE: 1"=50'

BORING LOCATION PLAN
Froude Street

FIGURE 3(b)



0 50 100 FEET

EXPLANATION

APPROXIMATE LOCATION OF EXPLORATORY BORING AND GROUNDWATER MONITORING WELL

LOCATION OF GENERALIZED GEOLOGIC CROSS-SECTION

BASE PLAN REPRODUCED FROM GOOGLE EARTH

GEOTECHNICAL PROFESSIONALS, INC.

SUNSET CLIFFS NATURAL PARK

GPI PROJECT NO. 2081.I

SCALE: 1"=50'

BORING LOCATION PLAN
Cordova Street

FIGURE 3(c)

EXPLANATION

APPROXIMATE LOCATION OF EXPLORATORY BORING AND GROUNDWATER MONITORING WELL

LOCATION OF GENERALIZED GEOLOGIC CROSS-SECTION

BASE PLAN REPRODUCED FROM GOOGLE EARTH

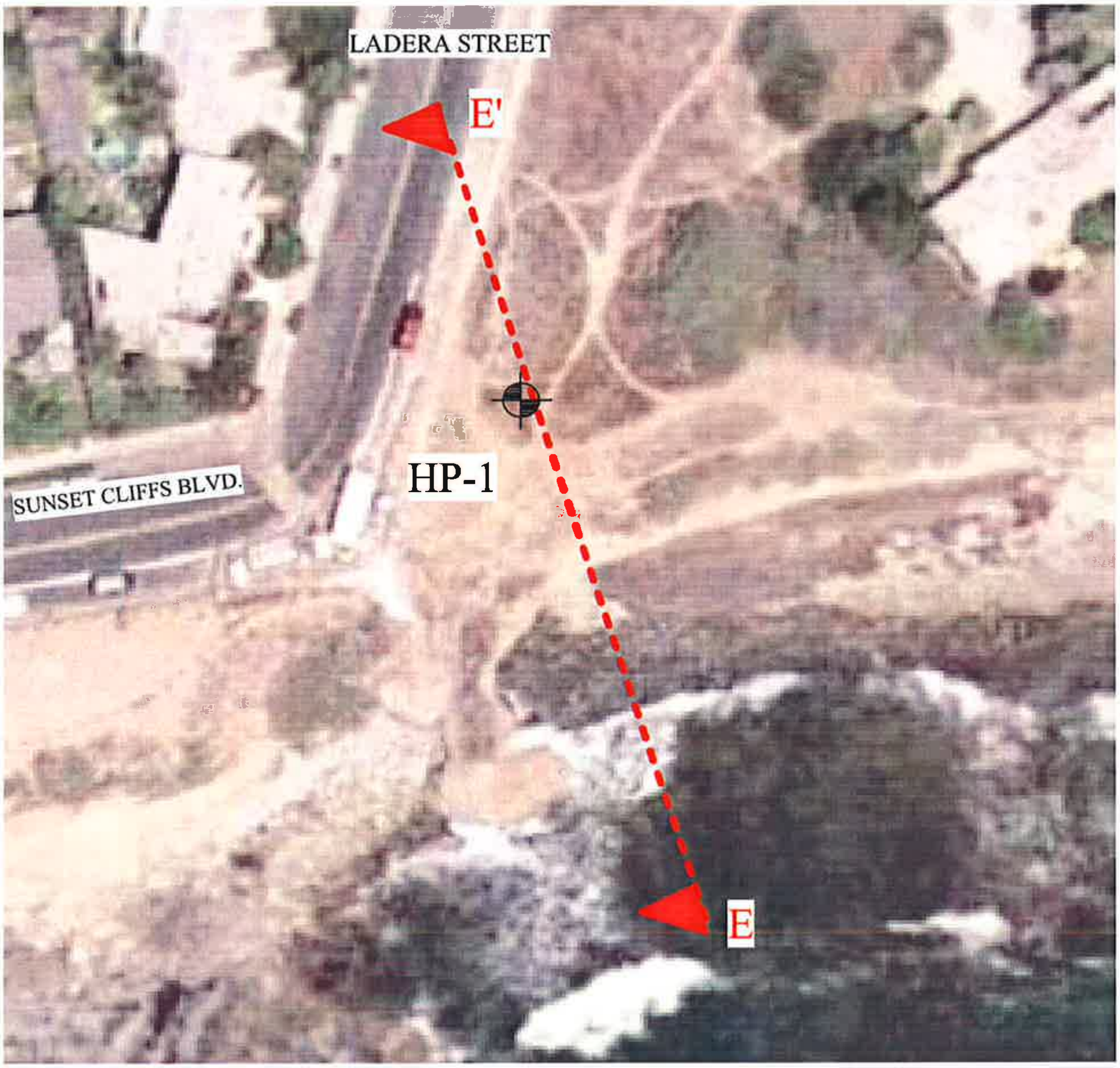
GPI GEOTECHNICAL PROFESSIONALS, INC.

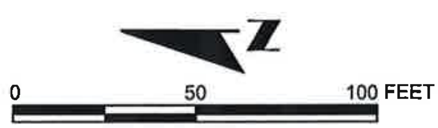
SUNSET CLIFFS NATURAL PARK

GPI PROJECT NO. 2081.I	SCALE: 1"=50'

BORING LOCATION PLAN
Monaco Street

FIGURE 3(d)





EXPLANATION

APPROXIMATE LOCATION OF EXPLORATORY BORING AND GROUNDWATER MONITORING WELL

LOCATION OF GENERALIZED GEOLOGIC CROSS-SECTION

BASE PLAN REPRODUCED FROM GOOGLE EARTH

GPI GEOTECHNICAL PROFESSIONALS, INC.

SUNSET CLIFFS NATURAL PARK

GPI PROJECT NO. 2081.I | SCALE: 1"=50'

BORING LOCATION PLAN
Ladera Street

FIGURE 3(e)

Figure 4A - Bluff face in front of parking lot south of Adair Street. Terrace deposits seen overlying Point Loma Formation. Terrace deposits of Bay Point Formation form moderate slope and erodible. Minor sea cave at base in Point Loma Formation.

Figure 4B: MSE wall for bluff stabilization along with rip-rap at base near Adiar Street. Terrace deposits of Bay Point Formation seen overlying Point Loma Formation.

Figure 4C: Bluff face in front of Froude Street intersection. Moderately sloping and eroding terrace deposits of Bay Point Formation overlying sedimentary rock of Point Loma Formation with rip-rap protecting base of bedrock.

Figure 4D: Bluff face behind beach at Cordova Street intersection. Large sand beach and shore platform exposed. Terrace deposits of Bay Point Formation overlying northeast dipping layers of Point Loma Formation overlying beach..

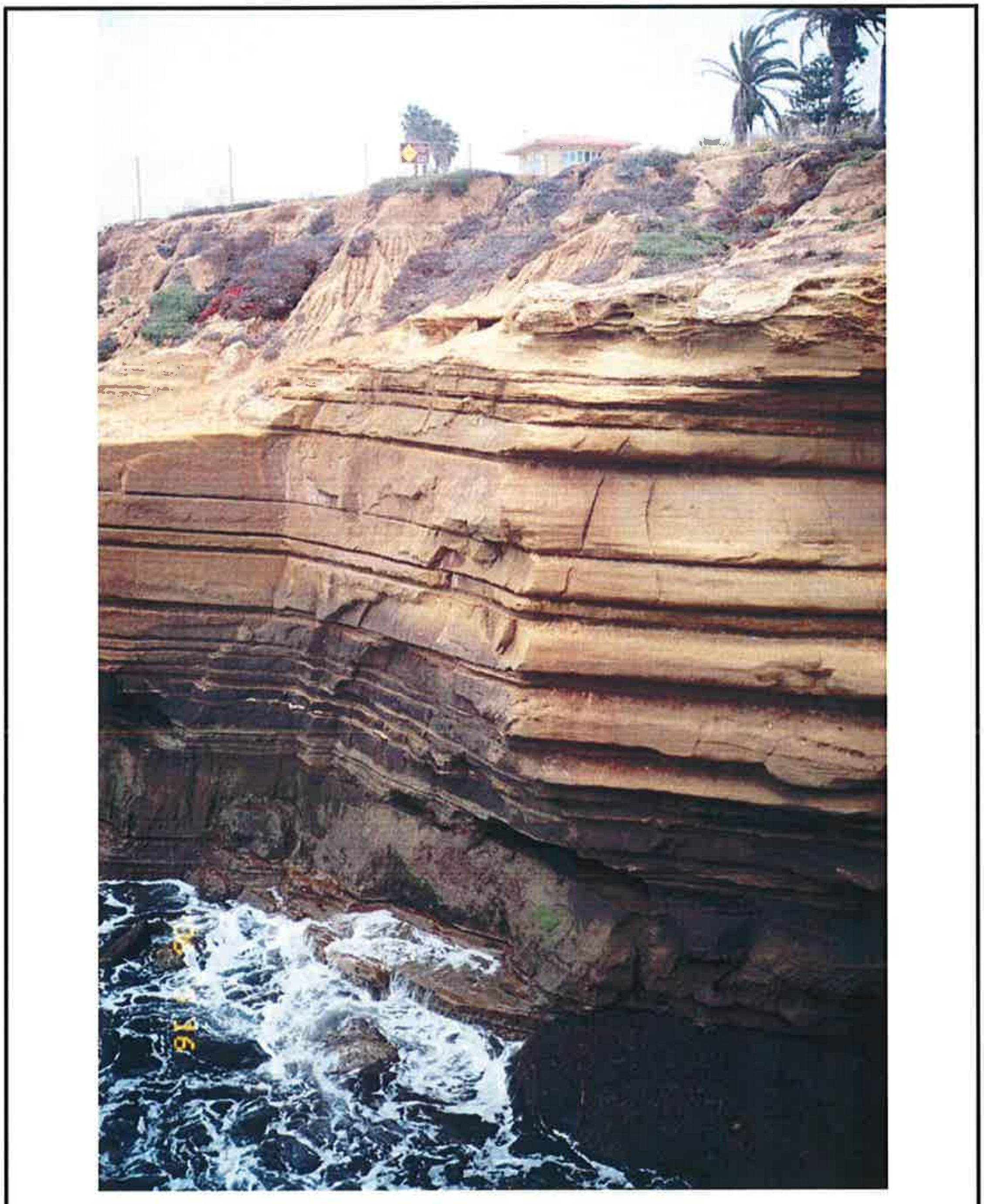


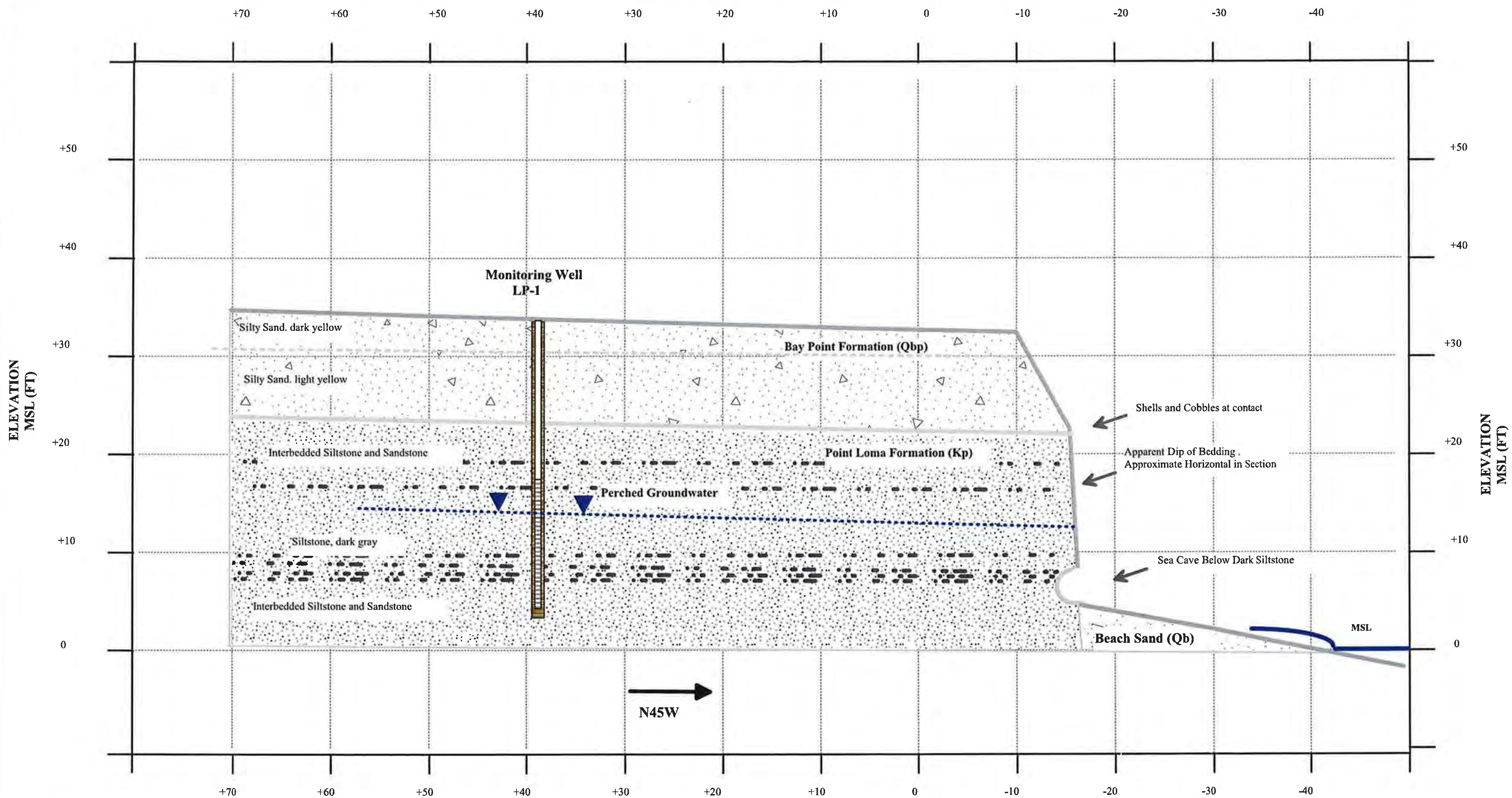
Figure 4E: Bluff face in front of Monaco Street and north of Pump Station No. 35. Sandstone with irregularly weathered with evidence of seepage at mid-cliff face.

Figure 4F: Storm drain outfalls north of Pump Station 35 at Monaco Street. Sea cave exposed at base with rip rap placed for cliff protection. Terrace deposits of Bay Point Formation and fill overlying Point Loma Formation just above level of outfalls.

Figure 4G: Bluff Face just south of Ladera Street stairway. Cobble beach in area of high wave energy. Seeps and stratification in Point Loma Formation observed to overlying terrace deposits of Bay Point Formation.

Figure 4H: Seepage in bluff face supporting vegetation. Stratification and weathering of Point Loma Formation with overlying terrace deposits of Bay Point Formation

Figure 4I: Bluff face just south of Ladera Street. Significant water seepage throughout lower levels of cliff. Relatively moderately sloped terrace deposits of Bay Point Formation overlying near vertical sedimentary bedrock of Point Loma Formation.



SUBSURFACE UNITS:

Bay Point Formation (Q_{bp})

Point Loma Formation (K_{pl})

Beach Sand (Q_b)

SECTION A-A': LOOKING SOUTH

HORIZONTAL SCALE 1"=10 '
VERTICAL SCALE: 1"=10 '

Notes:
1) This section is based on geologic conditions at exploration location and at surface exposures mapped during investigation. Localized variations could occur. The section is intended for descriptive purposes only.
2) See Figure 3 for location of cross-section.

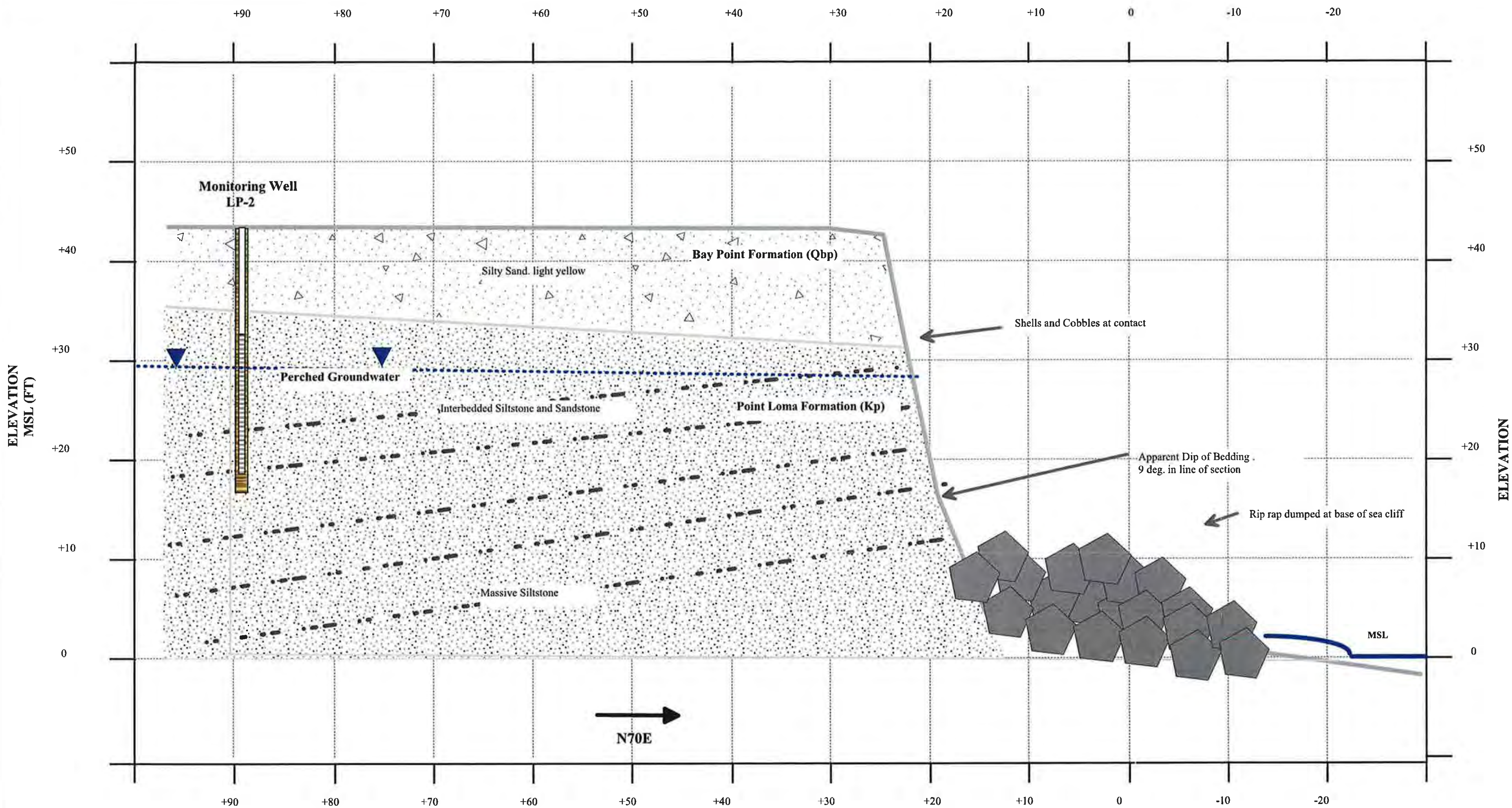
GPI GEOTECHNICAL PROFESSIONALS, INC.

SUNSET CLIFFS NATURAL PARK

GPI PROJECT NO.:2081.I | SCALE: AS NOTED

GENERALIZED GEOLOGIC CROSS-SECTION
Adair Street

FIGURE 5(a)



SUBSURFACE UNITS:

Bay Point Formation (Q_{bp})

Point Loma Formation (K_{pl})

Beach Sand (Q_b)

SECTION B-B': LOOKING SOUTH

HORIZONTAL SCALE 1"=10 '
VERTICAL SCALE: 1"=10 '

Notes:
1) This section is based on geologic conditions at exploration location and at surface exposures mapped during investigation. Localized variations could occur. The section is intended for descriptive purposes only.
2) See Figure 3 for location of cross-section.

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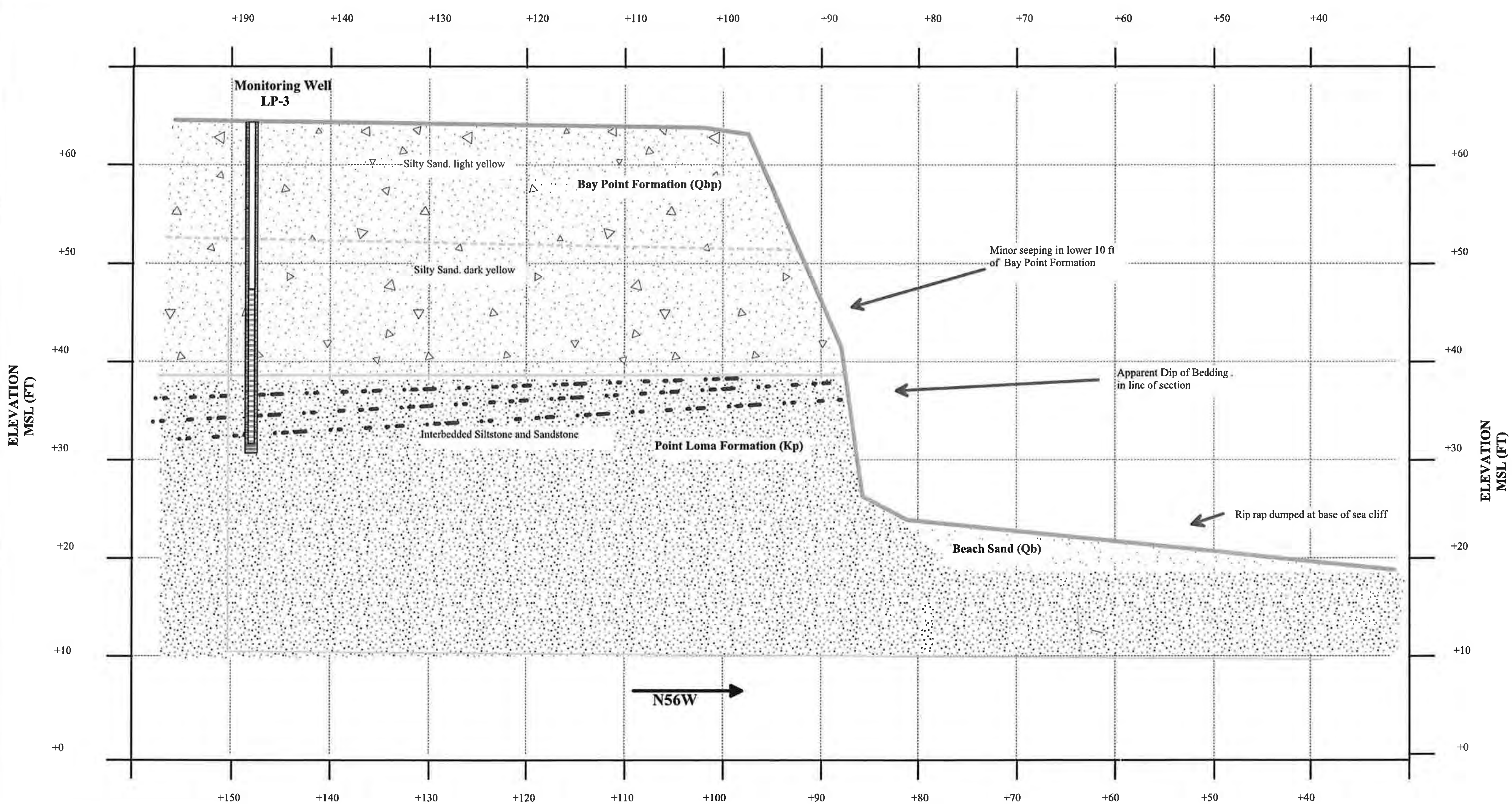
GPI PROJECT NO.:2081.I | SCALE: AS NOTED

GENERALIZED GEOLOGIC CROSS-SECTION
Froude Street

FIGURE 5(b)



SUBSURFACE UNITS:

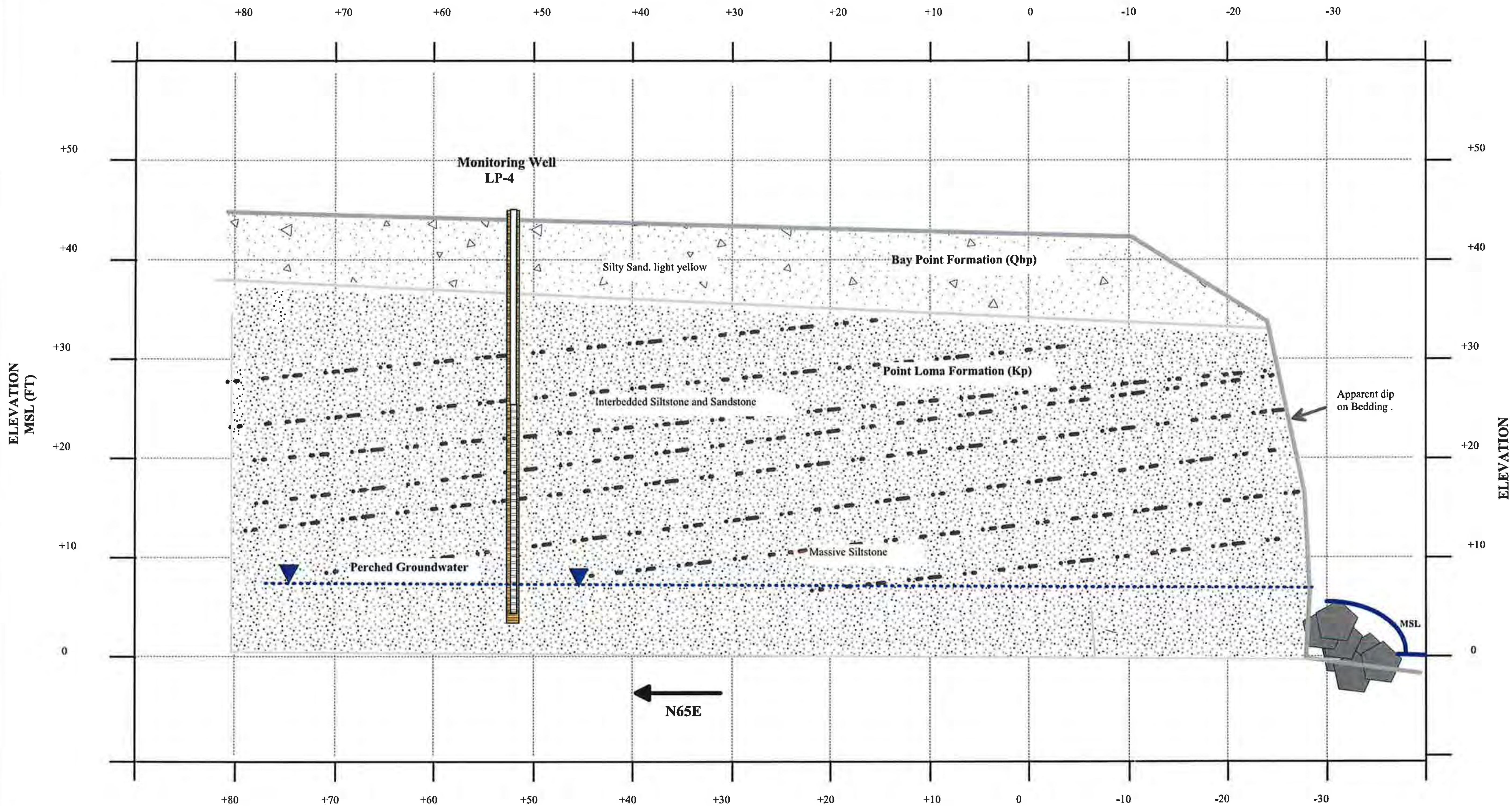
- Bay Point Formation (Q_{bp})
- Point Loma Formation (K_{pl})
- Beach Sand (Q_b)

SECTION C-C': LOOKING SOUTH

HORIZONTAL SCALE 1"=10 '
VERTICAL SCALE: 1"=10 '

Notes:
1) This section is based on geologic conditions at exploration location and at surface exposures mapped during investigation. Localized variations could occur. The section is intended for descriptive purposes only.
2) See Figure 3 for location of cross-section.

GPI GEOTECHNICAL PROFESSIONALS, INC.	GENERALIZED GEOLOGIC CROSS-SECTION Cordova Street
SUNSET CLIFFS NATURAL PARK	
GPI PROJECT NO.:2081.I — SCALE: AS NOTED	FIGURE 5(c)



SUBSURFACE UNITS:

- Bay Point Formation (Q_{bp})
- Point Loma Formation (K_{pl})
- Beach Sand (Q_b)

SECTION D-D': LOOKING SOUTH

HORIZONTAL SCALE 1"=10 '
VERTICAL SCALE: 1"=10 '

Notes:
1) This section is based on geologic conditions at exploration location and at surface exposures mapped during investigation. Localized variations could occur. The section is intended for descriptive purposes only.
2) See Figure 3 for location of cross-section.

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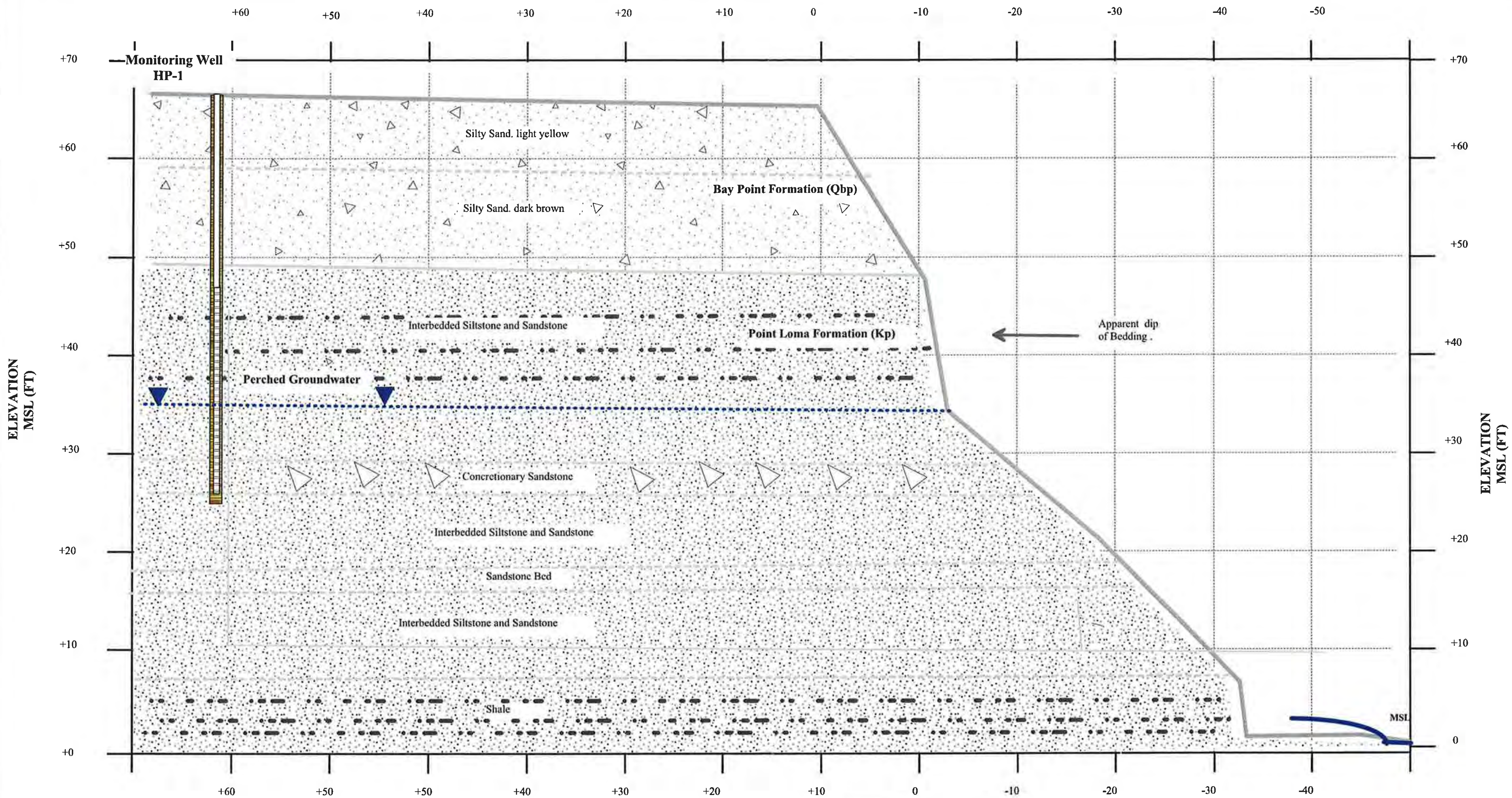
SUNSET CLIFFS NATURAL PARK

GPI PROJECT NO.:2081.I | SCALE: AS NOTED

GENERALIZED GEOLOGIC CROSS-SECTION
Monaco Street

FIGURE 5(d)



SUBSURFACE UNITS:

Bay Point Formation (Q_{bp})

Point Loma Formation (K_{pl})

Beach Sand (Q_b)

SECTION E-E': LOOKING SOUTH

HORIZONTAL SCALE 1"=10'
VERTICAL SCALE: 1"=10'

Notes:
1) This section is based on geologic conditions at exploration location and at surface exposures mapped during investigation. Localized variations could occur. The section is intended for descriptive purposes only.
2) See Figure 3 for location of cross-section.

GPI GEOTECHNICAL PROFESSIONALS, INC.

SUNSET CLIFFS NATURAL PARK

GPI PROJECT NO.:2081.I | SCALE: AS NOTED

GENERALIZED GEOLOGIC CROSS-SECTION
Ladera Street

FIGURE 5(e)

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APPENDIX A

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MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				0	Surface: 4 inches AC Pavement	35
					Bay Point Formation (Qbp): **SILTY SAND (SM),** dark yellowish brown, moist	
6.8	113	58	D	5	**SILTY SAND (SM)**, light yellowish brown, damp, dense, massive	30
				10		25
19.2	106	50/1"	D		Point Loma Formation (Kp): **SANDSTONE**, yellow brown, moist, very dense, with interbedded gray siltstone	
				15	@ 12 feet, yellow brown sandstone and gray siltstone	20
				20		15
				25	@ 25 to 26 feet, gray siltstone cuttings	10
					@ 27 feet, perched groundwater, fresh	
		50/6"	D	30	@ 30 feet, no sample recovery	5

Total Depth of 31 feet.
No caving.

Location: Parking Lot S of Adair Street

Well Construction:
Flush Mounted Well Cover.
0-15 feet: Solid 4" PVC Casing
15-30.5 feet: Slotted 4" PVC Casing

Well Backfill:
0-3 feet: concrete
3-12 feet: bentonite chips
12-31 feet: clean sand

SAMPLE TYPES
C Rock Core
S Standard Split Spoon
D Drive Sample
B Bulk Sample
T Tube Sample

DATE DRILLED: 5-16-06
EQUIPMENT USED: 8" Air Percussion
GROUNDWATER LEVEL (ft): 27.0

GPI

PROJECT NO.: 2081.I
SUNSET CLIFFS NATURAL PARK

LOG OF BORING NO. LP-1

FIGURE A-1

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS*	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				0	Surface: 6 inches AC Pavement	
					Bay Point Formation (Qbp):	
					SILTY SAND (SM), yellowish brown, moist, fine grained	40
		94/8"	D	5		
					SILTY SAND (SM), light tan, dry to slightly moist, very dense	
					Point Loma Formation (Kp):	35
					SANDSTONE, yellowish brown, moist, dense, some	
10.0		50/5"	D	10	discoloration of terrace deposit to dark brown at contact.	
						30
				15		
					@ 18 feet, perched groundwater during drilling	25
		50/5"	D	20	@ 20 feet, no sample recovery	
					@22.5 feet, cuttings become gray, cemented sandstone, drill rate slowed.	20
				25		

Total Depth of 26 feet.
No caving.

Location: Froude Street

Well Construction:
Flush Mounted Well Cover.
0-10 feet: Solid 4" PVC Casing
10-25.5 feet: Slotted 4" PVC Casing

Well Backfill:
0-3 feet: concrete
3-8 feet: bentonite chips
8-26 feet: clean sand

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 5-16-06
EQUIPMENT USED: 8" Air Percussion
GROUNDWATER LEVEL (ft): 18.0

GPI

PROJECT NO.: 2081.I
SUNSET CLIFFS NATURAL PARK

LOG OF BORING NO. LP-2

FIGURE A-2

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
				0	Surface: 8 inches AC Pavement	
					Bay Point Formation (Qbp): **SILTY SAND (SM),** yellowish brown, very moist, medium dense to dense, with trace clay.	
9.7	110	45	D			60
				5		
						55
6.9	109	85/5"	D	10	@ 10 feet, very dense	
					@ 12 feet, dark brown, moist	
					@ 14 feet, yellow brown	50
				15		
						45
7.1	102	50/6"	D	20	@ 20 feet, dark yellowish brown	
						40
12.1		50/3"	D	25	Point Loma Formation (Kp): **INTERBEDDED SANDSTONE AND SILTSTONE,** olive gray, moist, very dense,	
						35
		50/3"	D	30	**INTERBEDDED SILTSTONE AND CLAYSTONE,** gray, moist, very dense,	

This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Total Depth of 31 feet.
No caving.

Location: Cordova Street

Well Construction:
Flush Mounted Well Cover.
0-15 feet: Solid 4" PVC Casing
15-30.5 feet: Slotted 4" PVC Casing

Well Backfill:
0-3 feet: concrete
3-14 feet: bentonite chips
14-31 feet: clean sand

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 5-17-06
EQUIPMENT USED: 8" Air Rotary
GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 2081.I
SUNSET CLIFFS NATURAL PARK

LOG OF BORING NO. LP-3

FIGURE A-3

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
				0	Fill (Af): **SANDY SILT (ML),** brown, slightly moist, with claystone fragments	80
15.4	117	28	D	5	Residual Soils (Qr): **SANDY CLAY(CL),** brown, moist, soft to firm	
					Bay Point Formation (Qbp): **CLAYEY SAND/SANDY CLAY (SC/CL),** yellowish brown, moist, very stiff, with white irregular caliche masses.	75
				10		
11.7	124	29	D		**SILTY SAND/CLAYEY SAND (SM/SC):** dark reddish brown, moist to very moist, medium dense	70
				15		65
7.9	121	47	D	20	**SILTY SAND(SM),** yellowish brown, moist, dense, trace clay	
					SAND (SP), yellowish brown, very moist, dense, fine grained, with shell fragments.	60
				25		55
17.3	106	90/7"	D	30	Point Loma Formation (Kp): **SILTSTONE,** gray, slightly moist, hard.	

This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Total Depth of 30 feet.
No caving.

Location: Southern Coastal Terrace

Well Construction:
Flush Mounted Well Cover.
0-15 feet: Solid 4" PVC Casing
15-29.5 feet: Slotted 4" PVC Casing

Well Backfill:
0-3 feet: concrete
3-14 feet: bentonite chips
14-30 feet: clean sand

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 5-18-06
EQUIPMENT USED: 8" Air Rotary
GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 2081.I
SUNSET CLIFFS NATURAL PARK

LOG OF BORING NO. HP-2

FIGURE A-6

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				0	Surface: 4 inches AC Pavement	
					Fill (Af): **SILTY CLAY (CL)**, gray, very moist	185
22.4	94	28	D	5	Residual Soils (Qr): **SANDY CLAY(CL)**, gray to brown, very moist, firm	180
					Point Loma Formation (Kp): **SILTSTONE/CLAYSTONE**, gray, moist, hard, highly fractured/disturbed, some porosity between fragments.	
		82/11"	D	10	**SILTSTONE**, gray and brown, slightly moist, hard, fractured, with thin roots along fracture surfaces.	175
				15		170
19.0		50/5"	D	20		165
				25		

Total Depth of 25 feet.
No caving.

Location: Western Loop Road (North)

Well Construction:
Flush Mounted Well Cover.
0-10 feet: Solid 4" PVC Casing
10-24.5 feet: Slotted 4" PVC Casing

Well Backfill:
0-3 feet: concrete
3-9 feet: bentonite chips
9-25 feet: clean sand

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 5-15-06
EQUIPMENT USED: 8" Air Percussion
GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 2081.I
SUNSET CLIFFS NATURAL PARK

LOG OF BORING NO. HP-4

FIGURE A-8

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APPENDIX B

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APPENDIX B

LABORATORY TESTS

INTRODUCTION

Representative undisturbed soil samples, tube samples and bulk samples were carefully packaged in the field and sealed to prevent moisture loss. The samples were then transported to our Cypress office for examination and testing assignments. Laboratory tests were performed on selected representative samples as an aid in classifying the soils and to evaluate the physical properties of the soils affecting foundation design and construction procedures. Detailed descriptions of the laboratory tests are presented below under the appropriate test headings. Test results are presented in the figures that follow.

MOISTURE CONTENT AND DRY DENSITY

Moisture content and dry density were determined from a number of the ring samples from the borings. The samples were first trimmed to obtain volume and wet weight and then were dried in accordance with ASTM D 2216. After drying, the weight of each sample was measured, and moisture content and dry density were calculated. Moisture content and dry density values are presented on the boring logs in Appendix A.

PERMEABILITY

The coefficient of permeability of representative soil samples was performed by Geologic Associates on soil samples provided by GPI. The tests were performed by a constant-head method in accordance with ASTM D 2434. Relatively undisturbed soils samples were extruded from ring samples and wax placed around the sample perimeter to mitigate against side seepage. The samples were placed in a constant-head permeameter and saturated. The flow of water through the saturated samples at three constant head conditions under laminar flow was measured to estimate the coefficient of permeability. The results are as follows.

BORING NO.	DEPTH (ft)	GEOLOGIC/SOIL DESCRIPTION	COEFFICIENT OF PERMEABILITY (cm/sec)
HP-2	12	<u>Bay Point Formation</u>: Sandy Clay (CL)	2.0×10^{-7}
LP-1	12	<u>Point Loma Formation:</u> Silty Sandstone	2.6×10^{-6}
LP-4	30	<u>Point Loma Formation:</u> Silty Sandstone	1.3×10^{-4}
LP-1	5	<u>Bay Point Formation</u>: Clayey Sandy (SC)	1.1×10^{-4}
LP-3	10	<u>Bay Point Formation</u>: Sandy Clay (CL)	1.5×10^{-6}
HP-1	5	<u>Bay Point Formation</u>: Silty Sand (SM)	6.4×10^{-4}

5. It should be noted that the SCNP Master Plan shows the section of road, which traverses through the parkland as *Western Loop Road* ... not Lomaland Drive. PLNU Lomaland Drive connects with this road. *This reference is found on page 5 paragraph 2, lines 3 & 7; page 8, line 5; page 9, line 3.*

6. Page 2, paragraph 2, line 4, states "The majority of upland drainage at the Hillside Park .... Is conveyed as sheet flow...." Members of the SCNPRC believes that describing the majority of upland drainage as being conveyed as sheet flow is in error and wonders why other features such as 24", 18", 10" and other pipes, which end on parkland, are not mentioned as being significant in collecting, conveying, and concentrating runoff. The Western Loop Road carries some of this runoff to a culvert which has focused runoff. The Western Loop Road carries some of this runoff to a culvert which has focused runoff so as to create an unnaturally eroded deep ravine. On page 59 of the SCNP Master Plan, this ravine is described as "approximately 30 feet deep and 40 feet wide, divides the Hillside Park and breaks the continuity of the Park ..." Some of the other eroded gullies are also either formed or exacerbated by concentrated upland runoff. Some of this erosion is threatening the sludge line in the area to the west of the Young Hall Parking Lot. Concentrated runoff seems to be significantly contributing to cliff retreat at various locations.

7. Page 4, paragraph 2, line 6. Athletic field should be singular. *Additional graded areas include the Western Loop Road, construction of PLNU's sewage infrastructure, creation of the Theosophist's dump site and garden areas, and construction of the City's high pressure sludge line.*

8. Page 4, paragraph 3, line 5. Sunset *Cliffs* Boulevard

9. Page 4, paragraph 4. It is our understanding that subterranean seepage and piping are also major sources of erosion and retreat of the bluffs. Pat Abbott, San Diego State University professor who has been studying this for many years, is one of the experts advocating that piping is a primary cause of the development of sea caves and other erosional features involving the cliffs.

10. Page 5, line 6. This is called *Garbage Beach* ... not Abs.

11. Page 5, 3rd paragraph, 1st line: Not along "eastern boundary ..." Note map of park boundary.

12. Page 5, first paragraph after "Geological Units" does not contain a sentence.

13. Last paragraph, 1st line: We are not sure there is any "western bike lane". Should it be called something else?

14. Page 8, 3rd paragraph, lines 2&3: "... upper athletic field on Lomaland Drive seems a poor reference. Probably this refers to PLNU's ball field which would be better referred to as such. Note that they have two ball fields so it

should be explicit. In line 3, the well was not installed on the "eastern limits of the Hillside Park". See park boundary.

15. Page 9, penultimate paragraph, last line. "increases" should be singular.

16. Page 11, last of 1st paragraph. It sounds like the City cannot use the collected data. What is really intended here?

17. References: Should there be any reference to Pat Abbott's book? We recognize you are not providing an exhaustive bibliography and that the book would be useful only as providing an overview of conditions.

18. Table 1, Perched Groundwater Measurements, discusses HP-Lower Ballfield measurements. Since this test well could not be drilled near the cliffs off the NW corner of the ballfield, where significant erosion is occurring along the cliffs, we question whether this well totally addresses the ballfield issues. Naming this boring location something else would be preferred. Perhaps South Coastal Terrace would be appropriate. Also note that "Lomaland South" and Lomaland North should more accurately be named *Western Loop Road North* and *Western Loop Road South* since both are within the park boundaries.

19. Page 5, paragraph 2, line 6. Please place a period after field and omit "constructed for the college" which is both inaccurate and irrelevant for the purposes of this geotechnical report. The SCNP Master Plan calls for stopping the irrigation of the field , re-contouring the area, re-vegetating with native habitat, and building a trail and viewing area to provide increased access to the southern portion of the park.

We have incorporated the comments as suggested into a revised report with the exception of Comment No. 6. We have incorporated the portion of Comment No. 6 describing the upland drainage being conveyed by storm drain pipes as well as sheet flow. The conclusions drawn in Comment No. 6 are outside the scope of the geotechnical data report. Without other published references to attribute the comments concerning eroded gullies being exacerbated by concentrated upland runoff, erosion threatening the sludge line, or concentrated runoff significantly contributing to cliff retreat, it is not appropriate for GPI to state those conclusions even though they appear to be based on sound evidence. It is more appropriate for Dudek and Associates to access these factors in their drainage study as they relate to erosion and cliff retreat.

The limitation discussed in Comment 16 is not intended to preclude the City of San Diego using the data in this report for drainage studies at Sunset Cliffs Natural Park. We have added the City of San Diego as an intended user of our report for the drainage studies. The purpose of this paragraph is to preclude the City, Dudek or other consultants from using the data in this report for other uses such as design of structures or uses unrelated to the drainage study.

We trust this information satisfies your current needs. Please do not hesitate to call if you have any questions on the contents of this report.

Sincerely,
Geotechnical Professionals Inc.

Donald A. Cords, G.E.
Associate

REGISTERED PROFESSIONAL ENGINEER
DONALD A. CORDS
No. GE 2529
Exp. 6/30/07
GEOTECHNICAL
STATE OF CALIFORNIA
OCT - 6 2006

DC:sph

APPENDIX B
Hydrology and Hydraulic Analysis

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SUNSET CLIFFS NATURAL PARK
HYDROLOGY AND HYDRAULIC ANALYSIS

FINAL REPORT

Prepared for:
Paul Jacob, P.E.
Park Planning and Development Division
Park and Recreation
City of San Diego
202 C Street
San Diego, California

Prepared by:
Dudek
605 Third Street
Encinitas, California 92024

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Table of Contents

1. HYDROLOGY AND HYDRAULIC ANALYSIS

Hydrology and Hydraulics are the primary analytical components of a Drainage Study. The Hydrology component recognizes the various factors that contribute to movement of water. On a Drainage Study, these typically constitute elements and characteristics of geographical, geological and climatic features such as physical terrain, soil and vegetation types and rainfall intensities and frequencies. Hydraulics, on the other hand, deals with the mechanics of movement of water, identifies the means and methods of transportation of the discharge, and quantifies the physical attributes of the transportation process.

2. PURPOSE OF HYDROLOGY ANALYSIS

The Sunset Cliffs Natural Park Hydrology Analysis (Analysis) is an integral component of the Sunset Cliffs Natural Park Drainage Study (Study) conducted by Dudek on behalf of the City of San Diego Department of Parks and Recreation. The purpose of the Analysis is to identify the drainage basins both within and up stream of the Sunset Cliffs Natural Park (Park), establish drainage patterns, determine slopes of terrain and streams, and estimate runoff based on various storm frequencies.

Sunset Cliffs Natural Park is located along the Pacific Ocean on the Western portion of the Point Loma peninsula in the City of San Diego. The park consists of a linear park along Sunset Cliffs Boulevard and a hillside park located South of Ladera Street and West of Point Loma Nazarene Collage. The Park's South boundary is the Point Loma Ecological Reserve in the Navy property.

The Sunset Cliffs Natural Park's tributary drainage basins begin at the top of the ridge of the Point Loma peninsula and terminate at the Pacific Ocean to the West. A significant part of the drainage basin lies upstream of the park and is extensively developed. The land development upstream of the linear park segment is primarily single family dwelling units while the land development upstream of the hillside park is the Point Loma Nazarene University. See **Figures 1 and 2** for drainage basin maps. The basin delineation was based primarily on a 1"=200' scale contour map but adjustments were made to accommodate roads, alleys and other manmade objects not clearly shown on the contour map. In some instances the delineation line was centered along rooflines.

The following sections discuss the methodology used for analysis and calculations, and separately identify and characterize the sub-basins within the linear park and the hillside park.

2.1 HYDROLOGY METHODOLOGY

Rational and Modified Rational Methods as defined in the San Diego County Hydrology Manual, 2003 edition, (Manual), are utilized to determine discharge from the site under existing conditions. Since the size of the drainage basin is less than one square mile, use of the Rational Method is recommended. Furthermore, all additional data is extracted from equations, tables, Figures and Nomographs provided within the Manual.

2.1.1 Rational Method

The Rational Method (RM) formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area (A), runoff coefficient (C), and rainfall intensity (I) for a duration equal to the time of concentration (T_c), which is the time required for water to flow from the most remote point of the basin to the location being analyzed. The RM formula is expressed as follows:

$$\mathbf{Q = C\ I\ A}$$

Where:

Q = peak discharge, in cubic feet per second (cfs)

C = runoff coefficient, proportion of the rainfall that runs off the surface (no units)

I = average rainfall intensity for a duration equal to the T_c for the area, in inches per hour, for a selected storm frequency

A = drainage area contributing to the design location, in acres

The runoff coefficient, C, is based on land use and soil type. Soil type and runoff coefficients were selected from the soil type map and runoff coefficient tables provided in the manual. The source of the runoff coefficient, C, is in Table 3.1 of the Manual. In cases where the soil type map indicates a basin with mixed soil types, the two corresponding C values were averaged for calculation. Furthermore, since the C values presented in the Table 3.3 do not take into account the effects of steep slopes, which increase the runoff, the C values for the hillside park were averaged with the C values representing the Low Density Residential area C values. The resultant data yields slightly larger yet reasonable runoff values. See Appendix for the C value table.

The soil type for the project site is a mixture of “B” and “C” where the higher elevations consist of less permeable soil type “C” and the lower elevations consist of soil type “B”.

The intensity was calculated using the following equation:

$$\mathbf{I = 7.44\ P_6\ D^{-0.645}}$$

Where:

P_6 = adjusted 6-hour storm rainfall amount in inches

D = duration in minutes (use T_c)

The Intensity-Duration Design Chart and the equation are for the 6-hour storm rainfall amount. In general, P_6 for the selected frequency should be between 45% and 65% of P_{24} for the selected frequency. If P_6 is not within 45% to 65% of P_{24}, P_6 should be increased or decreased as necessary to meet this criterion. The isopluvial lines are based on precipitation gauge data.

P_6 and P_{24} can be read from the isopluvial maps provided in **Appendix**.

For the RM, the T_c at any point within the drainage area is given by:

$$\mathbf{T_c = T_i + T_t}$$

The Time of Concentration is the time required for runoff to flow from the most remote part of the drainage area to the point of convergence. The T_c is composed of two components:

initial time of concentration (T_i) and travel time (T_t). The T_i is the time required for runoff to travel across the surface of the most remote subarea in the study, or "initial subarea." The T_t is the time required for the runoff to flow in a watercourse (e.g., swale, channel, gutter, pipe) or series of watercourses from the initial subarea to the point of interest.

2.1.2 Modified Rational Method

The Modified Rational Method (MRM) shall be used to determine the combined flows at a given junction when two or more independent drainage basins converge at the junction. The method calculates the peak flow Q at the junction when $T_c 1 < T_c 2$

$QT1=Q1+ (Tc 1/ T_c 2)*Q2$
$QT2=Q2+ (I2/I1)*Q1$

Where:
QT1 and QT2 =Discharge rate at the junction, in cfs
Q1 and Q2 = Discharge rate at tributary area 1 and 2, in cfs
T_c1 and T_c2=Time of concentration at tributary area 1 and 2, in minutes
I1 and I2 = Intensity at tributary area 1 and 2, in inches/hour

Select the larger Q as peak flow at the junction

New Intensity:
$I=Q/ (CA)$

New Time of concentration:
$T_c = (7.44*P6/I)^{1.55}$

2.1.3 Rainfall Frequencies

The scope of work for the Study identified storm frequencies to be used for this Analysis as 1, 5, 10 and 50 year storm events. Rainfall data, P_6 and P_{24}, were extrapolated from the isopluvial maps included in the San Diego County Hydrology Manual for each of the storm frequencies. Due to the limitations of the isopluvial maps, the extrapolated data are at best approximations. The data are summarized in the **Table 1**.

Table 1

Storm Frequency	P6 (inch)	P24 (inch)
1 year	1.00	1.60
5 year	1.30	2.10
10 year	1.50	2.80
50 year	1.90	3.30

3. BASIN HYDROLOGY

The basin analysis will be performed under two separate sections; the Linear Park and the Hillside Park. Each section shall identify the characteristics of the terrain, determine time of travel for storm water runoff to travel from the furthest point of the basin to the discharge point, and in subsections, will delineate the boundaries of each basin and subbasin. Since the T_c for developed areas were based on the velocity of the flows conveyed via closed conduits such as pipes and culverts, and open conduits such as open channels and gutters, the process to determine storm water runoff became iterative. This process is outlined as follows:

1. Using Figure 2-2 of the San Diego County Drainage Design Manual (Design Manual); "6-inch Gutter and Roadway Discharge-Velocity Chart", a gutter flow was assumed for a given basin using the average slope of the basin.
2. Based on the assumed flow and the basin slope, a flow velocity was interpolated from Figure 2-2.
3. The velocity was used to calculate T_t along the gutter and the T_c for the basin.
4. The T_c was used to calculate rainfall intensity and runoff using the rational method. The runoff calculated by using the rational method was compared with the assumed gutter flow.
5. The process was repeated until the assumed gutter flow rate and the calculated runoff rate converge within 0.5 cfs.

Though the above analysis includes some hydraulic analysis of the basin, a more complete hydraulic analysis including the effects of existing curb inlets within subbasins will be covered in Section 4. Only the individual basin/sub-basin runoff data shall be presented in this section.

3.1 LINEAR PARK BASIN DATA

The runoff from drainage tributaries which impacts the Linear Park section originates at the top of the Point Loma Peninsula. The terrain is steep East to West and relatively flat in the North to South direction. Currently most of the runoff is conveyed to the ultimate discharge location via gutter flow. However, several larger drainage basins have storm drain systems to capture runoff upstream of Sunset Cliffs Blvd. This reduces the amount of gutter flow, which in turn reduces or prevents overwhelming the inlet structures downstream and reduces overtopping the West curb of Sunset Cliffs Blvd.

The Linear Park has a 242 acre upstream tributary area and can be delineated to six major basins. Four of these basins are further separated to smaller subbasins based upon delineation using contours, roads, curbs and the presence of storm drain structures. The following is a general description of each basin and **Table 2** summarizes the basin data.

3.1.1 Basin X

The Northernmost basin, generally bounded by Sunset Cliffs Boulevard, Osprey Street, the ridgeline and the centerline of Point Loma Avenue, is composed primarily of areas external to the Park and drains to a location outside of park boundary. The actual Northern boundary of the basin extends North of Coronado Avenue, however only the area South of Point Loma Avenue has any impact upon the vicinity of the Park. Basin X, which is fully developed, is composed of nine subbasins. Basin X9, the upper most basin, is bounded by La Paloma Street to the South, Trieste Drive to the West, the ridge line to the East and Point Loma Avenue to the North. The storm water runoff flows North to Point Loma Avenue, drains West toward

Froude Street and discharges to a curb inlet West of Froude Street. The bypass discharges to Basins X3 and X6. Basin X8 is located South and West of Basin X9 and is generally bounded by Froude Street, Granger Street and Devonshire Drive. Storm water runoff flows to Tivoli Street and drains West to two curb inlets. Bypass flow discharges to Basin X5. Basin X7 is bounded by Basin X8, Osprey Street and Devonshire Drive, and the runoff drains Northeast between the curbs of Granger Street to two curb inlets. The bypass discharges to Basin X5. Basin X6 is bounded by Basin X8, Ebers Street and Point Loma Avenue, and the runoff drains West along Adair Street to a curb inlet. The bypass flow discharges to Basin X3. Basin X5 is bounded by Basins X7, X8, Adair Street, Osprey Street and the alley West of Devonshire Drive, and the runoff drains North along Devonshire Drive toward Adair Street to a curb inlet. The bypass flow discharges to Basin X2. Basin X4, bounded by Osprey Street, Sunset Cliffs Boulevard and the alley West of Devonshire Drive, is the only part of Basin X that directly impacts the Park and drains North along Sunset Cliffs Boulevard to a curb inlet located South of Adair Street. Bypass from Basin A discharges to Basin X4. Basin X3, bounded by Basins X5, X6, Ebers Street and Point Loma Avenue, is a small basin which drains along Ebers Street to Adair Street where the runoff enters a curb inlet. The bypass flow discharges to Basin X2. Basin X2, bounded by Basins X4, X5, Sunset Cliffs Boulevard, Point Loma Avenue and Adair Street, drains along Adair street to a curb inlet. The bypass flow discharges to Basin X1. Basin X1, bounded by Basins X2, X3, X5 and Point Loma Avenue, drains to a curb inlet at a sag location at the Southeast corner of the intersection of Sunset Cliffs Boulevard and Point Loma Avenue. The runoff from the entire Basin X which does not get intercepted into curb inlets along the way converges at this location outside of the park boundary.

3.1.2 Basin A

The basin, located South of Basin X, generally bounded by Sunset Cliffs Boulevard, Osprey Street and Novara Street is a fully developed basin. Runoff flows in a Northwest direction, eventually draining to the Northwest corner of the basin at the intersection of Sunset Cliffs Boulevard and Osprey Street.

3.1.3 Basin B

The basin located South of Basin A consists of six individual subbasins. Basin B3, the largest of the subbasins, is bounded by Novara Street, Moana Drive, and Piedmont Drive. Storm water runoff flows in a Southwest direction to the intersection of Hill Street and Devonshire Drive. Basin B4, South of B3 and roughly bounded by Piedmont Drive and Hill Street, drains runoff in a Southwest direction to a similar point at the intersection of Hill Street and Novara Street. Southeast of Basin B4 is Basin B6. Runoff flows West down Hill Street and drains near the corner of Hill Street and Amiford Drive. Basin B5, the smallest of the subbasins, is situated downstream from the Western corner of Basin B6. Drainage flows in a Northwest direction toward the intersection of Hill Street and Moana Drive. All four of these drainage basins drain into Basin B2. Basin B2 is located West of these basins and bounded by Marseilles Street to the South and Cordova Street to the West. Drainage flows West down both Hill Street and Marseilles Street to a point at the intersection of Hill Street and Cordova Street. Basin B1 is located to the West of Basin B2 and is bounded to the East by Cordova Street and to the West by the Pacific Ocean. Runoff flows from East to West to an outlet into the ocean near the intersection of Hill Street and Sunset Cliffs Boulevard.

3.1.4 Basin C

Basin C is located South of Basin B and consists of four subbasins. Basin C4 is the largest of the four subbasins and stretches South down Amiford Drive, West down Monaco Street, and includes a small open-space area Southeast of the intersection of Amiford Drive and Monaco Street. Runoff generally flows in a Northeast direction toward the intersection of Monaco Street and Cordova Street. To the Southeast is Basin C3, an area bounded to the West by Cordova Street, to the East by Amiford Drive, to the North by Monaco Street, and to the South by Carmelo Street. In this basin, runoff flows West down Algeciras Street and then North down Cordova Street until it eventually drains at the intersection of Cordova Street and Monaco Street. Basin C1 is the smallest of the subbasins and is found between the Pacific Ocean to the West and Cordova Street to the East, and between Hill Street to the North and Monaco Street to the South. Runoff flows directly West down Monaco Street to a point in the ocean near the intersection of Sunset Cliffs Boulevard and Monaco Street. The rectangular-shaped Basin C2 is located directly South of Basin C1. Basin C2 is bordered by the Pacific Ocean to the West, Cordova Street to the East, and stretches North from Carmelo Street to Brindisi Street. Runoff flows approximately North down Sunset Cliffs Boulevard toward Basin C1.

3.1.5 Basin D

Basin D is composed of two subbasins, Basin D1 and Basin D2. The larger of the two, Basin D2, is located East of Basin D1 and is bounded to the East by Amiford Drive and to the Southwest by Lomaland Drive. Runoff flows West and then Southwest to a large storm drain cleanout located in a depressed lot near the corner of Amiford Drive and Stafford Place. The flow is routed via a storm drain culvert under Basin E and discharged to a shared driveway fronting Cornish Drive. Basin D1 is bounded to the West by the Pacific Ocean, to the South by Casitas Street, to the North by Carmelo Street, and to the East by Amiford Drive. Runoff drains West toward a point in the ocean near the corner of Carmelo Street and Sunset Cliffs Boulevard.

3.1.6 Basin E

Basin E stretches from Stafford Place in the East to the Pacific Ocean in the West; it is bounded to the North by Casitas Street and to the South by Ladera Street. Drainage runoff generally flows from East to West. Runoff travels down Casitas Street to Ladera Street, and eventually drains to the ocean at the corner of Ladera Street and Sunset Cliffs Boulevard.

Figure 1. Drainage Basins along the Linear Park

The **Table 2** summarizes the physical data for each of the basins in the linear park. The travel time (Tt) for each rainfall frequency differs due to the increase in gutter flow velocity as the rate of flow and depth of flow increases. Therefore, the travel time reduces as the velocity increases. The travel time was determined with the aid of Figure 2-2 of the San Diego County Drainage Design Manual. The process was iterative until the changes of velocity, based on the fixed slope and the previously calculated flow rate effectively stop changing the calculated flow rate. The number following Tt- refers to the storm event frequency.

Table 2

Basin	Area Acres	Elev I ft	End Elev ft	Soil Grp	C	Tt-1 min	Tt-5 min	Tt-10 min	Tt-50 min
X1	3.70	36.7	25.0	B	0.45	13.61	13.43	13.37	13.09
X2	2.25	34.2	28.5	B	0.45	13.71	13.71	13.71	13.44
X3	2.60	72.0	36.0	B	0.45	9.90	9.77	9.71	9.65
X4	5.72	42.5	32.0	C	0.48	21.08	18.25	17.54	17.31
X5	7.78	50.0	32.0	C	0.48	13.75	13.50	13.36	12.95
X6	4.60	72.0	39.0	C	0.48	7.64	7.46	7.38	7.26
X7	18.10	262.0	40.0	C	0.48	11.36	11.07	10.81	10.57
X8	37.15	300.0	36.0	C	0.48	16.79	16.49	16.20	15.51
X9	10.07	280.0	74.0	C	0.48	14.69	14.25	14.09	13.86
A	54.56	258.0	40.0	B&C	0.47	14.22	13.84	13.60	13.27
B1	6.48	65.0	46.0	B	0.45	19.57	19.42	19.15	19.02
B2	12.02	186.0	65.0	B	0.45	15.50	15.32	15.23	15.15
B3	47.56	314.0	90.0	C	0.48	20.81	20.24	20.11	19.62
B4	13.46	315.0	98.0	C	0.48	20.30	20.06	19.84	19.63
B5	3.09	250.0	228.0	B	0.45	12.15	11.98	11.90	11.68
B6	20.44	311.0	180.0	C	0.48	16.67	16.26	16.07	15.69
C1	6.22	80.0	42.0	B	0.45	17.60	17.29	17.14	17.07
C2	3.34	82.0	46.0	B	0.45	17.85	17.85	17.82	17.78
C3	17.52	205.5	70.5	B&C	0.47	15.32	15.25	14.99	14.71
C4	18.62	313.0	76.0	C	0.48	15.72	15.52	15.44	15.33
D1	10.44	207.5	59.0	B&C	0.47	14.98	14.84	14.72	14.56
D2	17.80	331.0	206.0	C	0.48	12.43	12.18	12.06	12.05
E	12.91	285.0	65.0	B&C	0.47	15.55	15.43	15.38	15.27

3.2 HILLSIDE PARK BASIN DATA

The runoff from drainage tributaries which impacts the Hillside Park section also originates along the ridgeline of the Point Loma Peninsula. The terrain is steep East to West and relatively flat in the North – South direction. The hillside park can be identified as being fully developed to the East of the Lomaland Drive/Western Loop road and minimally developed to the West of the road. Currently, a significant portion of the runoff entering the Hillside Park originates within the Point Loma Nazarene University (University) grounds. The University has installed an extensive storm drain system upstream of the park, especially within the athletic fields and some parking areas. The flows captured within the fields are discharged at several locations upstream of the Hillside Park with the intention of routing the flow to the 24-inch concrete pipe located beneath the existing Arizona crossing. In general, the basins located to Northeast and Southeast of the Arizona crossing drain toward the crossing along the roadway.

The Hillside Park has a 95 acre upstream tributary area and shall be delineated to thirteen basins. The area could be delineated into several dozen basins, however since they all converge within several major points of discharge, more basins will not yield more useful data. Two of these basins are further separated to smaller sub-basins to facilitate drainage boundaries and differing slope characteristics. The following is a general description of each basin and **Table 3** summarizes the basin data.

3.2.1 Basin F

Basin F is located directly South of Ladera Street and is the Northernmost part of the Hillside Park. Drainage runoff flows West through the center of the basin toward the Pacific Ocean, from its Eastern corner to a point on the coast approximately 360 feet South-southeast from the corner of Ladera Street and Sunset Cliffs Boulevard.

3.2.2 Basin G

Basin G is found South of Basin F, Basin E (Linear Park), and Basin D2. Drainage flows Southwest through the center of the basin from Lomaland Drive in the East to the Pacific Ocean in the West. The runoff is discharged at a point on the coast approximately 515 feet South-southeast from the corner of Ladera Street and Sunset Cliffs Boulevard.

3.2.3 Basin H

The smallest drainage basin of the entire system, Basin H, is located South of the Westernmost portion of Basin G. The small basin is roughly shaped like a triangle, and is bounded to the West by the Pacific Ocean and to the East by lower parking lot. Runoff travels West through the center of the basin and eventually drains at a point on the coast approximately 640 feet South-southeast from the corner of Ladera Street and Sunset Cliffs Boulevard.

3.2.4 Basin I

Basin I, another of the smaller drainage basins, is situated directly South of Basin G. The runoff flows through the center of the drainage area, from the eastern corner of the basin to a point on the coast approximately 760 feet South-southeast from the corner of Ladera Street and Sunset Cliffs Boulevard.

Figure 2. Drainage Basins along the Hillside Park

3.2.5 Basin J

Basin J is located South of, and adjacent to, Basin I. Basin J is larger than Basin I, although shaped similarly. It is bounded at its East by Lomaland Drive. The drainage line runs Southwest through the center of the basin, from the eastern corner to a point on the coast approximately 925 feet South-southeast from the corner of Ladera Street and Sunset Cliffs Boulevard.

3.2.6 Basin K

Basin K is located South of, and adjacent to, Basin J. It is bounded on the East by Lomaland Drive/Western Loop Road. Drainage runoff travels Southwest to its discharge point on the coast approximately 1,075 feet South-southeast from the corner of Ladera Street and Sunset Cliffs Boulevard.

3.2.7 Basin L

Basin L is located South of, and adjacent to, Basin K. It is bordered by Western Loop Road to the East, and the Pacific Ocean to the West. Runoff flows Southwest along the Northern portion of the basin, from the Northeast corner of the basin to a point on the coast approximately 1,235 feet South-southeast from the corner of Ladera Street and Sunset Cliffs Boulevard.

3.2.8 Basin M

Similar to the drainage basins of the Linear Park, Basin M consists of two subbasins. Basin M1 is the second smallest drainage basin in the entire drainage system. Runoff flows Northwest, down a ravine between two steep cliffs, toward the Pacific Ocean. Drainage discharges at a point on the coast approximately 1,300 feet South-southeast from the corner of Ladera Street and Sunset Cliffs Boulevard. In contrast, Basin M2 is one of the largest drainage basins in the drainage system. Basin M2 is bounded primarily by Lomaland Drive/ Western Loop Road to the West and Pepper Tree Lane to the East. Runoff flows along the Northern basin boundary and along the Southern portion of the basin, along Lomaland Drive South of Point Loma Nazarene College. Runoff discharges at a point in the Southwest corner of the drainage basin, near the entrance to a baseball field parking lot. The runoff flows into the Basin M1 via a small diameter (24-inch) pipe and an Arizona crossing.

3.2.9 Basin N

Basin N is located South of Basin M1, and is bounded to the West by cliffs overlooking the Pacific Ocean. The majority of this basin is designated as a baseball field. Drainage runoff flows along the Southern portion of the basin, to where it drains at a point on the coast approximately 1,685 feet South from the corner of Ladera Street and Sunset Cliffs Boulevard.

3.2.10 Basin O

Basin O is a rectangle-shaped drainage basin located South of Basin N. It is bounded to the West by cliffs overlooking the Pacific Ocean and to the East by Lomaland Drive/ Western Loop Road. Drainage runoff flows West through the center of the basin and drains at a point on the coast approximately 1,950 feet South-southeast from the corner of Ladera Street and Sunset Cliffs Boulevard.

3.2.11 Basin P

Basin P is situated South of Basin O. Like Basin O, it is bounded by cliffs overlooking the Pacific Ocean to the West and by Lomaland Drive to the East. A parking lot consumes a large portion of the basin. Runoff flows from the Northeast corner of the basin, over the parking lot structure, and then Northwest to a point on the coast approximately 2,125 feet South-southeast from the corner of Ladera Street and Sunset Cliffs Boulevard.

3.2.12 Basin Q

Basin Q is located South of the Southwest corner of Basin M2. Lomaland Drive forms the Western and Southern boundaries of the basin. Runoff flows toward the West, and primarily travels down Lomaland Drive. Drainage discharges at a point in the Northwest corner of the drainage basin, near the entrance to a baseball field parking lot.

3.2.13 Basin R

Basin R consists of two subbasins; Basin R1 and Basin R2. Basin R2 is the larger drainage basin of the two. Runoff flows Southwest, following a steep hill from the Northeast corner of the basin toward the Pacific Ocean. The runoff drains directly into Basin R1 at a point near the Southernmost loop of Lomaland Drive. Basin R1 is bounded by the Pacific Ocean to the West. Runoff flows Northwest along the Southern portion of the basin, along a steep hill. Runoff discharges at a point on the coast approximately 2,240 feet South-southeast from the corner of Ladera Street and Sunset Cliffs Boulevard.

Table 3 summarizes the physical data for each of the basins in the Hillside Park. The travel time for each rainfall frequency remains the same due to the terrain being primarily natural and none to minimal gutter flow.

Table 3

Basin	Area Acres	High Elev ft	Low Elev ft	Soils Group	C	Tt min
F	3.68	180.0	46.0	B	0.29	22.22
G	6.65	317.0	50.0	B&C	0.31	37.32
H	0.71	96.0	50.0	B	0.29	12.35
I	2.02	280.0	78.0	B&C	0.31	25.83
J	6.77	310.5	48.0	B&C	0.31	29.4
K	3.56	229.0	48.0	B	0.25	24.56
L	4.16	195.5	50.0	B	0.25	24.16
M1	1.79	128.0	38.0	B	0.25	23.57
M2	38.86	347.0	134.0	B&C	0.47	9.065
N	2.53	123.0	26.0	B	0.32	20.38
O	3.96	144.0	25.0	B	0.32	21.69
P	3.23	140.0	32.0	B	0.32	27.48
Q	4.87	284.5	144.0	B&C	0.34	8.975
R1	5.72	179.0	16.0	B	0.25	25.11
R2	6.66	353.0	179.0	C	0.29	24.67

3.3 RUNOFF DATA

Storm water runoff data was calculated for each of the drainage basins using the Rational Method. Modified rational method was used for basins with several sub-basins. It was assumed that existing curb inlets functioned as designed and inflow volume rate and shall be subtracted from the calculated rate during the hydraulic analysis as bypass rate to the down stream junction. The travel time determined for each of the developed basins reflects the assumption that the initial stream/gutter flow development will take place within each lot and assumed a one (1) percent slope around the buildings. The distance for the initial time of concentration was measured from the topography, (for accuracy), and the travel time after the initial time of concentration was based on the gutter flow charts provided in the Design Manual.

The runoff rates determined by this method were much greater than by using the overland flow method used for natural basins, however, the use here is justifiable since paved surfaces have less resistance and therefore transport runoff at much higher velocities, The greater velocities reduce travel time and leads to quicker peak times and higher flows. **Table 4** summarizes the developed data.

Table 4

Basin	1 Year Storm		5 Year Storm		10 Year Storm		50 Year Storm	
	I in/hr	Q cfs	I in/hr	Q cfs	I in/hr	Q cfs	I in/hr	Q cfs
X1	1.38	2.30	1.81	3.02	2.10	3.49	2.69	4.48
X2	1.37	1.39	1.79	1.81	2.06	2.09	2.65	2.68
X3	1.70	1.98	2.22	2.60	2.58	3.01	3.27	3.83
X4	1.04	2.86	1.49	4.08	1.76	4.83	2.25	6.17
X5	1.37	5.12	1.80	6.74	2.10	7.83	2.71	10.12
X6	2.00	4.43	2.65	5.85	3.07	6.79	3.93	8.69
X7	1.55	13.48	2.05	17.81	2.40	20.88	3.09	26.83
X8	1.21	21.51	1.59	28.29	1.85	33.01	2.41	43.01
X9	1.31	6.35	1.74	8.42	2.03	9.79	2.59	12.53
A	1.34	34.42	1.78	45.54	2.07	53.14	2.67	68.40
B1	1.09	3.18	1.43	4.16	1.66	4.84	2.11	6.16
B2	1.27	6.87	1.66	9.00	1.93	10.42	2.45	13.25
B3	1.05	23.97	1.39	31.73	1.61	36.76	2.07	47.32
B4	1.07	6.90	1.40	9.04	1.62	10.50	2.07	13.39
B5	1.49	2.07	1.95	2.71	2.26	3.14	2.90	4.03
B6	1.21	11.88	1.60	15.70	1.86	18.26	2.39	23.48
C1	1.17	3.27	1.54	4.31	1.79	5.00	2.27	6.34
C2	1.16	1.74	1.51	2.26	1.74	2.61	2.21	3.31
C3	1.28	10.54	1.67	13.74	1.95	16.03	2.50	20.55
C4	1.26	11.25	1.65	14.75	1.91	17.07	2.43	21.72
D1	1.30	6.37	1.70	8.33	1.97	9.67	2.51	12.33
D2	1.46	12.51	1.93	16.48	2.24	19.14	2.84	24.26
E	1.27	7.69	1.66	10.04	1.91	11.62	2.44	14.78
F	1.01	1.08	1.31	1.40	1.51	1.61	1.91	2.04
G	0.72	1.49	0.94	1.93	1.08	2.23	1.37	2.82
H	1.47	0.30	1.91	0.39	2.21	0.45	2.79	0.58
I	0.91	0.57	1.19	0.74	1.37	0.86	1.74	1.09
J	0.84	1.76	1.09	2.29	1.26	2.64	1.60	3.35
K	0.94	0.84	1.23	1.09	1.42	1.26	1.79	1.60
L	0.95	0.99	1.24	1.29	1.43	1.49	1.81	1.88
M1	0.97	0.43	1.26	0.56	1.45	0.65	1.84	0.82
M2	1.79	32.78	2.34	42.78	2.71	49.54	3.46	63.18
N	1.06	0.86	1.38	1.12	1.60	1.29	2.02	1.64
O	1.02	1.30	1.33	1.69	1.53	1.95	1.94	2.46
P	0.88	0.91	1.14	1.18	1.32	1.36	1.67	1.72
Q	1.81	2.99	2.37	3.93	2.75	4.56	3.50	5.79
R1	0.93	1.33	1.21	1.73	1.40	2.00	1.77	2.53
R2	0.94	1.82	1.22	2.36	1.41	2.73	1.79	3.45

4. HYDRAULIC ANALYSIS

The hydraulic analysis discussion within this section shall be limited to the surface runoff and related conveyance mechanisms. The primary flow conveyor for the majority of the basins is surface flow, either contained within the curbs of streets or along the historical or recently eroded streamlines. The analysis of the natural streams will be limited to identifying the erosion potential due to flow velocities. The analysis of the existing drainage facilities will determine the adequacy of each system, their limitations, and identify bypass flows if any exist. Several basins that have existing storm drain pipe networks were delineated into smaller subbasins with the points of convergence being at curb inlets, grated catch basins or a location where the discharge will be split into two or more downstream basins. An assumption was made that the existing drainage structures were constructed to meet the standard set forth in the San Diego Regional Standard Drawings Manual and that the pipes are capable of carrying the runoff captured by the curb inlets as designed. It was assumed that the carrying capacities of existing curbs reflects the performance curves defined in Figures 2-2 and Figure 2-3 of the Design Manual for six-inch and eight-inch curbs respectively. Unless otherwise noted, only curbs along Sunset Cliffs Boulevard are analyzed as eight-inch high curbs.

Certain scenarios were not analyzed, such as the effect of debris or objects located in the flow path within the gutter and the resultant routing of flow over to the sidewalk and bypassing the inlets where the flow was intended to go.

4.1 HYDRAULIC METHODOLOGY

The methodology used to perform hydraulic calculations conforms to the guidelines and equations provided within Chapter 2 of the Design Manual. The interception capacity of a curb inlet installed on a sloped street was calculated by using equation 2-2.

$$Q/L_T = 0.7(a+y)^{3/2}$$

Where
Q = interception capacity of the curb inlet, cubic feet per second;
y = depth of flow approaching the curb inlet (ft);
a = depth of depression of curb at inlet (ft); 4.0 inches standard.
L_T = length of clear opening of inlet for total interception (ft) or the actual opening in this case.

The interception capacity of a curb inlet installed on Sag was calculated by using equation 2-8.

$$Q = C_W L_W d^{(3/2)}$$

Where
Q = inlet capacity (ft^3/s);
C_W = weir discharge coefficient; 3.0 per Table 2-1 of Design Manual;
L_W = weir length (ft); and
d = flow depth (ft).

The interception capacity of a grated inlet installed on Sag was calculated by using equation 2-18 to calculate the capacity of the inlet installed on the downstream end of Basin D2.

$$Q = C_o A_e (2gd)^{1/2}$$

$$A_e = (1\text{-}C_A)A$$

Where

Q = inlet capacity of the grated inlet, cubic feet per second;
C_o = orifice coefficient (C_o=0.67 for U.S. Traditional Units);
g = gravitational acceleration (ft/s^2); 32.2 feet per second per second;
d = flow depth above inlet (ft);
A_e = effective (clogged) grate area square feet;
C_A = area clogging factor (C_A=0.50); and
A = actual opening area of the grate inlet; A=4.7 square feet; SDRSD No.D-15

The hydraulic analysis process starts off where the hydrology analysis ended when the flow rate for a given basin was iteratively determined. The following process was used to analyze each basin with developed conditions;

1. Using the previously determined individual basin runoff and the street slope, a depth of flow was estimated using the Figure 2-2 or Figure 2-3 of Design Manual. The longitudinal slope in the immediate vicinity of the inlet was used instead of the average basin slope.
2. Using the estimated depth of flow and the inlet physical data, the curb inlet capture capacity was calculated by using either equation 2-2 or 2-8.
3. The captured flow rate was subtracted from the calculated runoff rate to determine if a bypass will be added to the downstream basin.
4. Steps 1 through 3 were performed for every basin prior to analyzing run-on conditions. The purpose for this is to determine if the existing curb inlet is capable of handling the peak flow within the basin. In most cases T_c for an individual basin will be smaller than for a composition of basins in series and therefore, will have higher flow rates. If the inlet is capable of conveying the flows generated with the basin, bypass analysis is not needed.
5. Once it was determined that a basin receives run-on flows, a new runoff value calculation for the combined basins was performed using the method described on the San Diego County Hydrology Manual. The areas of all tributary basins were added to the basin of interest and a new T_c was calculated to determine new Intensity. New T_c was determined by adding the additional T_t needed to get the bypass flows from each of the upstream basin's discharge point to the discharge point of the receiving basin. The additional T_t was based on the gutter flow velocity determined for each upstream basin. The new T_c for the composite basin shall be the longest T_c calculated for each individual flow path including the receiving basin's T_c. A new flow rate is calculated using the combined basin size, the new T_c and the resultant intensity. The capture rates

determined earlier for each intercepting inlet within the composite basin were subtracted from the new runoff rate to determine if any will bypass to the next downstream basin.

6. It was assumed that the capture rate for combined basin is the same as the capture rate determined for the individual basin. This was done to simplify the calculations instead of having to determine a new runoff based on the depth of flow for combined area and the new T_c.
7. The process was repeated for 1-year, 5-year, 10-year and 50-year storm frequencies until ultimately the runoff exits the basins.

4.2 LINEAR PARK

The tributaries analyzed under this section primarily consist of lands external to the actual linear park. However, the potential exists that the runoff generated in these areas can cause a great deal of erosion within the park if not properly managed. The analysis shall be performed for all the basins to determine the effects of the existing improvements and identify shortcomings. For each improved basin, a discharge point and characteristics shall be identified, findings from the calculation shall be stated and the potential bypass route shall be identified. See **Tables 5** through **8** for a summary of data calculated. For each unimproved basin, the calculated flow velocity shall be declared and the potential to cause erosion shall be discussed.

4.2.1 Basin X

Basin X is composed of nine subbasins. Subbasin X9 is the uppermost basin. The runoff generated within the basin converges at a 20-feet long curb inlet located at the Southeast corner of the intersection of Point Loma Avenue and Froude Street. The analysis indicates that the inlet is capable of intercepting the runoff for the 1-year storm but not the other storms. The excess runoff and bypass flows due to blockages are routed to Basins X3 and X6. The analysis of the contours indicated that a major part of the bypass runoff will flow to X6, while visual observations indicated that the runoff will be equally split between the two downstream basins. Therefore the flow is assumed to split equally between Basins X3 and X6.

Basin X8 converges at two 15-foot long curb inlets located on either side of Tivoli Street just East of Devonshire Drive. The analysis indicates that the existing facilities are incapable of fully capturing the runoff from any of the storm events analyzed. In the case of a blockage and excess flows, the runoff will bypass to Basin X5.

Basin X7 also converges at two 14-foot long curb inlets located on either side of Grainger Street just East of Devonshire Drive. The analysis indicates that the existing facilities are incapable of fully capturing the runoff from any of the storm events analyzed. In the case of a blockage and excess flows, the runoff will again bypass to Basin X5.

Basin X6 converges at a 15-foot long curb inlet located on the South side of Adair Street just East of Ebers Street. The basin receives bypass flows from X9 and the analysis indicates that the existing facilities are incapable of fully capturing the runoff from any of the storm events analyzed. In the case of a blockage and excess flows, the runoff will again bypass to Basin X3.

Basin X5 converges at a 5-foot long curb inlet located on the South side of Adair Street just West of Devonshire Drive. The basin receives bypass flows from Basins X7 and X8, and the analysis indicates that the existing facilities are incapable of fully capturing the runoff from any of the storm events analyzed. In the case of a blockage and excess flows, the runoff will bypass to Basin X2.

Basin X4 converges at a 15-foot long curb inlet located on the East side of Sunset Cliffs Boulevard just South of Adair Street. The analysis indicates that the inlets are capable of intercepting the runoff for the 1-year storm and 5-year storm but not the other storms. However, the depth of flow for the analyzed storms ranged from four-inches to nearly eight-inches. In the case of a blockage and excess flows, the runoff will bypass to Basin X2.

Basin X3 converges at a 15-foot long curb inlet located on the North side of Adair Street just West of Ebers Street. The analysis indicates that the inlets are capable of intercepting the runoff from all but the 50-year storm. In the case of a blockage and excess flows, the runoff will bypass to Basin X2.

Basin X2 converges at a 20-foot long curb inlet located on the East side of Sunset Cliffs Boulevard just South of Point Loma Avenue. The analysis indicates that the inlets are not capable of intercepting the runoff from any of the storms. Furthermore, the depth of flow for the 10-year or 50-year storm was over six-inches and the intersection will be flooded. In the case of a blockage and excess flows, the runoff will again bypass to Basin X1. Once the intersection is flooded the inlet capacity will nearly triple since the inlet will start to function at Sag condition.

Basin X1 converges at a 20-foot long curb inlet located on the Southside of Point Loma Avenue just East of Sunset Cliffs Boulevard. The analysis indicates that the inlets are capable of intercepting the runoff for the storms with excessive flooding. The depth of flow for the 50-year storm was nearly 14 inches.

4.2.2 Basin A

The runoff from Basin A discharges through an existing 14-foot curb inlet operating as a weir at the West side of the intersection of Sunset Cliffs Boulevard and Osprey Street. The inlet is located at a local sag created by a cross gutter spanning from East to West. Sunset Cliffs Boulevard itself slopes down towards the North at a near flat 0.4 percent slope. The analysis indicates that the existing facilities are incapable of capturing the runoff from any of the storm events analyzed. The inlet area will be inundated during all the storms with a flow depth of over eight-inches. The excess flow will be both diverted to a down stream basin and flow over the curb into the Linear Park. The large basin size and steep slopes contribute to large flow rates that approach the inlet. In the case of a blockage and excess flows, the runoff will bypass to Basin X4.

4.2.3 Basin B

Basin B is composed of six subbasins. Subbasin B6 consists of the area upstream of the Sunset View Elementary School and the runoff is routed to two 14-foot long curb inlets located near the school on Hill Street, which has an average street slope of 6.4-percent, and to another curb inlet located approximately 300 feet down stream. The analysis indicated that the inlets are not capable of capturing the entire flows from any of the storm frequencies analyzed. The excess runoff and bypass flows due to blockages are routed to Basin B4. The depth of flow remained below the curb height.

Basin B5 consists of the area within the Sunset View Elementary School and the runoff is routed to a grated inlet catch basin enclosed on three sides with an approximately six inch high berm. The analysis indicated that the grated inlet is capable of conveying the runoffs from all the storm frequencies analyzed. In the case of a blockage of the grate, the flow will rise above the berm and flow onto Basin B6.

Basin B4 runoff discharges to two 14-foot long curb inlets located along Hill Street, immediately East of the intersection of Novara Street. The basin receives bypass flows from Basin B6 and possibly from B5. The analysis indicates that the two inlets are not capable of intercepting the entire runoff converging at the inlets. The excess runoff and bypass flows due to blockages are routed to the lower portion of Basin B3.

The runoff from Basin B3 discharges to three 20-foot long curb inlets located on the East side Novara Street and North of Hill Street, and a single 10-foot long curb inlet located on the Northwest corner of Hill Street at the intersection with Devonshire Drive. This basin receives bypass flows from Basins B4 and B6. The analysis indicates that the inlets are capable of intercepting the runoff for the 1-year storm but not the other storms. The excess runoff and bypass flows due to blockages are routed to the lower portion of Basin B2.

It should be noted that the three curb inlets are not located at ideal locations to capture the runoff from the basin. All three are installed on the East side of the Novara Street in a linear sequence. Novara Street does not have a well defined crown and the runoff coming down along Piedmont Drive during any significant storm event will have adequate kinetic energy to cross the Novara Street well upstream of the three curb inlets. At least one, if not two, of the curb inlets should have been placed on the West side of Novara Street in order to capture the flows which are most likely to flow along the West curb. This is a qualitative judgment based on contour data and visual analysis made during dry weather visits and, therefore, require additional in depth analysis of the Basin B3.

Basin B2 runoff discharges to a nine-foot long curb inlet located at the Southwest corner of the intersection of Hill Street with Cordova Street. The basin receives bypass from Basins B3, B4 and B6. Again, the analysis indicates that the inlet is capable of intercepting the runoff for the 1-year storm but not the other storms. The excess runoff and bypass flows due to blockages are routed to the lower portion of Basin B1.

The runoff from Basin B1 converges at a 17-feet curb inlet located on the West side of Sunset Cliffs Boulevard at the intersection of Hill Street. The basin receives bypass flows from all the subbasins. The analysis indicates that the curb inlet with eight-inch curb height and located at a sag point, does not have the capacity to convey the entire flow for all the storm frequencies analyzed even with flooding during the 50-year storm. The excessive runoff will overtop the curb near the inlet and cause erosion within the linear park. In the case of a blockage, the runoff will rise over the curb and flow over the cliff. The flow depths were well below curb height for the entire Basin for other storm events.

4.2.4 Basin C

Basin C is composed of four subbasins. Subbasin C4 is the largest and the runoff from the basin converges at a 14-foot long curb inlet located on the South side of Monaco Street North of Cordova Street. The runoff is conveyed between the curbs of the street and the analysis indicates the curb inlet is not capable of capturing the entire flows from any of the storm frequencies analyzed. The excess runoff and bypass flows due to blockages are routed to Basin C1. The depth of flow remained below the curb height.

Basin C3 runoff converges at a 14-foot long curb inlet located on the West side of Cordova Street South of Monaco Street. The runoff is conveyed between the curbs of the street and the analysis indicates the curb inlet is not capable of capturing the entire flows from any of the storm frequencies analyzed. The excess runoff and bypass flows due to blockages are routed to Basin C1. The depth of flow remained below the curb height but the 50-year runoff flow depth near the inlet, with a local slope of less than one-percent, was estimated to be 0.48 feet.

Basin C2 runs along the Sunset Cliffs Boulevard, flowing North, starting North of Carmelo Street, for approximately 640-feet. It receives bypass flows from Basin D1. The ultimate intended discharge point for the basin is a concrete spillway which directs flows over the cliff onto an existing gabion slope protection device. However, most of the flow will never reach the spillway. The West side curb along this section of Sunset Cliffs Boulevard is not built to a standard height of six or eight inches, instead the height is about three to four inches. The road itself has either a flat cross slope or a slight slant toward the West. As a result, the carrying capacity of this section of the road is minimal. For the purpose of analysis, a three inch height limitation was used to determine the curb capacity. The analysis indicates that the capacity of the road in the vicinity is approximately 1.9 cfs for each curb. The calculated flow for the basin and the run-on flow into the basin, range from seven cfs to 22 cfs for different storm frequencies. Only approximately four cfs will reach the concrete spillway and the remainder will flow over the berm and cliff along the length of the basin. The spillway has a flow capacity of approximately 5.3 cfs and is capable of discharging the possible four cfs of runoff. Based on the available contour data, the flows flowing along the East side of the road will cross the street to the West side before reaching the spillway. Therefore no bypass flows to Basin C1 are expected.

Basin C1 runoff converges at a 12-feet curb inlet on the West side of Sunset Cliffs Boulevard at the intersection of Monaco Street. The basin receives bypass flows from Basins C3, C4 and possibly C2. However, the bypass flow from C2 is expected to be negligible and will not be considered. The analysis indicates that the curb inlet with eight inch curb height and located at a sag point, does have the capacity to convey the entire flow for all the storm frequencies analyzed. However, the 50-year storm runoff requires nearly eight inches of head in order to convey the flow through the opening. It is most likely that the 100-year storm will overtop the curb and flow directly to the sewer pump station. Since the land to the West of the curb inlet is higher than the road and slopes upward, there will be adequate head to convey a greater amount of flow. However, eventually it is possible for the runoff to find its way over the curb to the Ocean via the large hole located North of the inlet. In the case of a blockage, the runoff will rise over the curb and flow into the sewer pump station and to the ocean via the hole located to the North.

4.2.5 Basin D

Basin D is composed of two subbasins. Subbasin D2 conveys runoff via brow ditches, sheet flow and open channels to a grated inlet located in a local depression with approximately five-feet of head. The analysis indicates that the runoff will pond to an approximate height of 3.7 feet during the 50-year storm and the inlet is capable of conveying the entire flow to a 24-inch pipe. In the case of a blockage of the grate, the flow will rise above the available five feet of depth and flow on to the intersection of Amiford Drive and Stafford Place and into Basin E.

The flow conveyed through the pipe exits the pipe through a end/retaining wall located between two private homes. The runoff flows down the driveway and enters Basin D1 via Cornish Drive. The runoff continues towards the West along the alley between Carmelo Street and Casitas Street, and merges with the runoff flowing along Cordova Street. The runoff exits the basin via a 12-foot curb inlet located on the West side of Sunset Cliffs Boulevard at the intersection of Carmelo Street. The analysis indicates that the curb inlet, located along a 2.1-percent street slope, does not have the capacity to convey the entire flow for all the storm frequencies analyzed. The excess flows are bypassed to Basin C2. In the case of a blockage, additional runoff shall be routed to Basin C2.

4.2.6 Basin E

Basin E runoff discharges through an existing 14-foot curb inlet and a grated inlet operating as a weir at the West side of the intersection of Sunset Cliffs Boulevard and Ladera Street. The inlets are located at a sag created by the intersecting curbs on the South and West sides, and the analysis indicates that the existing facilities are capable of capturing the approximately four inch deep runoff flow approaching the inlets during the 50-year storm event. The West curb is approximately eight-inches in the vicinity and extends North for approximately 30-feet. In the case of a blockage, flow will rise over the curb and/or flow around the North end and flow over the edge of the cliff face.

Table 5

Basin Inlet	Drainage Zone	Area ACRE	Tc MIN	Qt CFS	Qb CFS	Qi CFS	Icap CFS	D FT	Qby CFS
X1	X1 thru X9 + A	146.53	30.14	57.50	1.45	8.62	19.27	0.22	0.00
X2	X2 thru X9+A	142.83	29.00	57.54	0.91	16.87	9.71	0.45	7.16
X3	X3+X6+X9	17.27	19.13	9.10	1.37	1.37	3.98	0.19	0.00
X4	X4+A	60.29	26.17	25.72	2.44	2.44	6.33	0.38	0.00
X5	X5+X7+X8	63.02	18.08	34.79	4.29	18.21	2.24	0.41	15.96
X6	X6+X9/2	9.64	18.20	5.30	2.53	2.53	4.68	0.25	0.00
X7	X7	18.10	11.36	13.48	13.48	13.48	9.76	0.30	3.72
X8	X8	37.15	16.79	21.51	21.51	21.51	11.32	0.36	10.20
X9	X9	10.07	14.69	6.35	6.35	6.35	6.81	0.29	0.00
A	A	54.56	14.22	34.42	34.42	34.42	31.10	0.67	3.32
B1	B1-4+B6	99.95	25.38	43.82	2.84	5.95	18.30	0.26	0.00
B2	B2-4+B6	93.47	23.31	43.44	5.59	5.92	2.81	0.25	3.11
B3	B3+B4+B6	81.45	22.25	39.33	22.96	25.63	25.30	0.38	0.33
B4	B4+B6	33.90	22.00	16.49	6.55	8.67	6.00	0.24	2.67
B5	B5	3.09	12.15	2.07	2.07	2.07	2.85	0.22	0.00
B6	B6	20.44	16.67	11.88	11.88	11.88	9.76	0.30	2.12
C1	C1+C3+C4	42.36	23.38	19.46	2.86	14.20	14.23	0.29	0.00
C2	C2+D1+D2	31.58	17.74	17.41	1.84	2.78	5.30	0.25	0.00
C3	C3	17.52	15.32	10.54	10.54	10.54	5.90	0.38	4.64
C4	C4	18.62	15.72	11.25	11.25	11.25	4.54	0.27	6.71
D1	D1+D2	28.25	15.90	16.81	6.22	6.22	5.28	0.40	0.94
D2	D2	17.80	12.43	12.51	12.51	12.51	12.51	0.98	0.00
E	E	12.91	15.55	7.69	7.69	7.69	14.40	0.24	0.00

Qt = The total accumulative basin runoff
Qb = Basin runoff based on the new Tc
Qi = Runoff Rate at the inlet
Qby = Bypass Runoff at the inlet
Icap=Inlet Capacity
D =Depth of flow at the inlet.

Table 6: 5-Year Storm

Basin Inlet	Drainage Zone	Area ACRE	Tc MIN	Qt CFS	Qb CFS	Qi CFS	Icap CFS	Qby MIN	D FT
X1	X1 thru X9 + A	146.53	26.59	81.06	2.05	18.00	27.48	0.00	0.35
X2	X2 thru X9+A	142.83	25.45	81.38	1.28	27.97	12.02	15.95	0.57
X3	X3+X6+X9	17.27	18.57	12.07	1.81	2.33	4.26	0.00	0.22
X4	X4+A	60.29	23.22	36.12	3.43	3.43	8.42	0.00	0.53
X5	X5+X7+X8	63.02	17.74	45.78	5.65	29.35	2.66	26.69	0.50
X6	X6+X9/2	9.64	17.58	7.04	3.36	3.88	4.80	0.00	0.26
X7	X7	18.10	11.07	17.81	17.81	17.81	10.35	7.46	0.32
X8	X8	37.15	16.49	28.29	28.29	28.29	12.06	16.23	0.39
X9	X9	10.07	14.25	8.42	8.42	8.42	7.39	1.03	0.32
A	A	54.56	13.84	45.54	45.54	45.54	31.10	14.44	0.67
B1	B1-4+B6	99.95	19.42	57.74	3.74	19.38	19.42	0.00	0.28
B2	B2-4+B6	93.47	15.32	57.24	7.36	18.59	2.95	15.64	0.27
B3	B3+B4+B6	81.45	21.82	51.77	30.23	37.88	26.65	11.23	0.41
B4	B4+B6	33.90	21.58	21.70	8.62	13.97	6.32	7.65	0.26
B5	B5	3.09	11.98	2.71	2.71	2.71	3.25	0.00	0.24
B6	B6	20.44	16.26	15.70	15.70	15.70	10.35	5.35	0.32
C1	C1+C3+C4	42.36	22.89	25.64	3.76	21.15	21.21	0.00	0.49
C2	C2+D1+D2	31.58	17.35	22.96	2.42	5.02	5.30	0.00	0.25
C3	C3	17.52	15.25	13.74	13.74	13.74	6.28	7.46	0.41
C4	C4	18.62	15.52	14.75	14.75	14.75	4.82	9.92	0.29
D1	D1+D2	28.25	15.55	22.16	8.19	8.20	5.60	2.60	0.43
D2	D2	17.80	12.18	16.48	16.48	16.48	16.48	0.00	1.70
E	E	12.91	15.43	10.04	10.04	10.04	15.07	0.00	0.26

Qt = The total accumulative basin runoff
Qb = Basin runoff based on the new Tc
Qi = Runoff Rate at the inlet
Qby = Bypass Runoff at the inlet
Icap=Inlet Capacity
D =Depth of flow at the inlet.

Table 7: 10-Year Storm

Basin Inlet	Drainage Zone	Area ACRE	Tc MIN	Qt CFS	Qb CFS	Qi CFS	Icap CFS	Qby CFS	D FT
X1	X1 thru X9 + A	146.53	25.56	95.94	2.42	36.16	36.21	0.00	0.51
X2	X2 thru X9+A	142.83	24.42	96.44	1.52	46.98	13.24	33.74	0.63
X3	X3+X6+X9	17.27	18.35	14.03	2.11	3.27	4.32	0.00	0.22
X4	X4+A	60.29	22.34	42.72	4.06	20.58	9.32	11.27	0.59
X5	X5+X7+X8	63.02	17.42	53.44	6.60	36.85	2.66	34.19	0.50
X6	X6+X9/2	9.64	17.34	8.20	3.92	5.03	4.98	0.05	0.28
X7	X7	18.10	10.81	20.88	20.88	20.88	11.07	9.81	0.35
X8	X8	37.15	16.20	33.01	33.01	33.01	12.56	20.45	0.41
X9	X9	10.07	14.09	9.79	9.79	9.79	7.56	2.23	0.33
A	A	54.56	13.60	53.14	53.14	53.14	31.10	22.04	0.67
B1	B1-4+B6	99.95	19.15	67.14	4.35	28.24	28.25	0.00	0.46
B2	B2-4+B6	93.47	15.23	66.47	8.55	26.88	2.99	23.89	0.28
B3	B3+B4+B6	81.45	21.62	60.09	35.08	46.07	27.74	18.33	0.43
B4	B4+B6	33.90	21.39	25.18	10.00	17.55	6.56	10.99	0.27
B5	B5	3.09	11.90	3.14	3.14	3.14	3.55	0.00	0.26
B6	B6	20.44	16.07	18.26	18.26	18.26	10.71	7.55	0.34
C1	C1+C3+C4	42.36	22.16	30.21	4.43	26.05	26.06	0.00	0.65
C2	C2+D1+D2	31.58	17.10	26.74	2.82	6.43	6.53	0.00	0.33
C3	C3	17.52	14.99	16.03	16.03	16.03	6.54	9.49	0.43
C4	C4	18.62	15.44	17.07	17.07	17.07	4.94	12.13	0.30
D1	D1+D2	28.25	15.35	25.79	9.54	9.54	5.94	3.60	0.46
D2	D2	17.80	12.06	19.14	19.14	19.14	19.14	0.00	2.30
E	E	12.91	15.38	11.62	11.62	11.62	15.51	0.00	0.27

Qt = The total accumulative basin runoff
Qb = Basin runoff based on the new Tc
Qi = Runoff Rate at the inlet
Qby = Bypass Runoff at the inlet
Icap=Inlet Capacity
D =Depth of flow at the inlet.

Table 8: 50-Year Storm

Basin Inlet	Drainage Zone	Area ACRE	Tc MIN	Qt CFS	Qb CFS	Qi CFS	Icap CFS	Qby CFS	D FT
X1	X1 thru X9 + A	146.53	24.91	123.55	3.12	66.68	67.06	0.00	1.16
X2	X2 thru X9+A	142.83	23.83	124.10	1.96	77.63	14.07	63.55	0.67
X3	X3+X6+X9	17.27	18.07	17.95	2.70	6.94	4.50	2.44	0.24
X4	X4+A	60.29	21.80	54.98	5.22	33.19	10.87	22.32	0.69
X5	X5+X7+X8	63.02	16.64	69.74	8.61	53.57	2.66	50.91	0.50
X6	X6+X9/2	9.64	17.00	10.52	5.02	7.25	5.23	2.02	0.30
X7	X7	18.10	10.57	26.83	26.83	26.83	11.81	15.03	0.38
X8	X8	37.15	15.51	43.01	43.01	43.01	13.07	29.94	0.43
X9	X9	10.07	13.86	12.53	12.53	12.53	8.08	4.45	0.36
A	A	54.56	13.27	68.40	68.40	68.40	31.10	37.29	0.67
B1	B1-4+B6	99.95	19.02	86.07	5.58	46.12	37.77	8.35	0.67
B2	B2-4+B6	93.47	15.15	85.20	10.96	43.72	3.18	40.55	0.30
B3	B3+B4+B6	81.45	21.24	77.01	44.96	62.72	29.96	32.76	0.47
B4	B4+B6	33.90	21.02	32.26	12.81	24.49	6.72	17.76	0.28
B5	B5	3.09	11.68	4.03	4.03	4.03	4.54	0.00	0.30
B6	B6	20.44	15.69	23.48	23.48	23.48	11.81	11.68	0.38
C1	C1+C3+C4	42.36	22.25	38.17	5.60	35.39	26.66	8.73	0.67
C2	C2+D1+D2	31.58	16.93	34.09	3.60	9.35	9.36	0.00	0.53
C3	C3	17.52	14.71	20.55	20.55	20.55	7.19	13.36	0.48
C4	C4	18.62	15.33	21.72	21.72	21.72	5.29	16.43	0.33
D1	D1+D2	28.25	15.23	32.83	12.14	12.14	6.39	5.75	0.50
D2	D2	17.80	12.05	24.26	24.26	24.26	24.26	0.00	3.69
E	E	12.91	15.27	14.78	14.78	14.78	16.60	0.00	0.29

Qt = The total accumulative basin runoff
Qb = Basin runoff based on the new Tc
Qi = Runoff Rate at the inlet
Qby = Bypass Runoff at the inlet
Icap=Inlet Capacity
D =Depth of flow at the inlet.

4.3 Hillside Park

The Hillside Park hydraulic analysis for natural basins shall be limited to the discussion of the flow velocities and the possibilities of excessive erosion. The surface soil within the Hillside Park is easily dislodged from the ground and at many locations does not have the protection of plant cover and root support. Human and burrowing animal activities combined with natural elements create a situation conducive to for surface erosion which can be easily seen. See **Table 9** for a summary of calculated data.

4.3.1 Basin F

Basin F discharges to the ocean over the cliff South of Ladera Street. Since the basin is small and in general does not receive runoff from upstream, the runoff volumes do not converge to a single streamline and do not have high flow velocities. As a result there is no predominantly defined/eroded streamline. However, the land is exposed to the natural elements; the soil cohesion is weak and is subject to erosion due to rain fall impact.

4.3.2 Basin G

Basin G begins within the University and discharges to the ocean over the cliff. The linear basin has a defined flow line at the top for a couple hundred feet and the flow disperses. Defined flow along the midsection of the basin only occurs where there are foot paths. However the flow begins to concentrate near the discharge point North of the existing lower parking lot and has caused a significant amount of erosion. The average flow velocity for the basin is low, however the concentrated nature of the flow Northwest of the lower parking lot causes the erosion.

4.3.3 Basin H

Basin H is a small basin located West of the lower parking lot and is used as a sample basin to determine the runoff characteristics for many similar areas which were not analyzed. All these areas have runoff flows which can be characterized as sheet flow and do not contain defined flow paths. The analysis indicated that the concentrated runoff is less than 0.8 cfs per acre for the 50-year storm. Since the actual flows are not concentrated, the flow rate over the land is much less at any given location. It should be noted however, that this does not mean the area is not susceptible to erosion. The soil characteristics will still lead to uniform erosion mainly due to the rainfall impact.

4.3.4 Basin I

Basin I begins West of the University and discharges to the ocean over the cliff near Garbage Beach. The flow path is poorly defined, similar to Basin G, and diverges and converges along its length. The intended point of convergence is a grated inlet located at the Southwest corner of the lower parking lot and is a part of an existing storm drain system. However, the topography indicates that the flow will not concentrate at the grate, and observation during a storm verified this. The drainage system discharges the flow over the cliff via a concrete brow ditch. The surface erosion in the area clearly shows the drainage system is not functioning as it was designed to function.

4.3.5 Basin J

Basin J begins near the slopes North of the upper parking lot and drains over the cliffs South of the lower parking area. The upper parking lot concentrates the flows at the South West corner of the lot and the flow remains concentrated for a distance. The vegetation eventually disperses the flow. The most likely way the flow is conveyed to the West is via walking paths. A majority of the basin's land contains good ground cover and has experienced less erosion than other basins. However, near the cliff there is observable erosion and it was accelerated near large rocks placed along the cliff. These rocks cause flow to be concentrated around their contact with the ground, and successive storms erode the soil at this contact around the rock. In time the soil support will be reduced to a point of collapse and the rock will fall/move to a more stable location below. The process will repeat until the rock eventually finds its way to the beach. In addition, this undercutting of the rock combined with the animal habitats and fractured nature of the area geology may lead to subsurface piping.

4.3.6 Basin K

Basin K is quite similar to Basin J. It begins near the University, West of Western Loop Road and flows West toward the cliffs. For the most part it has good ground cover; however, there is evidence that the flow begins to concentrate to the South boundary immediately West of the access road. There are several walking paths that have experienced moderate to severe erosion in the area and some of the flow most likely goes over the South boundary to Basin L, the badlands. Again the rocks appear to have accelerated the erosion in the area.

4.3.7 Basin L

Basin L constitutes the area easily identified as badlands. The basin begins West of Western Loop Road and discharges over the cliff, and also may receive runoff from Basin M at locations where the curb is missing along the road. The sporadic ground cover has accelerated the surface erosion at many locations and has created several crevasses. Though the possibility exists that unobserved subsurface flow may have aided in the process of creating these crevasses, the most likely cause is surface erosion caused by rainfall impact and storm water runoff with the aid of weakly cohesive soil burdened with human and burrowing animal activities.

4.3.8 Basin M

Basin M is the largest basin impacting the linear park. The basin was delineated to two subbasins. Basin M1, which discharges over the cliff to the ocean, contains the ravine located North of the softball field. Basin M2, which converges at the existing Arizona crossing, constitutes a significant portion of the University. A majority of the University drainage systems are designed to converge at the Arizona crossing via pipe networks, surface flows or street flows. An Arizona crossing is typically an at-grade paved roadway located at a low point of a road that plays the role of a broad crested weir during a storm event and is capable of routing all of the flow converging at the crossing. All of the flow within Basin M2 is routed to Basin M1 via the Arizona crossing, which also includes a 24-inch low flow pipe that is designed to carry approximately 17 ± cfs of flow before the road is inundated. In addition to the Basin M2 tributary area, 60-percent of Basin Q is also expected to converge at the Arizona crossing. The erosion taking place downstream of the Arizona crossing indicates the flow velocities and volume combination acting upon the contact area between hard concrete and soft soil is destructive.

Basin M2 has a small tributary but conveys a significant part, if not all of the flow crossing the Arizona crossing. The possibility exists that some of the flow may be routed to Basin L during large storms, but was assumed to be contained within the ravine. The flow discharges to the ocean via a well defined gorge and is the significant discharge point with in the park.

The runoff calculations conducted for the combined basins yielded a significantly smaller runoff rate at the point of discharge, from 63.18 cfs to 34.38 cfs. This is due to the reduction in intensity due to increased Tc, larger upstream basin with faster flow velocities and small downstream basin with slow flow velocities. The design manual recommends the use of larger flow rate in these situations.

4.3.9 Basin N

The basin South of Basin M1 consists of the Northern side of the existing softball field. The basin does not contain any defined flow paths until the flow reaches the West side of the field near the cliff. The flows generated within the basin are small, yet the erosion near the cliff indicates concentrated flows at a couple of locations.

4.3.10 Basin O

The basin South of Basin N consists of the Southern side of the existing softball field and the Northern portion of the parking area. Again the basin does not contain any defined flow paths until the flow reaches the Southwest side of the field near the cliff. This basin receives a portion of Basin Q runoff, which for the purpose of the analysis was assumed to be 20-percent of Basin Q flow. The combined flows are not significant but the erosion near the cliff indicates concentrated flows.

4.3.11 Basin P

Basin P consists of the Southern side of the parking area. Again the basin does not contain any defined flow paths until the flow reaches the West side of the parking lot near the cliff. This basin also receives a portion of Basin Q runoff, which for the purpose of the analysis was assumed to be 20-percent of Basin Q flow. The combined flows are not significant but the erosion near the cliff is visible and is most likely caused by concentrated flows.

4.3.12 Basin Q

Basin Q would have been a part of Basin M2 if it did not appear to discharge some of the basin runoff to Basins O and P in addition to M2. The runoff from the basin is primarily conveyed over the road, and in general flows toward the Arizona crossing. However, immediately South of the crossing an opening in the curb discharges flow to Basin P, and the driveway to the parking lot diverts flow to Basin Q. For the purpose of analysis, it was assumed the flow split is 60-percent to Basin M2 with Basins O and P receiving 20-percent of the flow each.

4.3.13 Basin R

Basin R, which includes the Southern part of the Linear Park, is nearly totally unimproved and could have been analyzed as a single basin. However due to changes in slopes along the flow line, it was delineated into two basins; upper and lower, R2 and R1 respectively. The flow line is well defined and the runoff converges at the top of the cliff near the South end of the park. The erosion in the area is primarily caused by human and burrowing animal activities.

Table 9

Basin Outlet	Drainage Zones	Area ACRE	1 Year		5 Year		10 Year		50 Year	
			I IN/HR	Q CFS	I IN/HR	Q CFS	I IN/HR	Q CFS	I IN/HR	Q CFS
F	F	3.68	1.01	1.08	1.31	1.40	1.51	1.61	1.91	2.04
G	G	6.65	0.72	1.49	0.94	1.93	1.08	2.23	1.37	2.82
H	H	0.71	1.47	0.30	1.91	0.39	2.21	0.45	2.79	0.58
I	I	2.02	0.91	0.57	1.19	0.74	1.37	0.86	1.74	1.09
J	J	6.77	0.84	1.76	1.09	2.29	1.26	2.64	1.60	3.35
K	K	3.56	0.94	0.84	1.23	1.09	1.42	1.26	1.79	1.60
L	L	4.16	0.95	0.99	1.24	1.29	1.43	1.49	1.81	1.88
M1	M1+M2+60%Q	43.57	0.87	17.12	1.13	23.45	1.31	27.09	1.66	34.38
M2	M2	38.86	1.79	32.78	2.34	42.78	2.71	49.54	3.46	63.18
N	N	2.53	1.06	0.86	1.38	1.12	1.60	1.29	2.02	1.64
O	O+20%Q	4.94	0.92	1.47	1.20	1.86	1.38	2.15	1.75	2.72
P	P+20%Q	4.20	0.80	1.10	1.05	1.43	1.21	1.65	1.54	2.10
Q	Q	4.87	1.81	2.99	2.37	3.93	2.75	4.56	3.50	5.79
R1	R1+R2	12.39	0.63	2.12	0.82	2.76	0.95	3.19	1.20	4.04
R2	R2	6.66	0.94	1.82	1.22	2.36	1.41	2.73	1.79	3.45

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APPENDIX

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SUNSET CLIFFS HYDROLOGY

EQUATION

$I = 7.44\, P_6\, D^{-0.645}$

I = Intensity (in/hr)

P_6 = 6-Hour Precipitation (in)

D = Duration (min)

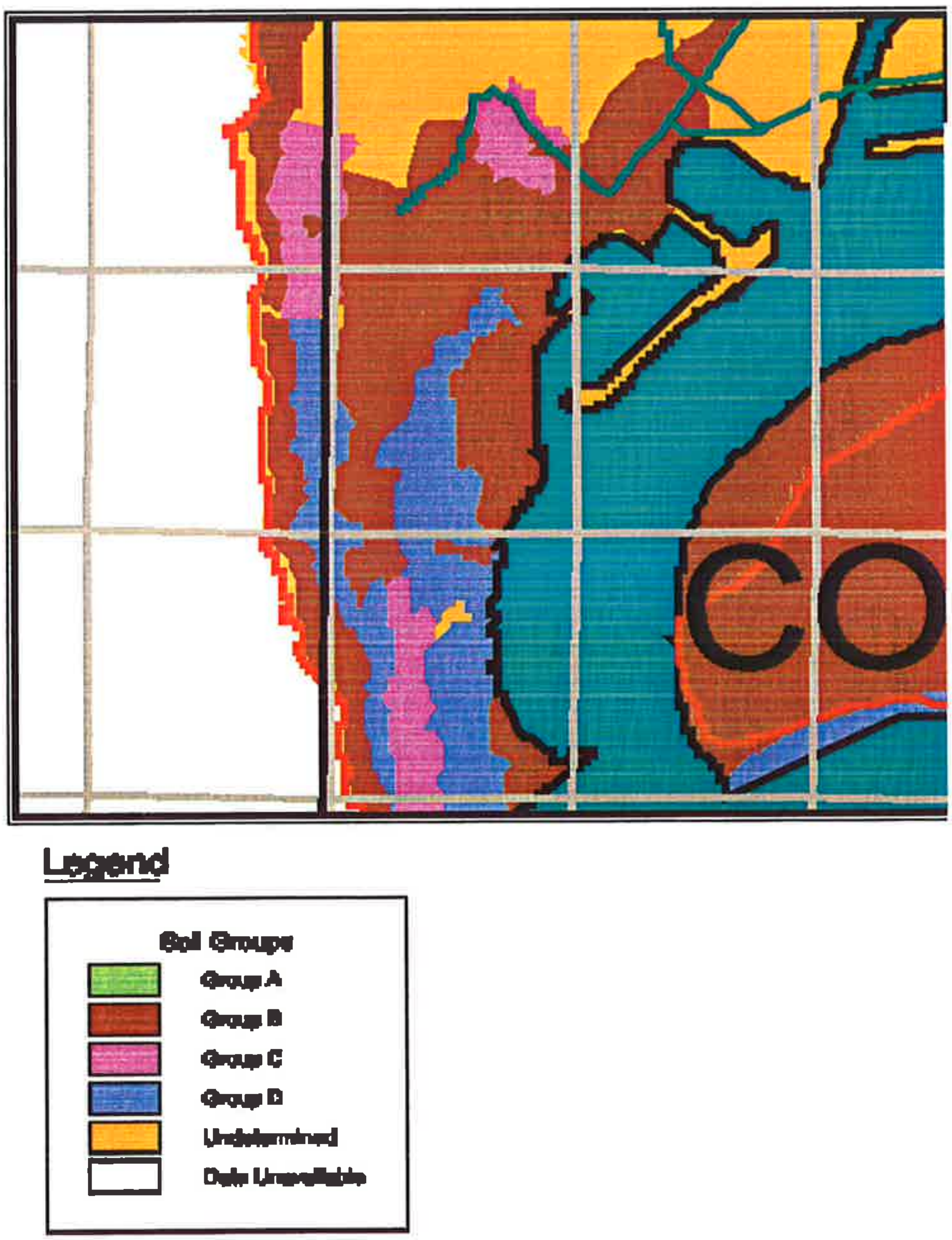
$$T = \frac{1.8\,(1.1-C)\sqrt{D}}{\sqrt[3]{S}}$$

$$T_c = \left(\frac{11.9L^3}{\Delta E}\right)^{0.385}$$

Tc = Time of concentration (hours)

L = Watercourse Distance (miles)

ΔE = Change in elevation along effective slope line (See Figure 3-5) (feet)



$$Q/L_T = 0.7\,(a+y)^{3/2}$$

where ...

Q = interception capacity c

y = depth of flow approacl

a = depth of depression of

L_T = length of clear openin

$$Q = C_W L_W d^{3/2}$$

where ...

Q = inlet capacity (ft^3/s),

C_W = weir discharge coeffic

L_W = weir length (ft); and

d = flow depth (ft).

$$Q = C_W P_e d^{3/2}$$

where ...

Q = inlet capacity of the gr

C_W = weir coefficient (C_W=3

P_e = effective grate perimet

d = flow depth approachin

To account for the effects of clogging fifty percent (C_L=0.50) shall be applied

$$P_e = (1-C_L)P$$

where ...

P_e = effective grate perimet

C_L = clogging factor (C_L=0.

P = actual grate perimeter (vanes); P=2W+L for g approaching from all s grate has an actual peri when flow approaches

$$Q = C_O A_e (2gd)^{1/2}$$

$$A_e = (1-C_A)A$$

where ...

Q = inlet capacity of the gr

C_O = orifice coefficient (C_O

g = gravitational accelerat

d = flow depth above inlet

A_e = effective (clogged) gr

C_A = area clogging factor ((

A = actual opening area of vanes). The actual op grate is A=4.7 ft^2. The *Design Manual* (HEC configurations.

NOTE: Cells with yellow fill is an averaged value used for this study.

Land Use			Runoff Coefficient “C”				
			Soil Type				
NRCS Elements	County Elements	% IMPER	A	B	B&C	C	D
Undisturbed Natural Terrain (Natural)	Permanent Open Space	0*	0.20	0.25	0.28	0.30	0.35
Special average for the hilside park				0.29	0.31	0.33	
Low Density Residential (LDR)	Residential, 1.0 DU/A or less	10	0.27	0.32	0.34	0.36	0.41
Low Density Residential (LDR)	Residential, 2.0 DU/A or less	20	0.34	0.38	0.40	0.42	0.46
Low Density Residential (LDR)	Residential, 2.9 DU/A or less	25	0.38	0.41	0.43	0.45	0.49
Medium Density Residential (MDR)	Residential, 4.3 DU/A or less	30	0.41	0.45	0.47	0.48	0.52
Medium Density Residential (MDR)	Residential, 7.3 DU/A or less	40	0.48	0.51	0.53	0.54	0.57
Medium Density Residential (MDR)	Residential, 10.9 DU/A or less	45	0.52	0.54	0.56	0.57	0.60
Medium Density Residential (MDR)	Residential, 14.5 DU/A or less	50	0.55	0.58	0.59	0.60	0.63
High Density Residential (HDR)	Residential, 24.0 DU/A or less	65	0.66	0.67	0.68	0.69	0.71
High Density Residential (HDR)	Residential, 43.0 DU/A or less	80	0.76	0.77	0.78	0.78	0.79
Commercial/Industrial (N. Com)	Neighborhood Commercial	80	0.76	0.77	0.78	0.78	0.79
Commercial/Industrial (G. Com)	General Commercial	85	0.80	0.80	0.81	0.81	0.82
Commercial/Industrial (O.P. Com)	Office Professional/Commercial	90	0.83	0.84	0.84	0.84	0.85
Commercial/Industrial (Limited I.)	Limited Industrial	90	0.83	0.84	0.84	0.84	0.85
Commercial/Industrial (General I.)	General Industrial	95	0.87	0.87	0.87	0.87	0.87

Runoff calculations - 1-yr Storm

NOTES:

Data were taken from the county

I=7.44*P6*Tc^(-0.645)   Q=CiA

NOTE: P6/P24 SHALL WITH 45%-65%, NOT APPLICABLE TO DESERT

EQUATION
$I = 7.44\ P_6\ D^{-0.645}$
I = Intensity (in/hr)
P_6 = 6-Hour Precipitation (in)
D = Duration (min)

85% STORM

P6	1.00	INCH
P24	1.60	INCH
P6/P24	62.50%	
ADJUSTED P6	1.00	INCH

Flow rate is based on   Q=CiA

Drainage		Area	C	Tc	Intensity	Q	REMARK
Zone		ACRE		MIN	IN/HR	CFS	
X1	161,285	3.70	0.45	13.61	1.38	2.30	All basins contributes, discharge to CI.
X1 thru X9 + A	6,382,907	146.53	0.475	30.14	0.83	57.50	All basins converge. Bypass floods the intersection.
X2	98,067	2.25	0.45	13.71	1.37	1.39	Basins X3-X9 contributes, discharge to CI and Bypass to X1.
X2 thru X9+A	6,221,622	142.83	0.475	29.00	0.85	57.54	Basins X3-X9 added to X2, discharge to CI and Bypass to X1.
X3	113,134	2.60	0.45	9.90	1.70	1.98	Discharges to X5
X3+X6+X9	752,144	17.27	0.475	19.13	1.11	9.10	Half of X9 and X6 was added to this basin. Bypass goes to X5.
X4	249,330	5.72	0.48	21.08	1.04	2.86	Discharges to X2
X4+A	2,626,047	60.29	0.471	26.17	0.91	25.72	Runoff from basin A is added to the basin
X5	338,886	7.78	0.48	13.75	1.37	5.12	Discharges to X2
X5+X7+X8	2,745,364	63.02	0.48	18.08	1.15	34.79	All upstrem basins are aded and the bypass to X2.
X6	200,548	4.60	0.48	7.64	2.00	4.43	Discharges to 15' CI, bypass to X3.
X6+X9/2	419,779	9.64	0.48	18.20	1.15	5.30	Half of X9 was added to this basin. Bypass goes to X3.
X7	788,258	18.10	0.48	11.36	1.55	13.48	Bypass goes to X5.
X8	1,618,220	37.15	0.48	16.79	1.21	21.51	Bypass goes to X5.
X9	438,462	10.07	0.48	14.69	1.31	6.35	The flow is split between X3 and X6.
A	2,376,717	54.56	0.47	14.22	1.34	34.42	Imp w/gutter flow, dis. to the ocean
B1	282,112	6.48	0.45	19.57	1.09	3.18	Imp w/gutter flow, dis. to the ocean
B2	523,579	12.02	0.45	15.50	1.27	6.87	Imp w/gutter flow, dis. to SD inlets, need cap analysis
B3	2,071,522	47.56	0.48	20.81	1.05	23.97	Imp w/gutter flow, dis. to SD inlets, need cap analysis
B4	586,419	13.46	0.48	20.30	1.07	6.90	Imp w/gutter flow, dis. to SD inlets, need cap analysis
B5	134,586	3.09	0.45	12.15	1.49	2.07	Imp surface flow, discharges to SD inlet
B6	890,161	20.44	0.48	16.67	1.21	11.88	Imp w/gutter flow, dis. to SD inlets, need cap analysis
B4+B6	1,476,580	33.90	0.48	22.00	1.01	16.49	Total flow from B4 + B6 not accounting the inlets along the way.
B3+B4+B6	3,548,102	81.45	0.48	22.25	1.01	39.33	Total flow from B3- B4 + B6 not accounting the inlets along the way.
B2-4+B6	4,071,681	93.47	0.476	23.31	0.98	43.44	Total flow from B2- B4 + B6 not accounting the inlets along the way.
B1-4+B6	4,353,793	99.95	0.474	25.38	0.92	43.82	Total flow from B1- B4 + B6 not accounting the inlets along the way.
C1	270,876	6.22	0.45	17.60	1.17	3.27	Imp w/gutter flow, dis. to the ocean
C1+C3+C4	1,845,232	42.36	0.471	23.38	0.97	19.46	Since the C1 slope is less steep than C4, C1 velocities were used.
C2	145,280	3.34	0.45	17.85	1.16	1.74	Imp w/gutter flow, dis. to SD inlets, need cap analysis
C2+D1+D2	1,375,669	31.58	0.474	17.74	1.16	17.41	Combined flow of C2, D1 and D2
C3	763,232	17.52	0.47	15.32	1.28	10.54	Imp w/gutter flow, dis. to SD inlets, need cap analysis
C4	811,124	18.62	0.48	15.72	1.26	11.25	Imp w/gutter flow, dis. to SD inlets, need cap analysis
D1	454,937	10.44	0.47	14.98	1.30	6.37	Imp w/gutter flow, dis. to the ocean at Carmelo St. Bypass to C2
D1+D2	1,230,389	28.25	0.476	15.90	1.25	16.81	Combined flow of D1 and D2
D2	775,452	17.80	0.48	12.43	1.46	12.51	Q is routed to D1 via a pipe across Amiford to Cornish
E	562,275	12.91	0.47	15.55	1.27	7.69	Imp w/gutter flow, dis. to the ocean at Ladera
F	160,475	3.68	0.29	22.22	1.01	1.08	Unimproved w/dirt road section, dis to ocean
G	289,727	6.65	0.31	37.32	0.72	1.49	Unimproved w/dirt road section, dis to ocean
H	30,968	0.71	0.29	12.35	1.47	0.30	Unimproved W. of lower parking area. Disch to Pacific
I	87,879	2.02	0.31	25.83	0.91	0.57	Unimproved w/dirt road section, dis to ocean
J	294,738	6.77	0.31	29.40	0.84	1.76	Unimp lower area and upper parking area, dis to Pacific.
K	155,130	3.56	0.25	24.56	0.94	0.84	Unimproved w/dirt road section, dis to ocean
L	181,037	4.16	0.25	24.16	0.95	0.99	Bad Lands, unimp w/dirt road section, dis to ocean
M1	77,901	1.79	0.25	23.57	0.97	0.43	Grand canyon Dischrges to ocean
M1+M2+60%Q	1,897,998	43.57	0.452	27.91	0.87	17.12	Bad lands with additional flows from upstream basins M1 and 60% of Q
M2	1,692,767	38.86	0.47	9.07	1.79	32.78	Major section of PLNC, converge at AZ x-ing
N	110,202	2.53	0.32	20.38	1.06	0.86	Unimproved w/lower field section, dis to ocean
O	172,659	3.96	0.32	21.69	1.02	1.30	Unimp w/lower field & parking section, dis to Pacific
O+20%Q	215,102	4.94	0.324	25.70	0.92	1.47	Basin O and 20% of Q
P	140,640	3.23	0.32	27.48	0.88	0.91	Unimp w/parking section, dis to Pacific
P+20%Q	183,083	4.20	0.325	31.49	0.80	1.10	Basin P and 20% of Q
Q	212,216	4.87	0.34	8.98	1.81	2.99	Part of PLNC flows N. along Lomaland Dr.
R1	249,379	5.72	0.25	25.11	0.93	1.33	Unimproved w/dirt road section, dis to ocean
R1+R2	539,545	12.39	0.272	45.79	0.63	2.12	Combined flows of R1 and R2
R2	290,166	6.66	0.29	24.67	0.94	1.82	Primarily unimp and will remain so.
TOTAL		815.69				410.70	

Runoff calculations - 5-yr Storm

NOTES:

Data were taken from the county

I=7.44*P6*Tc^(-0.645) Q=CiA

NOTE: P6/P24 SHALL WITH 45%-65%, NOT APPLICABLE TO DESERT

EQUATION
$I = 7.44\, P_6\, D^{-0.645}$
I = Intensity (in/hr)
P_6 = 6-Hour Precipitation (in)
D = Duration (min)

85% STORM

P6	1.30	INCH
P24	2.10	INCH
P6/P24	61.90%	
ADJUSTED P6	1.30	INCH

Flow rate is based on Q=CiA

Drainage		Area	C	Tc	Intensity	Q	REMARK
Zone		ACRE		MIN	IN/HR	CFS	
X1	161,285	3.70	0.45	13.43	1.81	3.02	All basins contributes, discharge to CI.
X1 thru X9 + A	6,382,907	146.53	0.475	26.59	1.17	81.06	All basins converge. Bypass floods the intersection.
X2	98,067	2.25	0.45	13.71	1.79	1.81	Basins X3-X9 contributes, discharge to CI and Bypass to X1.
X2 thru X9+A	6,221,622	142.83	0.475	25.45	1.20	81.38	Basins X3-X9 added to X2, discharge to CI and Bypass to X1.
X3	113,134	2.60	0.45	9.77	2.22	2.60	Discharges to X5
X3+X6+X9	752,144	17.27	0.475	18.57	1.47	12.07	Half of X9 and X6 was added to this basin. Bypass goes to X5.
X4	249,330	5.72	0.48	18.25	1.49	4.08	Discharges to X2
X4+A	2,626,047	60.29	0.471	23.22	1.27	36.12	Runoff from basin A is added to the basin
X5	338,886	7.78	0.48	13.50	1.80	6.74	Discharges to X2
X5+X7+X8	2,745,364	63.02	0.48	17.74	1.51	45.78	All upstrem basins are aded and the bypass to X2.
X6	200,548	4.60	0.48	7.46	2.65	5.85	Discharges to 15' CI, bypass to X3.
X6+X9/2	419,779	9.64	0.48	17.58	1.52	7.04	Bypass goes to X3
X7	788,258	18.10	0.48	11.07	2.05	17.81	Bypass goes to X5
X8	1,618,220	37.15	0.48	16.49	1.59	28.29	Bypass goes to X5
X9	438,462	10.07	0.48	14.25	1.74	8.42	The flow is split between X3 and X6
A	2,376,717	54.56	0.47	13.84	1.78	45.54	Imp w/gutter flow, dis. to the ocean
B1	282,112	6.48	0.45	19.42	1.43	4.16	Imp w/gutter flow, dis. to the ocean
B2	523,579	12.02	0.45	15.32	1.66	9.00	Imp w/gutter flow, dis. to SD inlets, need cap analysis
B3	2,071,522	47.56	0.48	20.24	1.39	31.73	Imp w/gutter flow, dis. to SD inlets, need cap analysis
B4	586,419	13.46	0.48	20.06	1.40	9.04	Imp w/gutter flow, dis. to SD inlets, need cap analysis
B5	134,586	3.09	0.45	11.98	1.95	2.71	Imp surface flow, discharges to SD inlet
B6	890,161	20.44	0.48	16.26	1.60	15.70	Imp w/gutter flow, dis. to SD inlets, need cap analysis
B4+B6	1,476,580	33.90	0.48	21.58	1.33	21.70	Total flow from B4 + B6 not accounting the inlets along the way.
B3+B4+B6	3,548,102	81.45	0.48	21.82	1.32	51.77	Total flow from B3- B4 + B6 not accounting the inlets along the way.
B2,3,4+B6	4,071,681	93.47	0.476	22.83	1.29	57.24	Total flow from B2- B4 + B6 not accounting the inlets along the way.
B1-4+B6	4,353,793	99.95	0.474	24.85	1.22	57.74	Total flow from B1- B4 + B6 not accounting the inlets along the way.
C1	270,876	6.22	0.45	17.29	1.54	4.31	Imp w/gutter flow, dis. to the ocean
C1+C3+C4	1,845,232	42.36	0.471	22.89	1.28	25.64	Since the C1 slope is less steep than C4, C1 velocities were used.
C2	145,280	3.34	0.45	17.85	1.51	2.26	Imp w/gutter flow, dis. to SD inlets, need cap analysis
C2+D1+D2	1,375,669	31.58	0.474	17.35	1.54	22.96	Combined flow of C2, D1 and D2
C3	763,232	17.52	0.47	15.25	1.67	13.74	Imp w/gutter flow, dis. to SD inlets, need cap analysis
C4	811,124	18.62	0.48	15.52	1.65	14.75	Imp w/gutter flow, dis. to SD inlets, need cap analysis
D1	454,937	10.44	0.47	14.84	1.70	8.33	Imp w/gutter flow, dis. to the ocean
D1+D2	1,230,389	28.25	0.476	15.55	1.65	22.16	Combined flow of D1 and D2
D2	775,452	17.80	0.48	12.18	1.93	16.48	Q is routed to D1 via a pipe across Amiford to Cornish
E	562,275	12.91	0.47	15.43	1.66	10.04	Imp w/gutter flow, dis. to the ocean at Ladera
F	160,475	3.68	0.29	22.22	1.31	1.40	Unimproved w/dirt road section, dis to ocean
G	289,727	6.65	0.31	37.32	0.94	1.93	Unimproved w/dirt road section, dis to ocean
H	30,968	0.71	0.29	12.35	1.91	0.39	Unimproved W. of lower parking area. Disch to Pacific
I	87,879	2.02	0.31	25.83	1.19	0.74	Unimproved w/dirt road section, dis to ocean
J	294,738	6.77	0.31	29.40	1.09	2.29	Unimp lower area and upper parking area, dis to Pacific.
K	155,130	3.56	0.25	24.56	1.23	1.09	Unimproved w/dirt road section, dis to ocean
L	181,037	4.16	0.25	24.16	1.24	1.29	Bad Lands, unimp w/dirt road section, dis to ocean
M1	77,901	1.79	0.25	23.57	1.26	0.56	Grand canyon Dischrges to ocean
M1+M2+60%Q	2,094,395	48.08	0.431	27.86	1.13	23.45	Bad lands with additional flows from upstream basins M1 and 60% of Q
M2	1,692,767	38.86	0.47	9.01	2.34	42.78	Major section of PLNC, converge at AZ x-ing
N	110,202	2.53	0.32	20.38	1.38	1.12	Unimproved w/lower field section, dis to ocean
O	172,659	3.96	0.32	21.69	1.33	1.69	Unimp w/lower field & parking section, dis to Pacific
O+20%Q	222,535	5.11	0.304	25.57	1.20	1.86	Basin O and 20% of Q
P	140,640	3.23	0.32	27.48	1.14	1.18	Unimp w/parking section, dis to Pacific
P+20%Q	183,083	4.20	0.325	31.36	1.05	1.43	Basin P and 20% of Q
Q	212,216	4.87	0.34	8.84	2.37	3.93	Part of PLNC flows N. along Lomaland Dr.
R1	249,379	5.72	0.25	25.11	1.21	1.73	Unimproved w/dirt road section, dis to ocean
R1+R2	539,545	12.39	0.272	45.79	0.82	2.76	Combined flows of R1 and R2
R2	290,166	6.66	0.29	24.67	1.22	2.36	Primarily unimp and will remain so.
TOTAL		19.71				541.00	

Runoff calculations - 10-yr Storm

NOTES:

Data were taken from the county

I=7.44*P6*Tc^(-0.645)   Q=CiA

NOTE: P6/P24 SHALL WITH 45%-65%, NOT APPLICABLE TO DESERT

EQUATION

$I = 7.44\, P_6\, D^{-0.645}$

I = Intensity (in/hr)

P_6 = 6-Hour Precipitation (in)

D = Duration (min)

85% STORM

P6	1.50	INCH
P24	2.80	INCH
P6/P24	53.57%	
ADJUSTED P6	1.50	INCH

Flow rate is based on Q=CiA

Drainage		Area	C	Tc	Intensity	Q	REMARK
Zone		ACRE		MIN	IN/HR	CFS	
X1	161,285	3.70	0.45	13.37	2.10	3.49	All basins contributes, discharge to CI.
X1 thru X9 + A	6,382,907	146.53	0.475	25.56	1.38	95.94	All basins converge. Bypass floods the intersection.
X2	98,067	2.25	0.45	13.71	2.06	2.09	Basins X3-X9 contributes, discharge to CI and Bypass to X1.
X2 thru X9+A	6,221,622	142.83	0.475	24.42	1.42	96.44	Basins X3-X9 added to X2, discharge to CI and Bypass to X1.
X3	113,134	2.60	0.45	9.71	2.58	3.01	Discharges to X5
X3+X6+X9	752,144	17.27	0.475	18.35	1.71	14.03	Half of X9 and X6 was added to this basin. Bypass goes to X5.
X4	249,330	5.72	0.48	17.54	1.76	4.83	Discharges to X2
X4+A	2,626,047	60.29	0.471	22.34	1.50	42.72	Runoff from basin A is added to the basin
X5	338,886	7.78	0.48	13.36	2.10	7.83	Discharges to X2
X5+X7+X8	2,745,364	63.02	0.48	17.42	1.77	53.44	All upstrem basins are aded and the bypass to X2.
X6	200,548	4.60	0.48	7.38	3.07	6.79	Discharges to 15' CI, bypass to X3.
X6+X9/2	419,779	9.64	0.48	17.34	1.77	8.20	Bypass goes to X3
X7	788,258	18.10	0.48	10.81	2.40	20.88	Bypass goes to X5
X8	1,618,220	37.15	0.48	16.20	1.85	33.01	Bypass goes to X5
X9	438,462	10.07	0.48	14.09	2.03	9.79	The flow is split between X3 and X6
A	2,376,717	54.56	0.47	13.60	2.07	53.14	Imp w/gutter flow, dis. to the ocean
B1	282,112	6.48	0.45	19.15	1.66	4.84	Imp w/gutter flow, dis. to the ocean
B2	523,579	12.02	0.45	15.23	1.93	10.42	Imp w/gutter flow, dis. to SD inlets, need cap analysis
B3	2,071,522	47.56	0.48	20.11	1.61	36.76	Imp w/gutter flow, dis. to SD inlets, need cap analysis
B4	586,419	13.46	0.48	19.84	1.62	10.50	Imp w/gutter flow, dis. to SD inlets, need cap analysis
B5	134,586	3.09	0.45	11.90	2.26	3.14	Imp surface flow, discharges to SD inlet
B6	890,161	20.44	0.48	16.07	1.86	18.26	Imp w/gutter flow, dis. to SD inlets, need cap analysis
B4+B6	1,476,580	33.90	0.48	21.39	1.55	25.18	Total flow from B4 + B6 not accounting the inlets along the way.
B3+B4+B6	3,548,102	81.45	0.48	21.62	1.54	60.09	Total flow from B3- B4 + B6 not accounting the inlets along the way.
B2,3,4+B6	4,071,681	93.47	0.476	22.60	1.49	66.47	Total flow from B2- B4 + B6 not accounting the inlets along the way.
B1-4+B6	4,353,793	99.95	0.474	24.55	1.42	67.14	Total flow from B1- B4 + B6 not accounting the inlets along the way.
C1	270,876	6.22	0.45	17.14	1.79	5.00	Imp w/gutter flow, dis. to the ocean
C1+C3+C4	1,845,232	42.36	0.471	22.16	1.51	30.21	Since the C1 slope is less steep than C4, C1 velocities were used.
C2	145,280	3.34	0.45	17.82	1.74	2.61	Imp w/gutter flow, dis. to SD inlets, need cap analysis
C2+D1+D2	1,375,669	31.58	0.474	17.10	1.79	26.74	Combined flow of C2, D1 and D2
C3	763,232	17.52	0.47	14.99	1.95	16.03	Imp w/gutter flow, dis. to SD inlets, need cap analysis
C4	811,124	18.62	0.48	15.44	1.91	17.07	Imp w/gutter flow, dis. to SD inlets, need cap analysis
D1	454,937	10.44	0.47	14.72	1.97	9.67	Imp w/gutter flow, dis. to the ocean
D1+D2	1,230,389	28.25	0.476	15.35	1.92	25.79	Combined flow of D1 and D2
D2	775,452	17.80	0.48	12.06	2.24	19.14	Q is routed to D1 via a pipe across Amiford to Cornish
E	562,275	12.91	0.47	15.38	1.91	11.62	Imp w/gutter flow, dis. to the ocean at Ladera
F	160,475	3.68	0.29	22.22	1.51	1.61	Unimproved w/dirt road section, dis to ocean
G	289,727	6.65	0.31	37.32	1.08	2.23	Unimproved w/dirt road section, dis to ocean
H	30,968	0.71	0.29	12.35	2.21	0.45	Unimproved W. of lower parking area. Disch to Pacific
I	87,879	2.02	0.31	25.83	1.37	0.86	Unimproved w/dirt road section, dis to ocean
J	294,738	6.77	0.31	29.40	1.26	2.64	Unimp lower area and upper parking area, dis to Pacific.
K	155,130	3.56	0.25	24.56	1.42	1.26	Unimproved w/dirt road section, dis to ocean
L	181,037	4.16	0.25	24.16	1.43	1.49	Bad Lands, unimp w/dirt road section, dis to ocean
M1	77,901	1.79	0.25	23.57	1.45	0.65	Grand canyon Dischrges to ocean
M1+M2+60%Q	2,094,395	48.08	0.431	27.81	1.31	27.09	Bad lands with additional flows from upstream basins M1 and 60% of Q
M2	1,692,767	38.86	0.47	8.96	2.71	49.54	Major section of PLNC, converge at AZ x-ing
N	110,202	2.53	0.32	20.38	1.60	1.29	Unimproved w/lower field section, dis to ocean
O	172,659	3.96	0.32	21.69	1.53	1.95	Unimp w/lower field & parking section, dis to Pacific
O+20%Q	222,535	5.11	0.304	25.49	1.38	2.15	Basin O and 20% of Q
P	140,640	3.23	0.32	27.48	1.32	1.36	Unimp w/parking section, dis to Pacific
P+20%Q	183,083	4.20	0.325	31.27	1.21	1.65	Basin P and 20% of Q
Q	212,216	4.87	0.34	8.76	2.75	4.56	Part of PLNC flows N. along Lomaland Dr.
R1	249,379	5.72	0.25	25.11	1.40	2.00	Unimproved w/dirt road section, dis to ocean
R1+R2	539,545	12.39	0.272	45.79	0.95	3.19	Combined flows of R1 and R2
R2	290,166	6.66	0.29	24.67	1.41	2.73	Primarily unimp and will remain so.
TOTAL		19.71				628.51	

Runoff calculations - 50-yr Storm

NOTES:

Data were taken from the county

I=7.44*P6*Tc^(-0.645) Q=CiA

NOTE: P6/P24 SHALL WITH 45%-65%, NOT APPLICABLE TO DESERT

EQUATION

$I = 7.44\, P_6\, D^{-0.645}$

I = Intensity (in/hr)

P_6 = 6-Hour Precipitation (in)

D = Duration (min)

85% STORM

P6	1.90	INCH
P24	3.30	INCH
P6/P24	57.58%	
ADJUSTED P6	1.90	INCH

Flow rate is based on Q=CiA

Drainage		Area	C	Tc	Intensity	Q	REMARK
Zone		ACRE		MIN	IN/HR	CFS	
X1	161,285	3.70	0.45	13.09	2.69	4.48	All basins contributes, discharge to CI.
X1 thru X9 + A	6,382,907	146.53	0.475	24.91	1.78	123.55	All basins converge. Bypass floods the intersection.
X2	98,067	2.25	0.45	13.44	2.65	2.68	Basins X3-X9 contributes, discharge to CI and Bypass to X1.
X2 thru X9+A	6,221,622	142.83	0.475	23.83	1.83	124.10	Basins X3-X9 added to X2, discharge to CI and Bypass to X1.
X3	113,134	2.60	0.45	9.65	3.27	3.83	Discharges to X5
X3+X6+X9	752,144	17.27	0.475	18.07	2.19	17.95	Half of X9 and X6 was added to this basin. Bypass goes to X5.
X4	249,330	5.72	0.48	17.31	2.25	6.17	Discharges to X2
X4+A	2,626,047	60.29	0.471	21.80	1.94	54.98	Runoff from basin A is added to the basin
X5	338,886	7.78	0.48	12.95	2.71	10.12	Discharges to X2
X5+X7+X8	2,745,364	63.02	0.48	16.64	2.31	69.74	All upstrem basins are aded and the bypass to X2.
X6	200,548	4.60	0.48	7.26	3.93	8.69	Discharges to 15' CI, bypass to X3.
X6+X9/2	419,779	9.64	0.48	17.00	2.27	10.52	Bypass goes to X3
X7	788,258	18.10	0.48	10.57	3.09	26.83	Bypass goes to X5
X8	1,618,220	37.15	0.48	15.51	2.41	43.01	Bypass goes to X5
X9	438,462	10.07	0.48	13.86	2.59	12.53	The flow is split between X3 and X6
A	2,376,717	54.56	0.47	13.27	2.67	68.40	Imp w/gutter flow, dis. to the ocean, bypass to X4
B1	282,112	6.48	0.45	19.02	2.11	6.16	Imp w/gutter flow, dis. to the ocean, bypass to A
B2	523,579	12.02	0.45	15.15	2.45	13.25	Imp w/gutter flow, dis. to curb inlets and bypass to mid B1
B3	2,071,522	47.56	0.48	19.62	2.07	47.32	Imp w/gutter flow, dis. to curb inlets and bypass to mid B2
B4	586,419	13.46	0.48	19.63	2.07	13.39	Imp w/gutter flow, dis. to curb inlets and bypass to lower B3
B5	134,586	3.09	0.45	11.68	2.90	4.03	Imp surface flow, discharges to a grated inlet w/6" dike on 3 sides
B6	890,161	20.44	0.48	15.69	2.39	23.48	Imp w/gutter flow, dis. to curb inlets and may bypass to downstream
B4+B6	1,476,580	33.90	0.48	21.02	1.98	32.26	Total flow from B4 + B6 not accounting the inlets along the way.
B3+B4+B6	3,548,102	81.45	0.48	21.24	1.97	77.01	Total flow from B3- B4 + B6 not accounting the inlets along the way.
B2,3,4+B6	4,071,681	93.47	0.476	22.19	1.91	85.20	Total flow from B2- B4 + B6 not accounting the inlets along the way.
B1-4+B6	4,353,793	99.95	0.474	24.11	1.81	86.07	Total flow from B1- B4 + B6 not accounting the inlets along the way.
C1	270,876	6.22	0.45	17.07	2.27	6.34	Imp w/gutter flow, dis. to the ocean
C1+C3+C4	1,845,232	42.36	0.471	22.25	1.91	38.17	Since the C1 slope is less steep than C4, C1 velocities were used.
C2	145,280	3.34	0.45	17.78	2.21	3.31	Imp w/gutter flow, dis. to SD inlets, need cap analysis
C2+D1+D2	1,375,669	31.58	0.474	16.93	2.28	34.09	Combined flow of C2, D1 and D2
C3	763,232	17.52	0.47	14.71	2.50	20.55	Imp w/gutter flow, dis. to SD inlets, need cap analysis
C4	811,124	18.62	0.48	15.33	2.43	21.72	Imp w/gutter flow, dis. to SD inlets, need cap analysis
D1	454,937	10.44	0.47	14.56	2.51	12.33	Imp w/gutter flow, dis. to the ocean
D1+D2	1,230,389	28.25	0.476	15.23	2.44	32.83	Combined flow of D1 and D2
D2	775,452	17.80	0.48	12.05	2.84	24.26	Q is routed to D1 via a pipe across Amiford to Cornish
E	562,275	12.91	0.47	15.27	2.44	14.78	Imp w/gutter flow, dis. to the ocean at Ladera
F	160,475	3.68	0.29	22.22	1.91	2.04	Unimproved w/dirt road section, dis to ocean
G	289,727	6.65	0.31	37.32	1.37	2.82	Unimproved w/dirt road section, dis to ocean
H	30,968	0.71	0.29	12.35	2.79	0.58	Unimproved W. of lower parking area. Disch to Pacific
I	87,879	2.02	0.31	25.83	1.74	1.09	Unimproved w/dirt road section, dis to ocean
J	294,738	6.77	0.31	29.40	1.60	3.35	Unimp lower area and upper parking area, dis to Pacific.
K	155,130	3.56	0.25	24.56	1.79	1.60	Unimproved w/dirt road section, dis to ocean
L	181,037	4.16	0.25	24.16	1.81	1.88	Bad Lands, unimp w/dirt road section, dis to ocean
M1	77,901	1.79	0.25	23.57	1.84	0.82	Grand canyon Dischrges to ocean
M1+M2+60%Q	2,094,395	48.08	0.431	27.72	1.66	34.38	Bad lands with additional flows from upstream basins M1 and 60% of Q
M2	1,692,767	38.86	0.47	8.87	3.46	63.18	Major section of PLNC, converge at AZ x-ing
N	110,202	2.53	0.32	20.38	2.02	1.64	Unimproved w/lower field section, dis to ocean
O	172,659	3.96	0.32	21.69	1.94	2.46	Unimp w/lower field & parking section, dis to Pacific
O+20%Q	222,535	5.11	0.304	25.45	1.75	2.72	Basin O and 20% of Q
P	140,640	3.23	0.32	27.48	1.67	1.72	Unimp w/parking section, dis to Pacific
P+20%Q	183,083	4.20	0.325	31.23	1.54	2.10	Basin P and 20% of Q
Q	212,216	4.87	0.34	8.72	3.50	5.79	Part of PLNC flows N. along Lomaland Dr.
R1	249,379	5.72	0.25	25.11	1.77	2.53	Unimproved w/dirt road section, dis to ocean
R1+R2	539,545	12.39	0.272	45.79	1.20	4.04	Combined flows of R1 and R2
R2	290,166	6.66	0.29	24.67	1.79	3.45	Primarily unimp and will remain so.
TOTAL		19.71				803.15	

SUNSET CLIFFS HYDROLOGY

NOTES:
1. The hydrology analysis is based on San Diego County Hydrology Manual per scope.
2. Due to the high slopes, the overland travel distance shall be limited to 50' in most cases.
3. Density of the area was determined to be about 3 to 4.2 dwellings per acre. Will use C(B)=0.45 &C(B&C)=0.48

EQUATIONS:
1. Sheet Flow: T(i)=1.8(1.1-C)(D^0.5)/(S^0.333)
2. Natural Watersheds: Tc=((11.9*L^3/Eh-El))^0.385, Tc=HOUR, L=MILES

Determining Q and Tt for Gutter Flow for 2 gutters.
1. The gutter flow capacity is determined by using Fig 2-2 of Co Drainage Manuel.
2. The Slope 2 is the average slope of the flow traveling along the gutter
3. Fig 2-2 has a range of velocities that the given slope will maintain.
4. V-(#) is assumed for the given storm frequency based on theSlope 2.
5. Tt is calculated by using the assumed velocity and the length and added to the T(i).
6. Q is calculated in the Q-#-yr sheets devided by two for each gutter.
7. Q calculated is compared with the Q on the figure 2-2.
8. If the Q calc'ed is greater than Q possible, then assumed V-(#) is reduced, if Q calc'ed is less than Q possible, the V-(#) is increased until Q calculated and Q possible according to figure 3-6.
9. Once the difference is within couple tenths, the calculations stop. If needed the depth of flow can be determined by using the Figure 3-6.

Assumed value to determine Tt-#
Velocity and W data were outside of the Figure
Data calculated in another sheet divided by 2
The flow will overtop 6" curb.
Bypass flows from upstream basin is added.
Curb inlet at Sag
Grated Inlet-Orifice
Grated Inlet-Weir

Assumed velocities are based on slope 2 and are taken from SDC Drainage Design Manual Fig 2-2

Basin	Area	Area	Elev 1	Dist to Mid	Elev 2	Dist to End	End Elev	Slope 1	Slope 2	Soil Grp	C	T(i)	Tc	V-1	Tt-1	Q	Depth	V-5	Tt-5	Q	Depth	V-10	Tt-10	Q	Depth	V-50	Tt-50	Q	Depth	CI size	Bypass to
			ft	ft	ft	ft	ft	%	%			min	min	ft/sec	min	calc	ft	ft/sec	min	calc	ft	ft/sec	min	calc	ft	ft/sec	min	calc	ft	ft	
X1	161,285	3.70	36.7	50	36.5	802	25	1.0%	1.5%	B	0.45	8.27	N/A	2.35	13.61	0.70	0.22	2.43	13.43	0.91	0.35	2.46	13.37	1.04	0.51	2.60	13.09	1.34	1.16	20	
X2	98,067	2.25	34.2	50	34.1	670	28.5	1.0%	0.9%	B	0.45	8.27	N/A	1.90	13.71	1.15	0.45	1.90	13.71	1.51	0.57	1.90	13.71	1.75	0.63	2.00	13.44	2.24	0.67	20	
X3	113,134	2.60	72	40	71	973	36	2.5%	3.8%	B	0.45	5.45	N/A	3.50	9.90	[illegible]	0.19	3.60	9.77	1.51	0.22	3.65	9.71	5.96	0.22	3.70	9.65	2.60	0.24	15	
X4	249,330	5.72	42.5	50	42	1237	32	1.0%	0.8%	C	0.48	7.89	N/A	1.50	21.08	1.43	0.38	1.91	18.25	15.55	0.53	2.05	17.54	15.55	0.59	2.10	17.31	15.55	0.69	15	flooding?
X5	338,886	7.78	50	60	48	1264	32	3.3%	1.3%	C	0.48	5.79	N/A	2.52	13.75	2.58	0.41	2.60	13.50	3.37	0.50	2.65	13.36	3.91	0.50	2.80	12.95	5.06	0.50	5	
X6	200,548	4.60	72	40	70	840	39	5.0%	3.9%	C	0.48	4.13	N/A	3.80	7.64	2.22	0.25	4.00	7.46	3.75	0.26	4.10	7.38	5.18	0.28	4.25	7.26	7.90	0.30	15	X3
X7	788,258	18.10	262	65	258	2543	40	6.2%	8.8%	C	0.48	4.91	N/A	6.40	11.36	6.74	0.30	6.70	11.07	8.91	0.32	7.00	10.81	10.44	0.35	7.30	10.57	13.42	0.38	28	X5
X8	1,618,220	37.15	300	40	299.6	3717	36	1.0%	7.2%	C	0.48	7.06	N/A	6.30	16.79	10.76	0.38	6.50	16.49	14.15	0.39	6.70	16.20	16.50	0.41	7.25	15.51	21.50	0.43	28	X5
X9	438,462	10.07	280	60	279.4	2346	74	1.0%	9.0%	C	0.48	8.64	N/A	6.30	14.69	6.35	0.29	6.80	14.25	8.42	0.32	7.00	14.09	9.79	0.33	7.30	13.86	12.63	0.38	20	X3(50%)-X6(50%)
A	2,376,717	54.56	258	82	252	3674	40	7.3%	5.9%	B&C	0.47	5.29	N/A	6.70	14.22	17.21	0.67	7.00	13.84		0.67	7.20	13.60		0.67	7.50	13.27		0.67	14	X4(100%), slope in CI visinity is 0.4% but at the CI it is a Sag
B1	282,112	6.48	65	106	64	1235	46	1.0%	1.6%	B	0.45	12.05	N/A	2.50	19.57	1.58	0.28	2.55	19.42		0.28	2.65	19.15		0.48	2.70	19.02		0.67		Sag inlet
B2	523,579	12.02	186	103	158	1301	65	1.0%	7.8%	B	0.45	11.87	N/A	5.50	15.50	3.43	0.25	5.80	15.32	4.50	0.27	5.95	15.23	5.21	0.28	6.10	15.15	6.62	0.30	9	
B3	2,071,522	47.56	314	102	310	3708	90	1.0%	6.1%	C	0.48	11.27	N/A	6.30	20.81	11.99	0.38	6.70	20.24	16.86	0.41	6.80	20.11	18.38	0.43	7.20	19.62	23.68	0.47	60	
B4	586,419	13.46	315	180	314	2193	98	1.0%	10.7%	C	0.48	14.97	N/A	6.30	20.30	3.45	0.24	6.60	20.06	4.52	0.26	6.90	19.84	5.25	0.27	7.20	19.63	6.69	0.28	20	
B5	134,586	3.09	250	49	236	583	228	1.0%	1.5%	B	0.45	8.19	N/A	2.25	12.15	[illegible]	0.22	2.35	11.98	[illegible]	0.24	2.40	11.90	[illegible]	0.26	2.55	11.68	[illegible]	0.30	[illegible]	Grated Inlet-Weir
B6	890,161	20.44	311	90	308	2080	180	1.0%	6.4%	C	0.48	10.59	N/A	5.45	16.67	5.94	0.30	5.85	16.26	7.85	0.32	6.05	16.07	9.13	0.34	6.50	15.69	11.74	0.38	28	B4(100%), inlet structures were 2x14' inlets.
C1	270,876	6.22	80	105	79	1249	42	1.0%	3.2%	B	0.45	11.99	N/A	3.40	17.60	[illegible]	0.29	3.60	17.29		0.49	3.70	17.14	[illegible]	0.65	3.75	17.07	[illegible]	0.67		Sag inlet
C2	145,280	3.34	82	165	72	792	46	1.0%	4.1%	B	0.45	15.03	N/A	3.70	17.85	1.34	0.25	3.70	17.85	12.21	0.25	3.75	17.82	3.11	0.33	3.80	17.78	4.53	0.53		Bypass from D1 was added to determine depth needed. But actual available is about 3-4 inches
C3	763,232	17.52	205.5	95	204	1655	70.5	1.0%	8.6%	B&C	0.47	11.05	N/A	6.10	15.32	5.27	0.38	6.20	15.25	6.87	0.41	6.60	14.99	8.01	0.43	7.10	14.71	10.27	0.48	14	
C4	811,124	18.62	313	110	311	1694	76	1.0%	13.2%	C	0.48	11.70	N/A	7.40	15.72	5.62	0.27	7.80	15.52	7.37	0.29	7.95	15.44	8.53	0.30	8.20	15.33	10.86	0.33	14	
D1	454,937	10.44	207.5	75	206	1838	59	1.0%	8.3%	B&C	0.47	9.82	N/A	5.70	14.98	9.44	0.40	5.85	14.84	12.41	0.43	6.00	14.72	14.41	0.46	6.20	14.56	16.30	0.50	12	Added the Q from D2 to determine depth of flow only. Bypass to C2
D2	775,452	17.80	331	85	330	1372	206	1.0%	9.6%	C	0.48	10.29	N/A	10.01	12.43	[illegible]	0.88	11.36	12.18	[illegible]	1.70	12.14	12.06	[illegible]	2.30	12.20	12.05	[illegible]	3.89	[illegible]	Grated Inlet-Orifice. Entire flow added to D1
E	562,275	12.91	285	100	281	1869	65	1.0%	12.2%	B&C	0.47	11.34	N/A	7.00	15.55	3.84	0.24	7.20	15.43	5.02	0.26	7.30	15.38	[illegible]	0.27	7.50	15.27	[illegible]	0.29	[illegible]	
F	160,475	3.68	180	50	162	853	46	36.0%	14.4%	B	0.29	3.12	19.09	0.70	22.22																
G	289,727	6.65	317	50	312.5	1720	50	9.0%	15.7%	B&C	0.31	4.83	32.48	0.86	37.32																
H	30,968	0.71	96	50	89	315	50	14.0%	14.7%	B	0.29	4.28	8.07	0.55	12.35																
I	87,879	2.02	280	50	269	1132	78	22.0%	17.7%	B&C	0.31	3.59	22.24	0.81	25.83																
J	294,738	6.77	310.5	50	300.5	1396	48	20.0%	18.8%	B&C	0.31	3.70	25.70	0.87	29.40																
K	155,130	3.56	229	50	211.5	1038	48	35.0%	16.5%	B	0.25	3.31	21.26	0.77	24.56																
L	181,037	4.16	195.5	50	188	927	50	15.0%	15.7%	B	0.25	4.39	19.77	0.74	24.16																
M1	77,901	1.79	128	50	122	763	38	12.0%	11.8%	B	0.25	4.73	18.85	0.63	23.57																
M2	1,692,767	38.86	347	80	344	1526	134	3.8%	14.5%	B&C	0.47	6.53	29.97	9.50	9.07	[illegible]	0.37	9.70	9.01	[illegible]	0.38	9.90	8.96	[illegible]	0.41	10.30	8.87	[illegible]	0.43	[illegible]	
N	110,202	2.53	123	50	105	701	26	36.0%	12.1%	B	0.32	3.01	17.37	0.62	20.38																
O	172,659	3.96	144	50	140	764	25	8.0%	16.1%	B	0.32	4.96	16.73	0.71	21.69																
P	140,640	3.23	140	50	136	943	32	8.0%	11.6%	B	0.32	4.96	22.51	0.66	27.48																
Q	212,216	4.87	284.5	50	282.5	1122	144	4.0%	12.9%	B&C	0.34	6.09	24.90	6.20	8.98	[illegible]	0.18	6.50	8.84	[illegible]	0.19	6.70	8.76	[illegible]	0.21	6.80	8.72	[illegible]	0.23	[illegible]	
R1	249,379	5.72	179	50	169	1011	16	20.0%	15.9%	B	0.25	3.99	21.12	0.76	25.11																
R2	290,166	6.66	353	50	335	1034	179	36.0%	15.9%	C	0.29	3.12	21.54	0.76	24.67																
		339.63																													

Elev 1 is the elevation at the furtherst location of the stream. Typically at a residence for developed area.
Elev 2 is the elevation at the location where the stream is gathering. Typically at a down stream end of a residence for developed area.
End Elevation is the elevation at the discharge point.

			#of Hous	Houses/acre
Sub of "A"	216592	4.97	20	4.02
Sub of "B"	413913	9.50213	40	4.21
D1	454,937	10.44	33	3.16

Assumed value to determine Tt-#
Velocity and W data were outside of the Figure
Data calculated in another sheet divided by 2
The flow will overtop 6" curb.
Bypass flows from upstream basin is added.
Curb inlet at Sag
Grated Inlet-Orifice and the flow travels on a brow ditch. Used Hydrology feature of AutoCad to run cacls
Grated Inlet-Weir

Hydraulic analysis of Sub Basins

This sheet calculate the inlet capacity and the bypass flows at each inlet.
The bypass, if any, will be added to the gutter flow of down stream basin under the "Q" columns of sheet "Basin Data-Tt"

			1-year						5-year			
Drainage Zone	Area ACRE	C	Tc MIN	Q Basins CFS	Q Basin CFS	Q inlet CFS	Inlet Cap CFS	Bypass CFS	Tc MIN	Q Basins CFS	Q Basin CFS	Q inlet CFS
X1 thru X9 + A	146.53	0.47	30.14	57.50	1.45	8.62	19.27	0.00	26.59	81.06	2.05	23.83
X2 thru X9+A	142.83	0.48	29.00	57.54	0.91	16.87	9.71	7.16	25.45	81.38	1.28	33.81
X3+X6+X9	17.27	0.48	19.13	9.10	1.37	1.37	3.98	0.00	18.57	12.07	1.81	2.33
X4+A	60.29	0.47	26.17	25.72	2.44	4.93	6.33	0.00	23.22	36.12	3.43	14.26
X5+X7+X8	63.02	0.48	18.08	34.79	4.29	18.21	2.24	15.96	17.74	45.78	5.65	29.35
X6+X9/2	9.64	0.48	18.20	5.30	2.53	2.53	4.68	0.00	17.58	7.04	3.36	3.88
X7	18.10	0.48	11.36	13.48	13.48	13.48	9.76	3.72	11.07	17.81	17.81	17.81
X8	37.15	0.48	16.79	21.51	21.51	21.51	11.32	10.20	16.49	28.29	28.29	28.29
X9	10.07	0.48	14.69	6.35	6.35	6.35	6.81	0.00	14.25	8.42	8.42	8.42
A	54.56	0.47	14.22	34.42	34.42	34.42	31.10	3.32	13.84	45.54	45.54	45.54
B1-4+B6	99.95	0.47	25.38	43.82	2.84	5.95	18.30	0.00	19.42	57.74	3.74	19.38
B2-4+B6	93.47	0.48	23.31	43.44	5.59	5.92	2.81	3.11	15.32	57.24	7.36	18.59
B3+B4+B6	81.45	0.48	22.25	39.33	22.96	25.63	25.30	0.33	21.82	51.77	30.23	37.88
B4+B6	33.90	0.48	22.00	16.49	6.55	8.67	6.00	2.67	21.58	21.70	8.62	13.97
B5	3.09	0.45	12.15	2.07	2.07	2.07	2.85	0.00	11.98	2.71	2.71	2.71
B6	20.44	0.48	16.67	11.88	11.88	11.88	9.76	2.12	16.26	15.70	15.70	15.70
C1+C3+C4	42.36	0.47	23.38	19.46	2.86	14.20	14.23	0.00	22.89	25.64	3.76	21.15
C2+D1+D2	31.58	0.47	17.74	17.41	1.84	2.78	5.30	0.00	17.35	22.96	2.42	5.02
C3	17.52	0.47	15.32	10.54	10.54	10.54	5.90	4.64	15.25	13.74	13.74	13.74
C4	18.62	0.48	15.72	11.25	11.25	11.25	4.54	6.71	15.52	14.75	14.75	14.75
D1+D2	28.25	0.48	15.90	16.81	6.22	6.22	5.28	0.94	15.55	22.16	8.19	8.20
D2	17.80	0.48	12.43	12.51	12.51	12.51	12.51	0.00	12.18	16.48	16.48	16.48
E	12.91	0.47	15.55	7.69	7.69	7.69	14.40	0.00	15.43	10.04	10.04	10.04
M1	1.79	0.25	23.57	0.43					23.57	0.56		
M2	38.86	0.47	9.07	32.78					9.01	42.78		
R1	5.72	0.25	25.11	1.33					25.11	1.73		
R2	6.66	0.29	24.67	1.82					24.67	2.36		

Inlet Cap CFS	Bypass CFS	Tc MIN	Q Basins CFS	10-year Q Basin CFS	Q inlet CFS	Inlet Cap CFS	Bypass CFS	Tc MIN	Q Basins CFS	50-year Q Basin CFS	Q inlet CFS	Inlet Cap CFS	Bypass CFS
27.48	0.00	25.56	95.94	2.42	36.16	36.21	0.00	24.91	123.55	3.12	66.68	67.06	0.00
12.02	21.79	24.42	96.44	1.52	46.98	13.24	33.74	23.83	124.10	1.96	77.63	14.07	63.55
4.26	0.00	18.35	14.03	2.11	3.27	4.32	0.00	18.07	17.95	2.70	6.94	4.50	2.44
8.42	5.84	22.34	42.72	4.06	20.58	9.32	11.27	21.80	54.98	5.22	33.19	10.87	22.32
2.66	26.69	17.42	53.44	6.60	36.85	2.66	34.19	16.64	69.74	8.61	53.57	2.66	50.91
4.80	0.00	17.34	8.20	3.92	5.03	4.98	0.05	17.00	10.52	5.02	7.25	5.23	2.02
10.35	7.46	10.81	20.88	20.88	20.88	11.07	9.81	10.57	26.83	26.83	26.83	11.81	15.03
12.06	16.23	16.20	33.01	33.01	33.01	12.56	20.45	15.51	43.01	43.01	43.01	13.07	29.94
7.39	1.03	14.09	9.79	9.79	9.79	7.56	2.23	13.86	12.53	12.53	12.53	8.08	4.45
31.10	14.44	13.60	53.14	53.14	53.14	31.10	22.04	13.27	68.40	68.40	68.40	31.10	37.29
19.42	0.00	19.15	67.14	4.35	28.24	28.25	0.00	19.02	86.07	5.58	46.12	37.77	8.35
2.95	15.64	15.23	66.47	8.55	26.88	2.99	23.89	15.15	85.20	10.96	43.72	3.18	40.55
26.65	11.23	21.62	60.09	35.08	46.07	27.74	18.33	21.24	77.01	44.96	62.72	29.96	32.76
6.32	7.65	21.39	25.18	10.00	17.55	6.56	10.99	21.02	32.26	12.81	24.49	6.72	17.76
3.25	0.00	11.90	3.14	3.14	3.14	3.55	0.00	11.68	4.03	4.03	4.03	4.54	0.00
10.35	5.35	16.07	18.26	18.26	18.26	10.71	7.55	15.69	23.48	23.48	23.48	11.81	11.68
21.21	0.00	22.16	30.21	4.43	26.05	26.06	0.00	22.25	38.17	5.60	35.39	26.66	8.73
5.30	0.00	17.10	26.74	2.82	6.43	6.53	0.00	16.93	34.09	3.60	9.35	9.36	0.00
6.28	7.46	14.99	16.03	16.03	16.03	6.54	9.49	14.71	20.55	20.55	20.55	7.19	13.36
4.82	9.92	15.44	17.07	17.07	17.07	4.94	12.13	15.33	21.72	21.72	21.72	5.29	16.43
5.60	2.60	15.35	25.79	9.54	9.54	5.94	3.60	15.23	32.83	12.14	12.14	6.39	5.75
16.48	0.00	12.06	19.14	19.14	19.14	19.14	0.00	12.05	24.26	24.26	24.26	24.26	0.00
15.07	0.00	15.38	11.62	11.62	11.62	15.51	0.00	15.27	14.78	14.78	14.78	16.60	0.00
		23.57	0.65					23.57	0.82				
		8.96	49.54					8.87	63.18				
		25.11	2.00					25.11	2.53				
		24.67	2.73					24.67	3.45				

X2

Figure 2-3

$Q_5 = 16.9 \rightarrow 8.5$
$Q_{10} = 52.5 \rightarrow 26.3$
$Q_5 = 37.4 = 18.7$

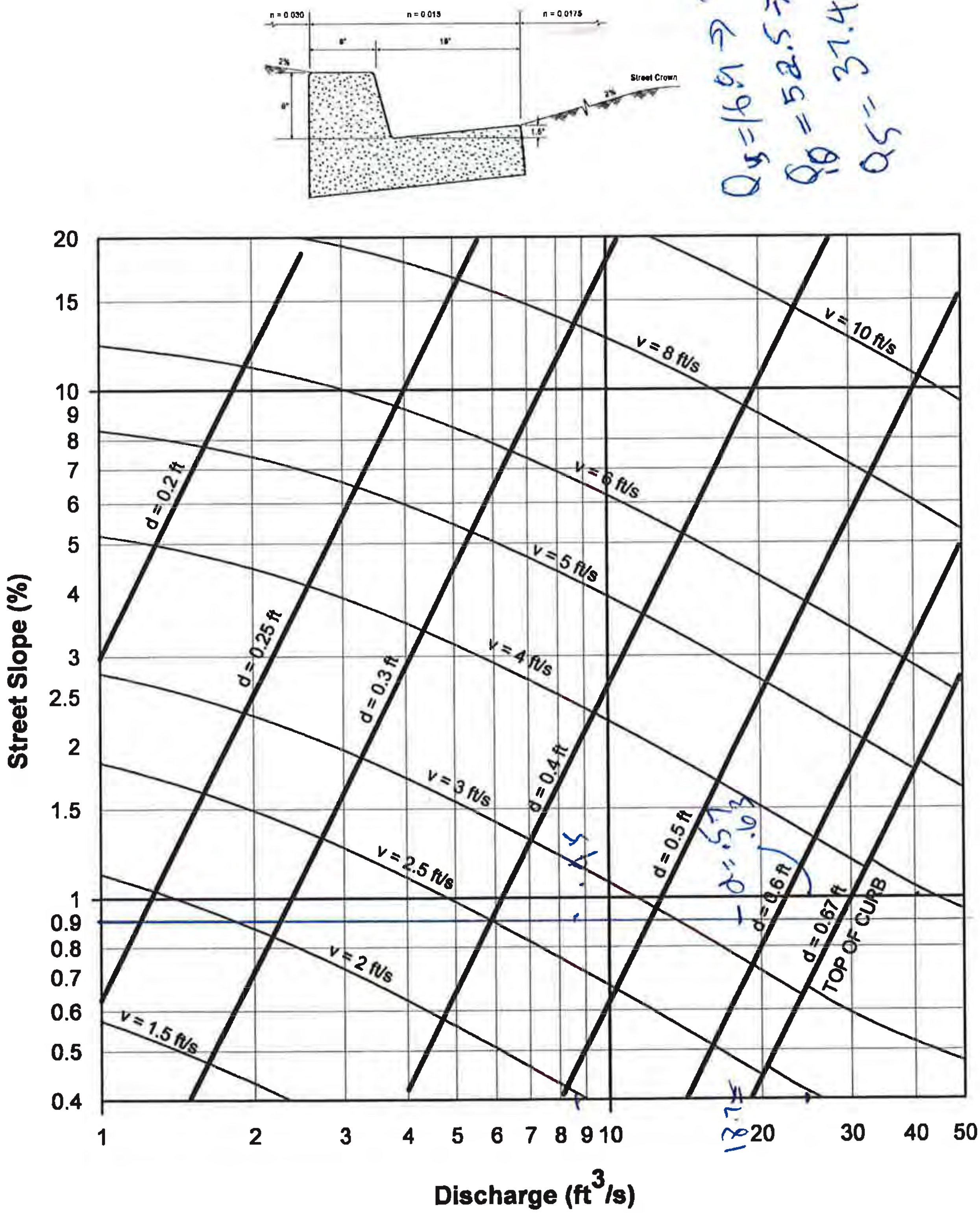


Figure 2-3 8-inch Gutter and Roadway Discharge-Velocity Chart

11/6/06

Basin X#2

Sunset Cliffs Natural Park

Figure 2-2

$Q_1 = 16.9 \rightarrow 8.5$

$Q_5 : 52.5 \rightarrow 26.3$

Replaced W/ 8" curb for overflow analysis

Q_1, Q_5 & Q_{10} is less than 1 CFS

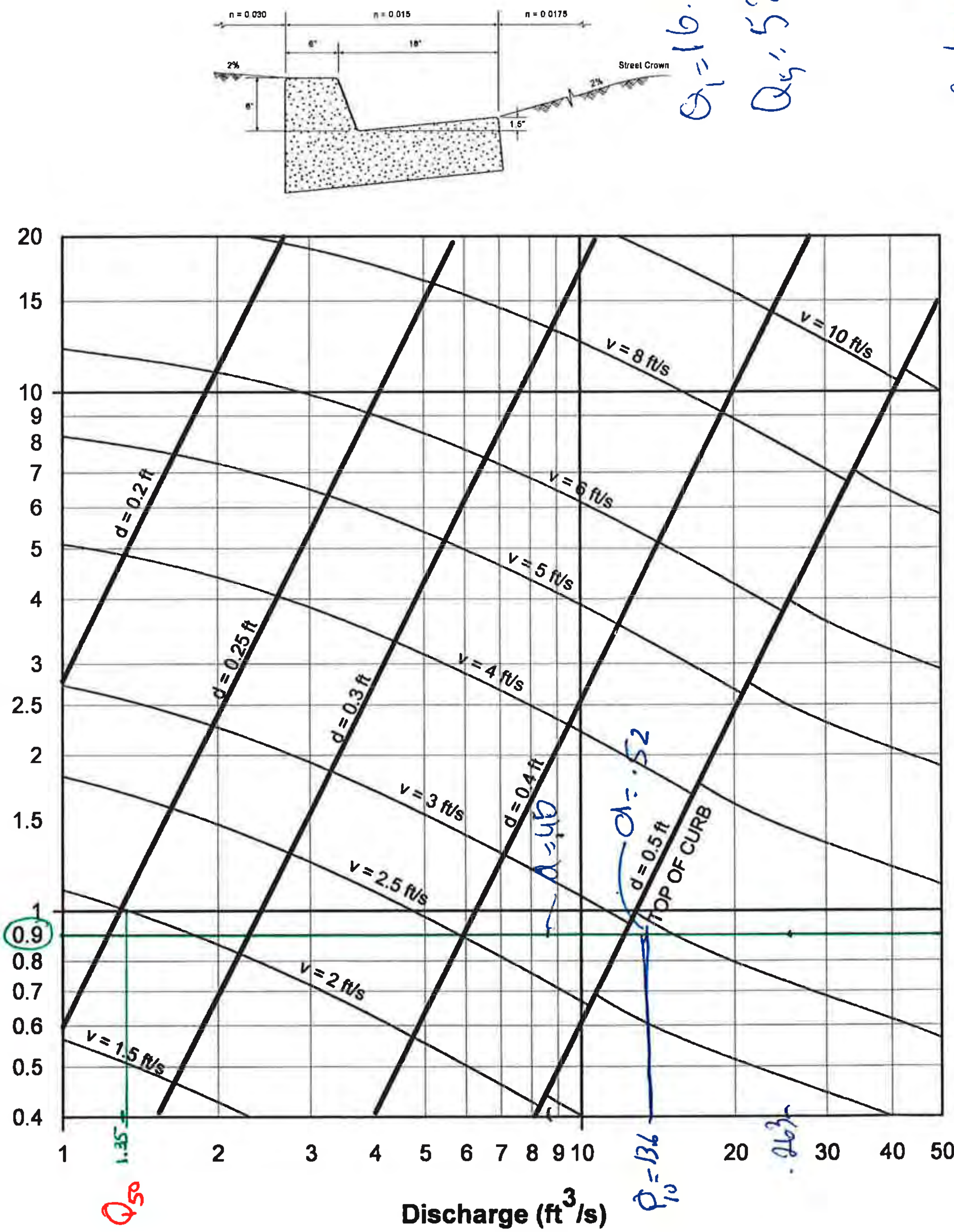


Figure 2-2 6-inch Gutter and Roadway Discharge-Velocity Chart

X81

Figure 2-2

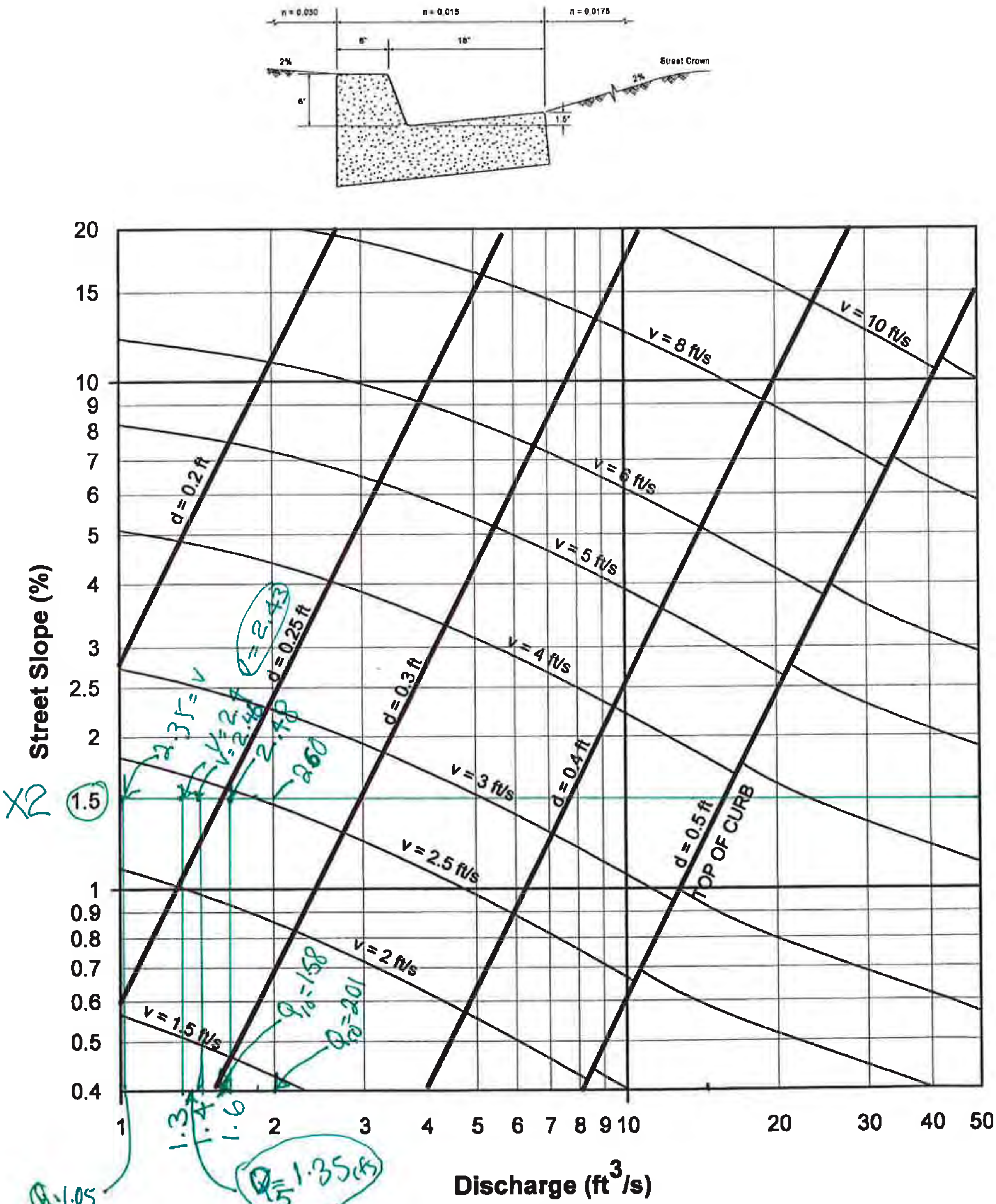


Figure 2-2 6-inch Gutter and Roadway Discharge-Velocity Chart

X-3

Figure 2-2

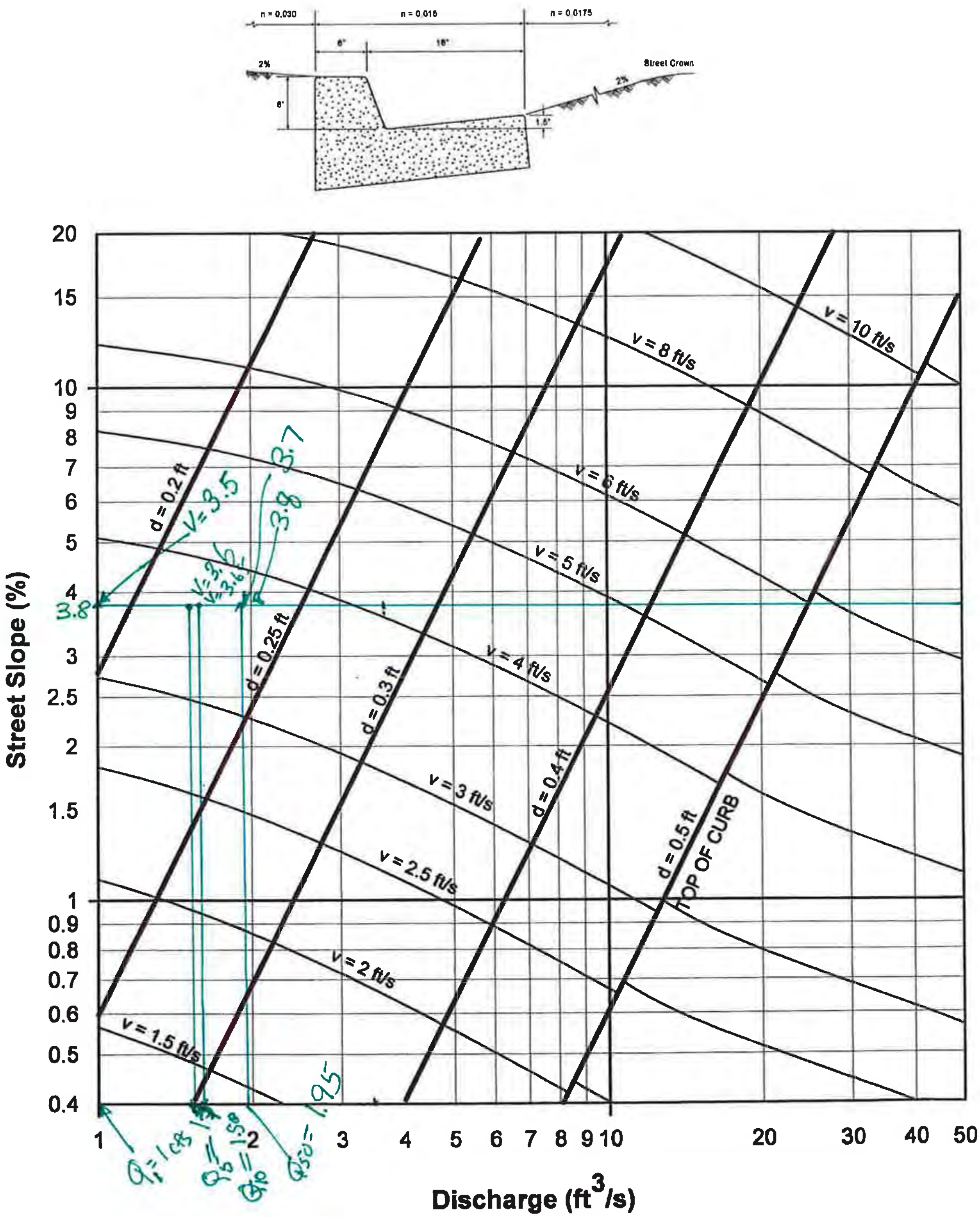


Figure 2-2 6-inch Gutter and Roadway Discharge-Velocity Chart

X-4. combined flows of X4 & A 12/6/06

Figure 2-3

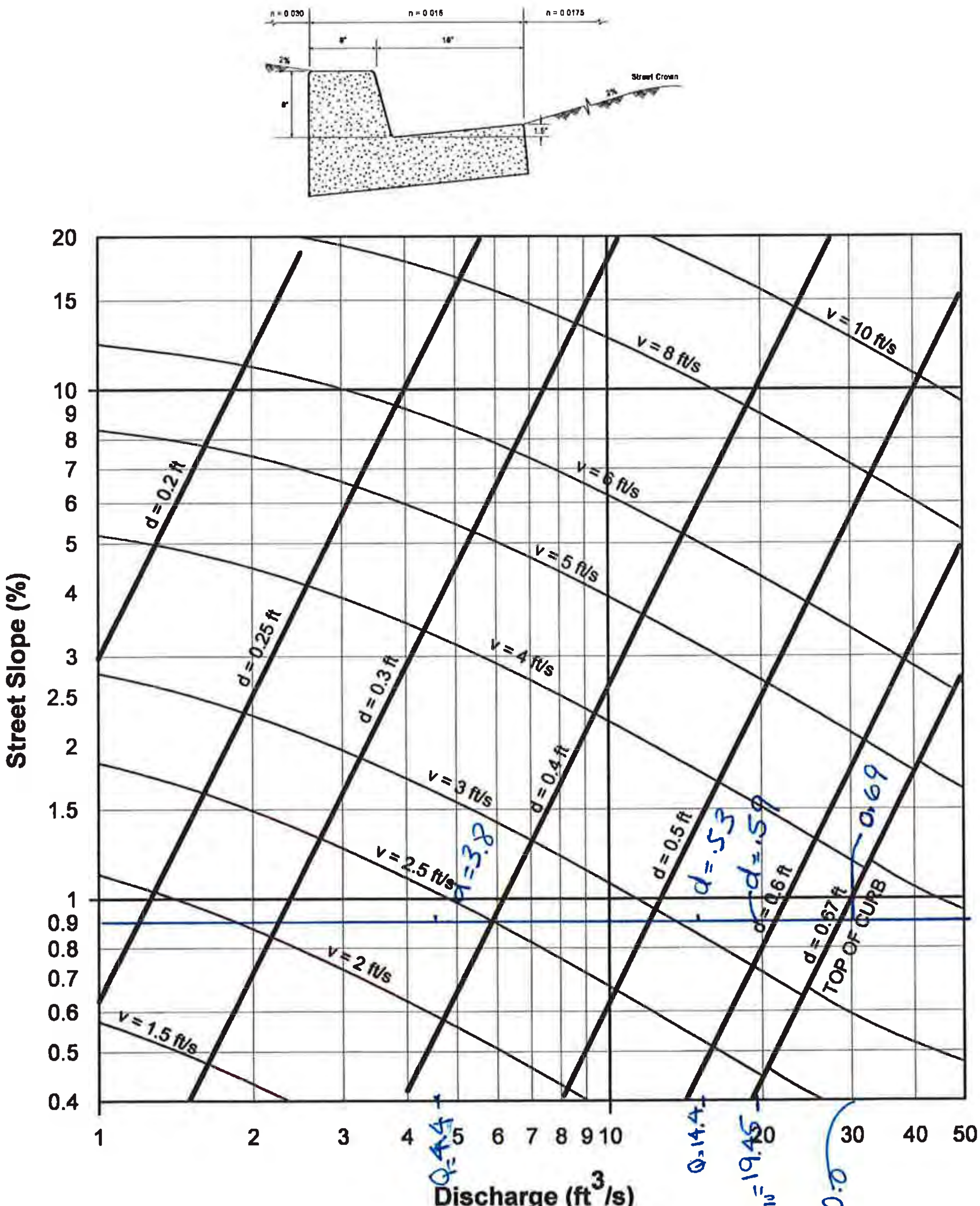


Figure 2-3 8-inch Gutter and Roadway Discharge-Velocity Chart

X-4

Figure 2-2

Blue text: combined flows of A+X4
$Q_1 = 4.4$, $Q_5 = 14.10$, $Q_{10} = 19.45$, $Q_{50} = 30.0$ cfs

0.55 ≈ ½ ftc

Note: Replaced w/ Fig 2-3

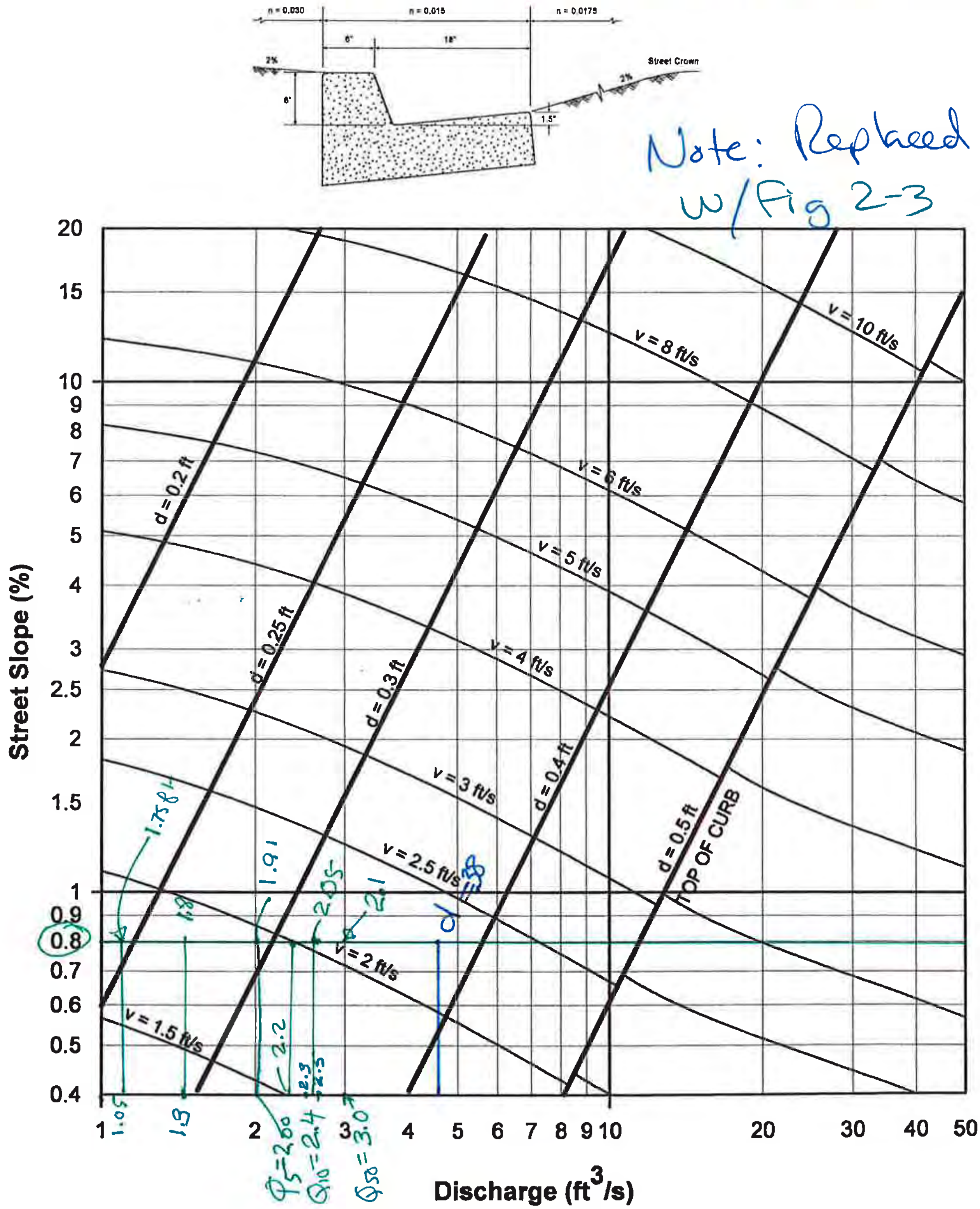


Figure 2-2 6-inch Gutter and Roadway Discharge-Velocity Chart

X5

Figure 2-2

$Q_1 = 18.21$ cfs → 9.1
$Q_5 = 29.3$ cfs → 18.4
$Q_{10} = 36.8$ → 18.4 ovelo
Q_{50} → 53.6 → 28.3

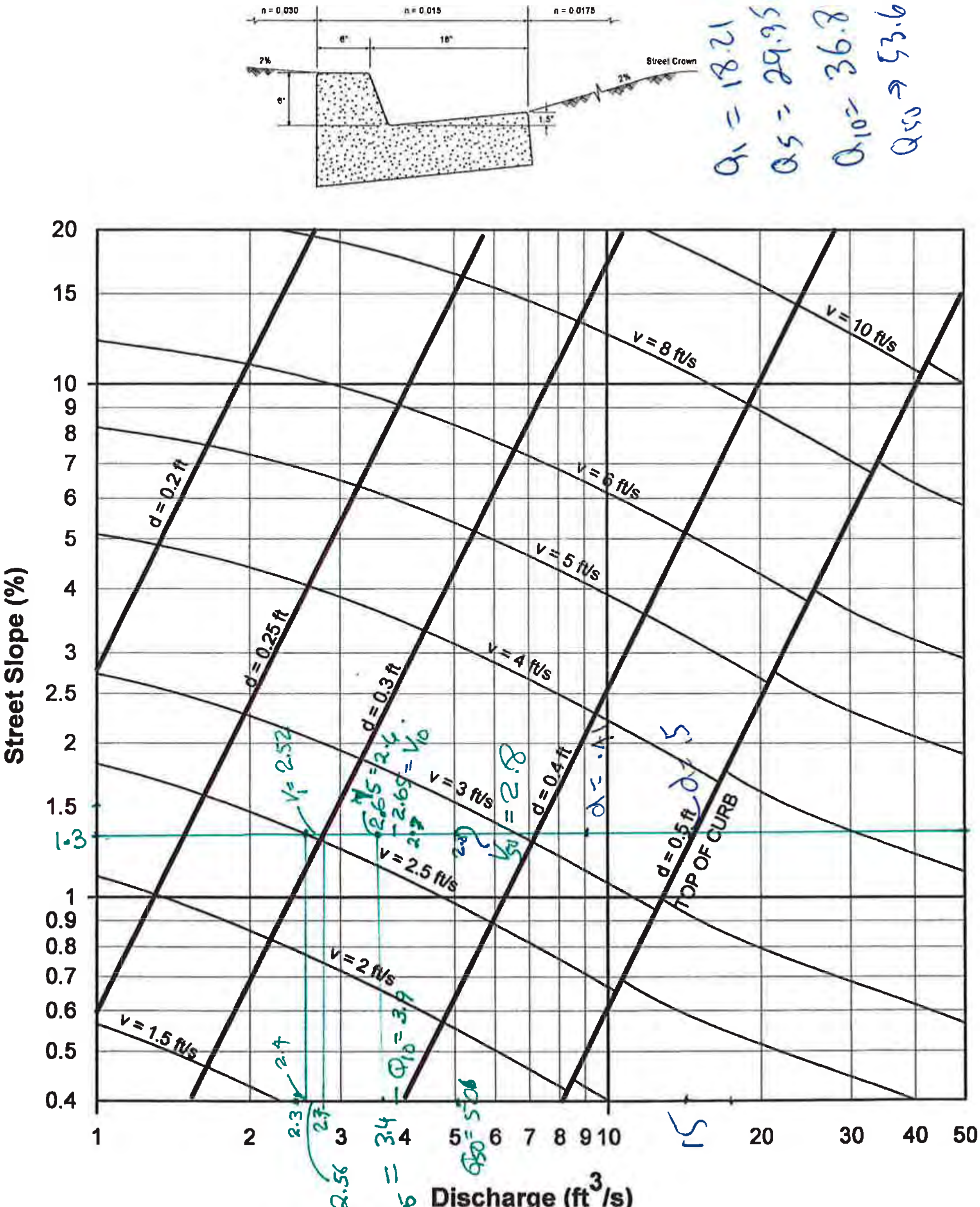


Figure 2-2 6-inch Gutter and Roadway Discharge-Velocity Chart

X6

Figure 2-2

Accumulative data

$Q_{10} = 5.03 \rightarrow 2.57$ cfs

$Q_{50} = 7.05 \rightarrow 3.65$

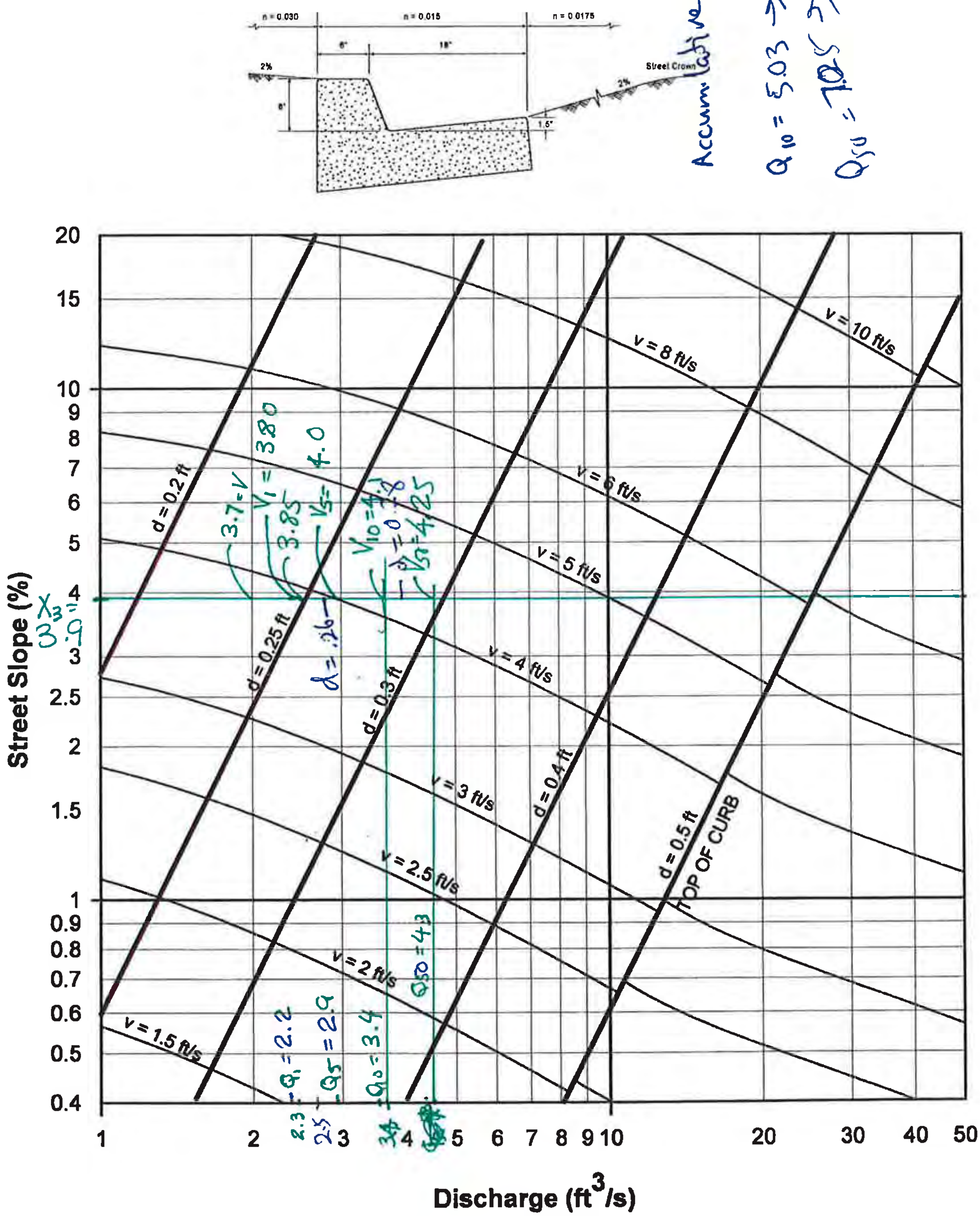


Figure 2-2 6-inch Gutter and Roadway Discharge-Velocity Chart

X7

Figure 2-2

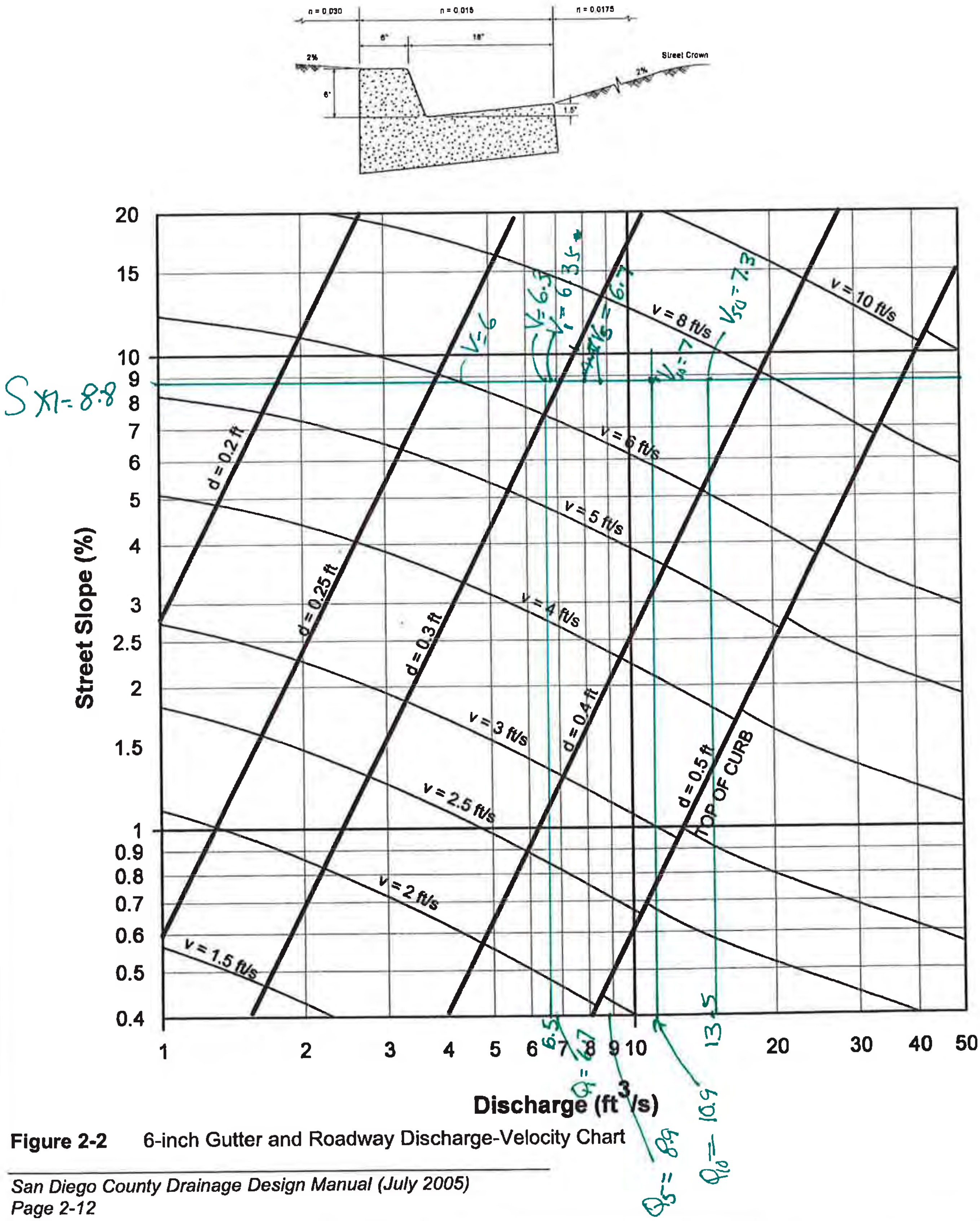
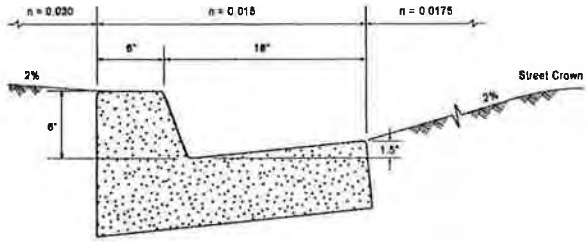


Figure 2-2 6-inch Gutter and Roadway Discharge-Velocity Chart

X8

Figure 2-2



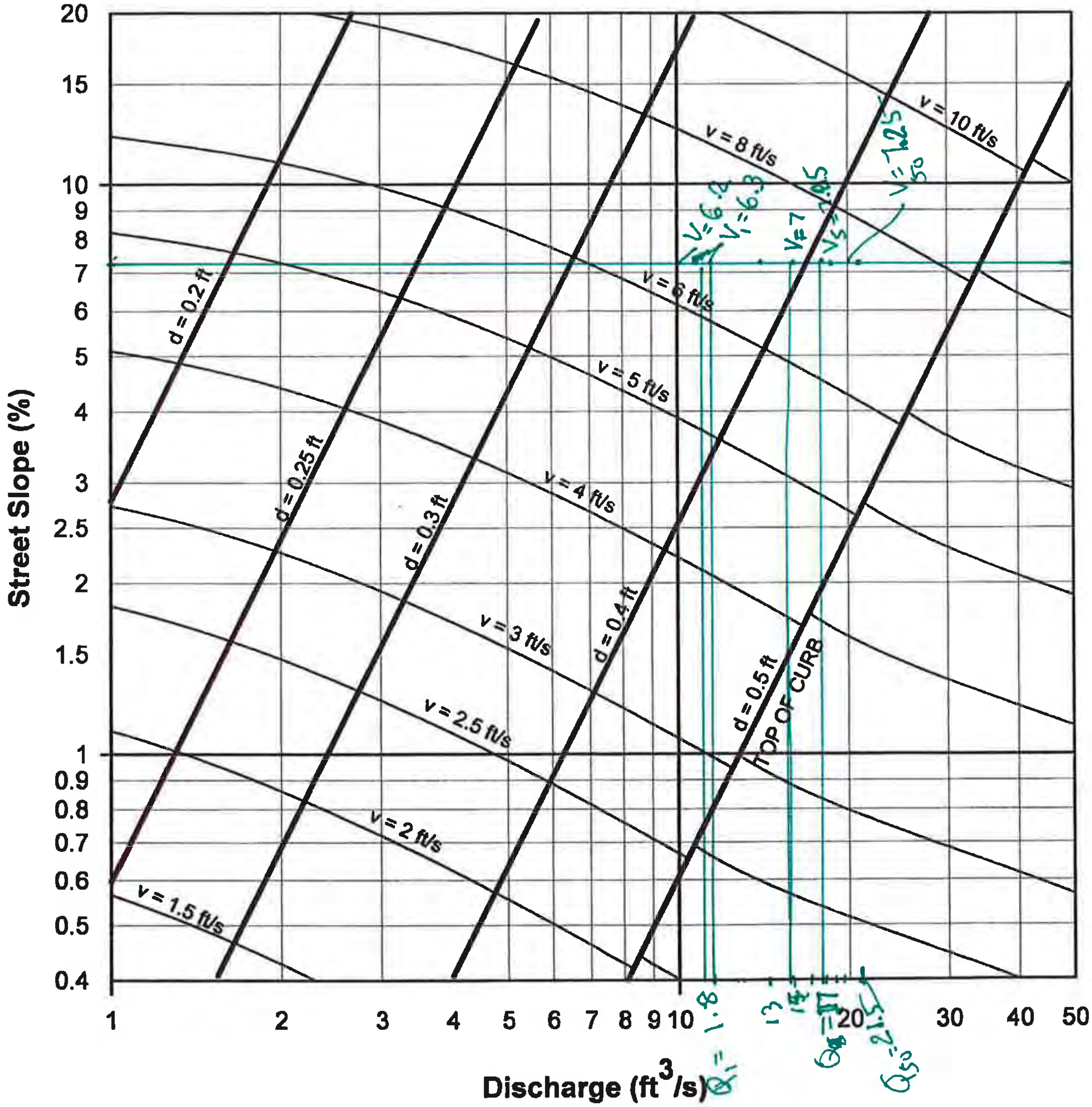


Figure 2-2 6-inch Gutter and Roadway Discharge-Velocity Chart

X9

Figure 2-2

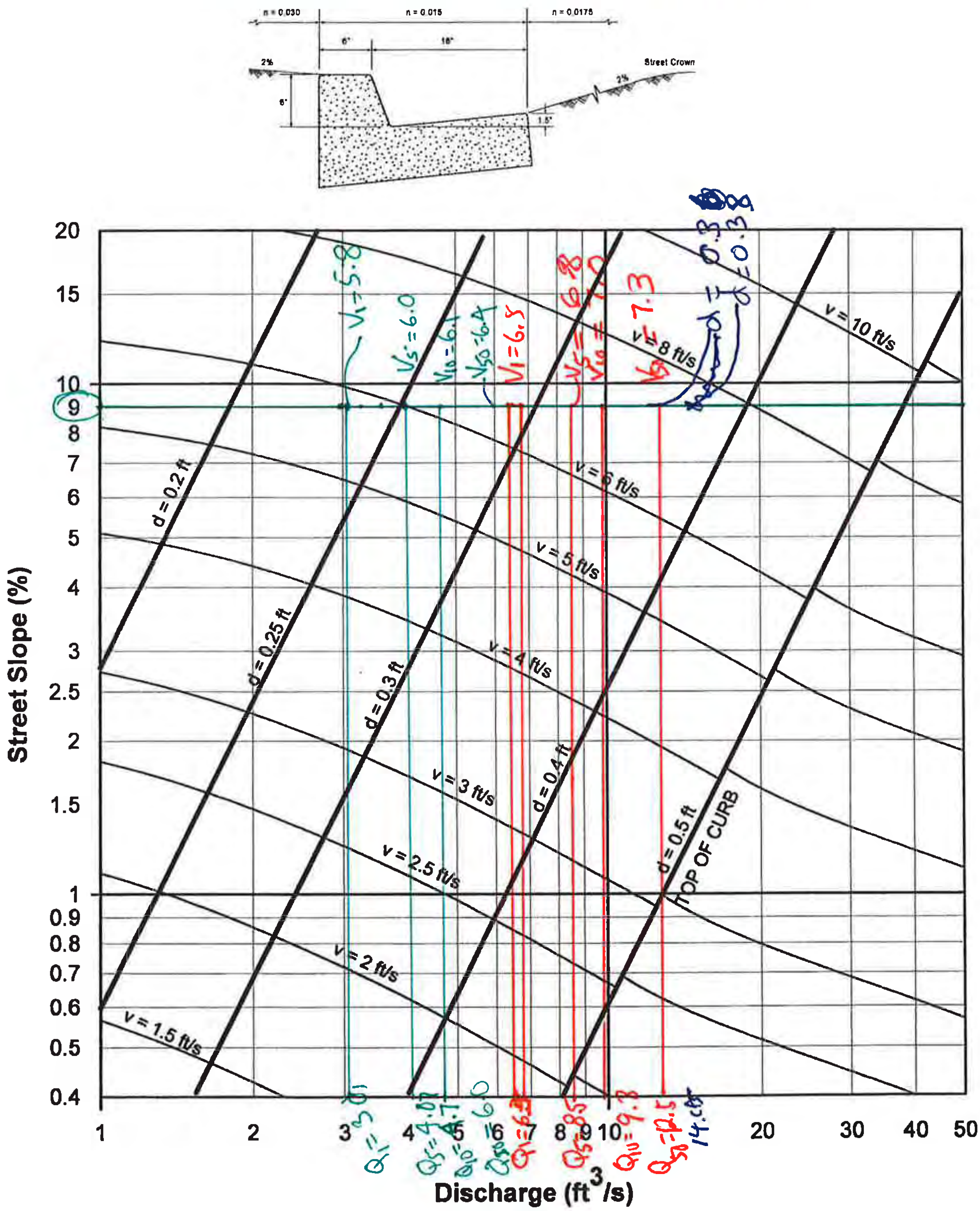


Figure 2-2 6-inch Gutter and Roadway Discharge-Velocity Chart

A

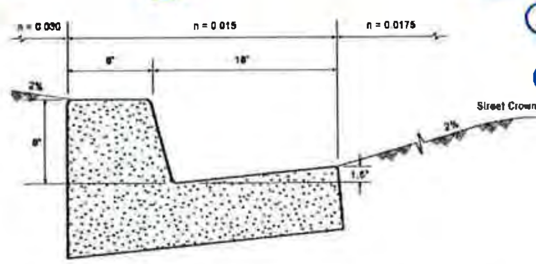
Figure 2-3

The Q is much greater than the rated Q for T.O.C. @ $Q_1 = 34.4$

$Q_5 = 45.54$

$Q_{10} = 53.14$

$Q_{50} = 68.40$.



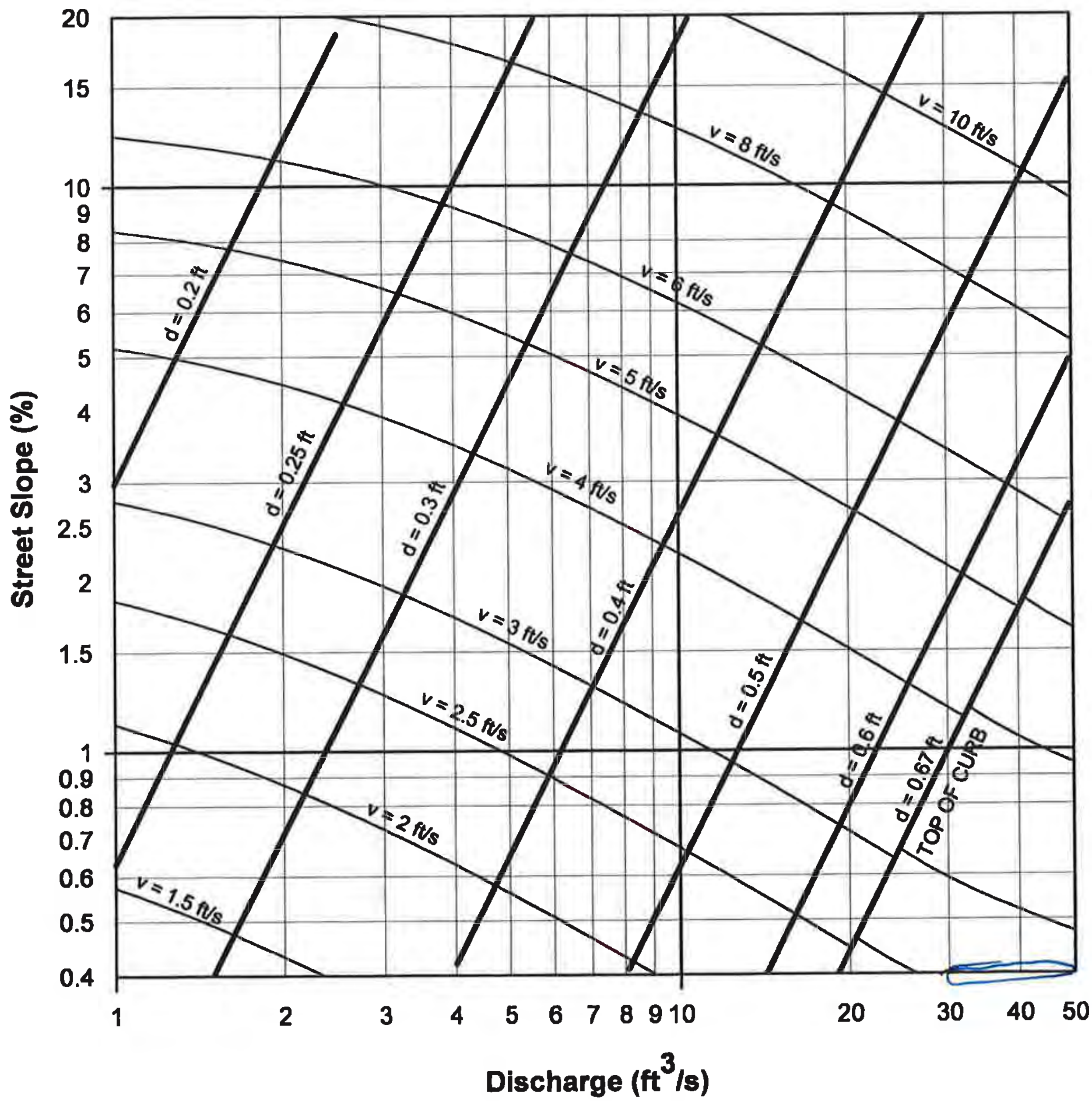


Figure 2-3 8-inch Gutter and Roadway Discharge-Velocity Chart

A

Figure 2-2

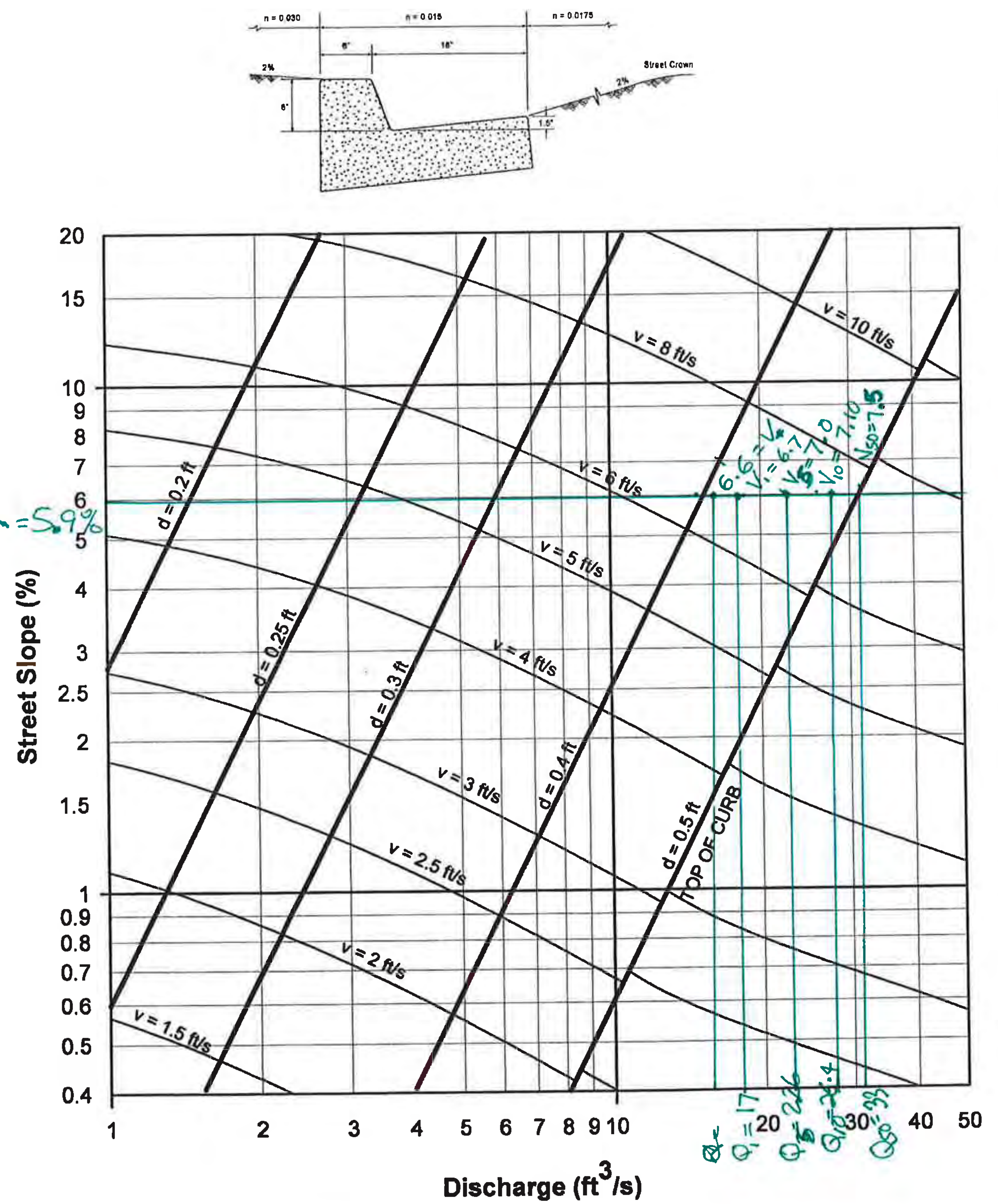


Figure 2-2 6-inch Gutter and Roadway Discharge-Velocity Chart

B1

Figure 2-2

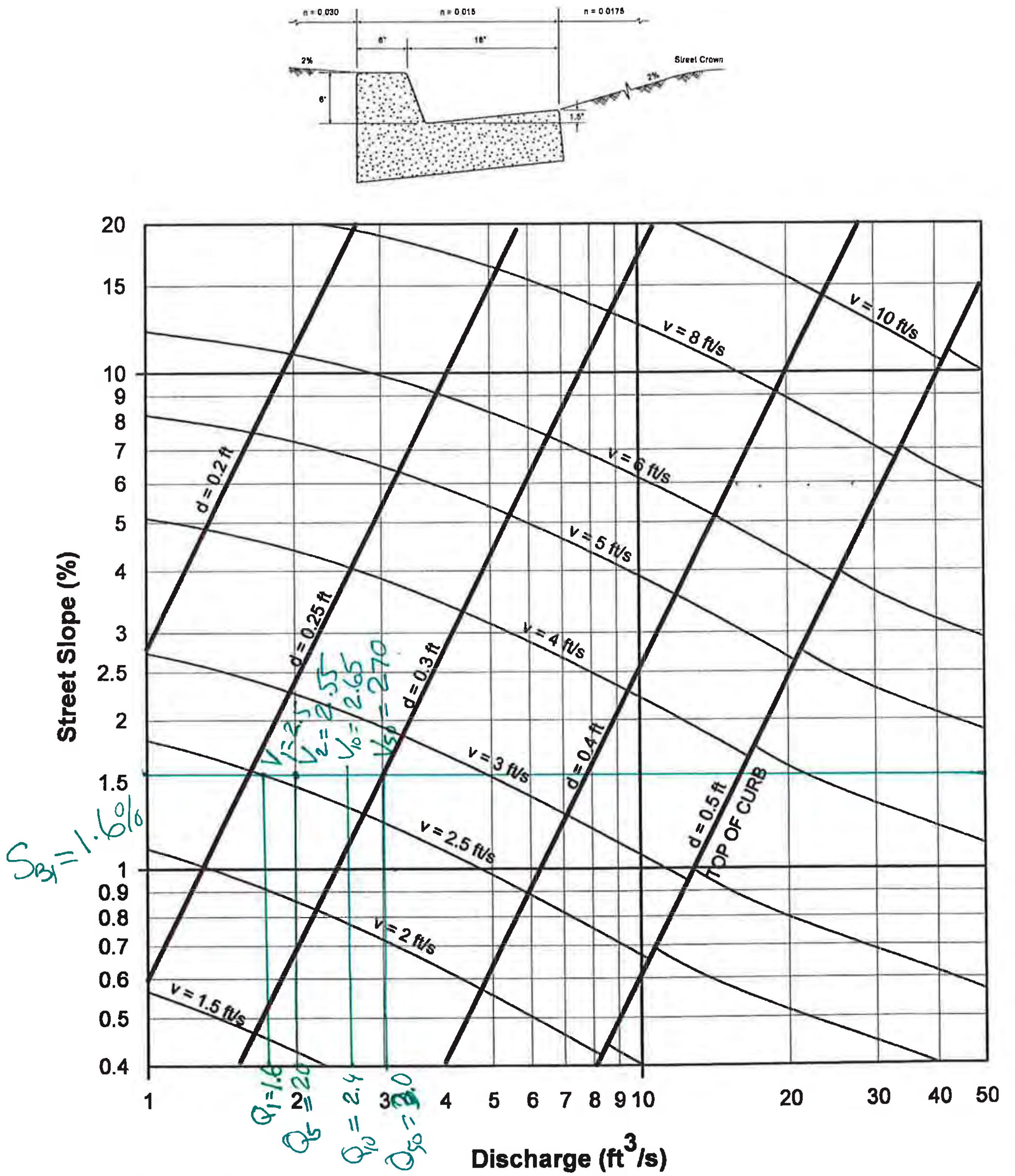


Figure 2-2 6-inch Gutter and Roadway Discharge-Velocity Chart

B2

Figure 2-2

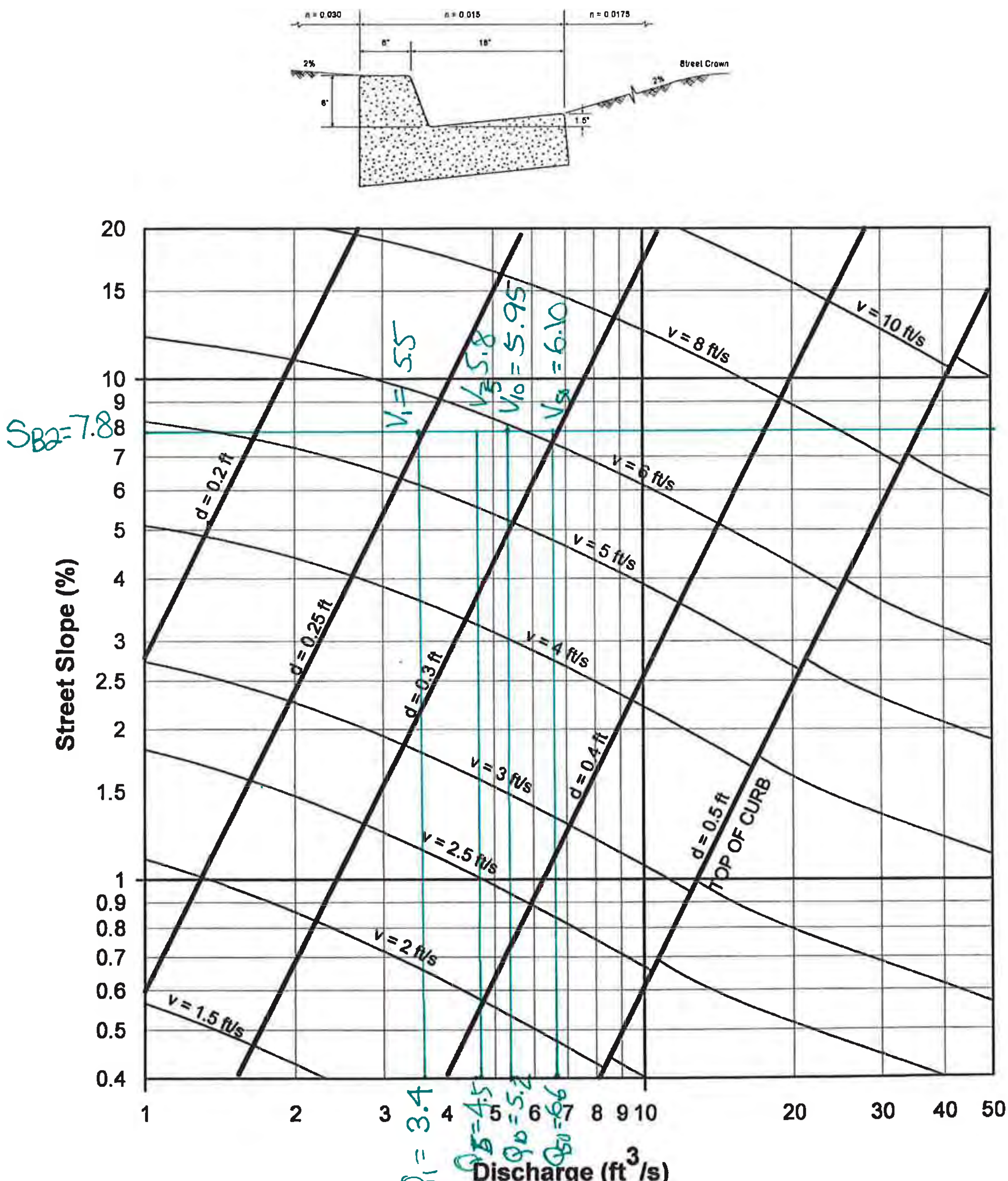


Figure 2-2 6-inch Gutter and Roadway Discharge-Velocity Chart

B3

Figure 2-2

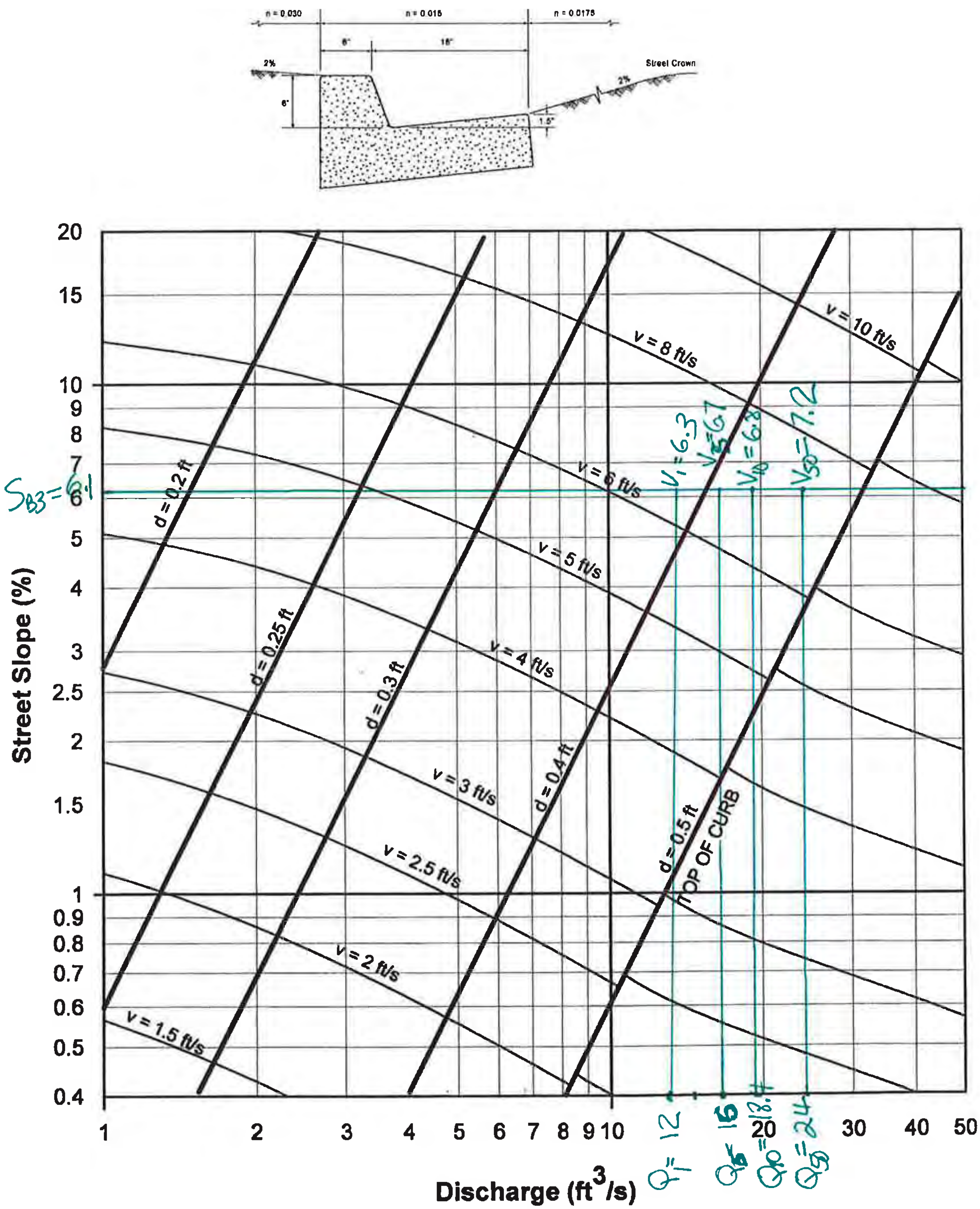


Figure 2-2 6-inch Gutter and Roadway Discharge-Velocity Chart

B4

Figure 2-2

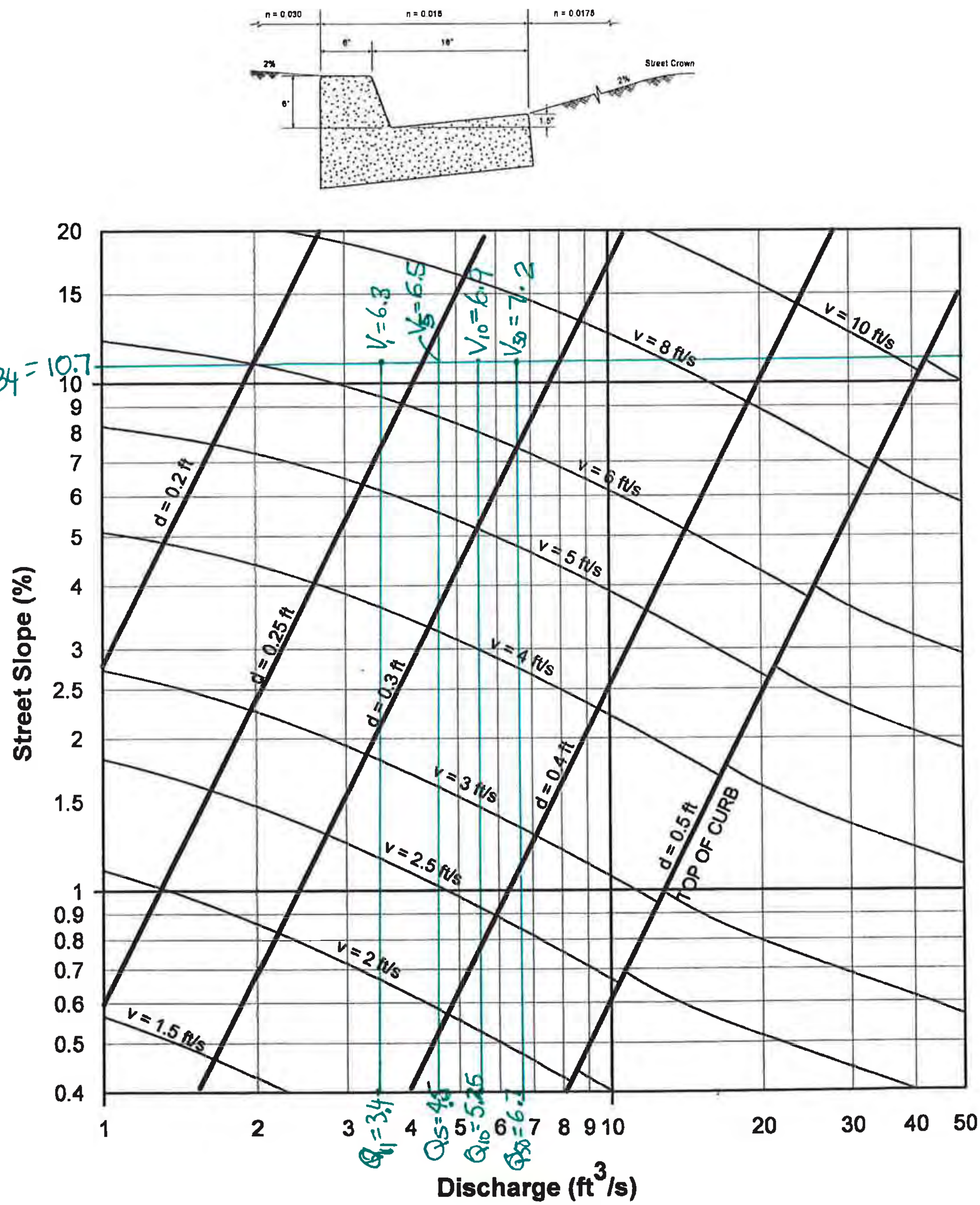


Figure 2-2 6-inch Gutter and Roadway Discharge-Velocity Chart

B5

Figure 2-2

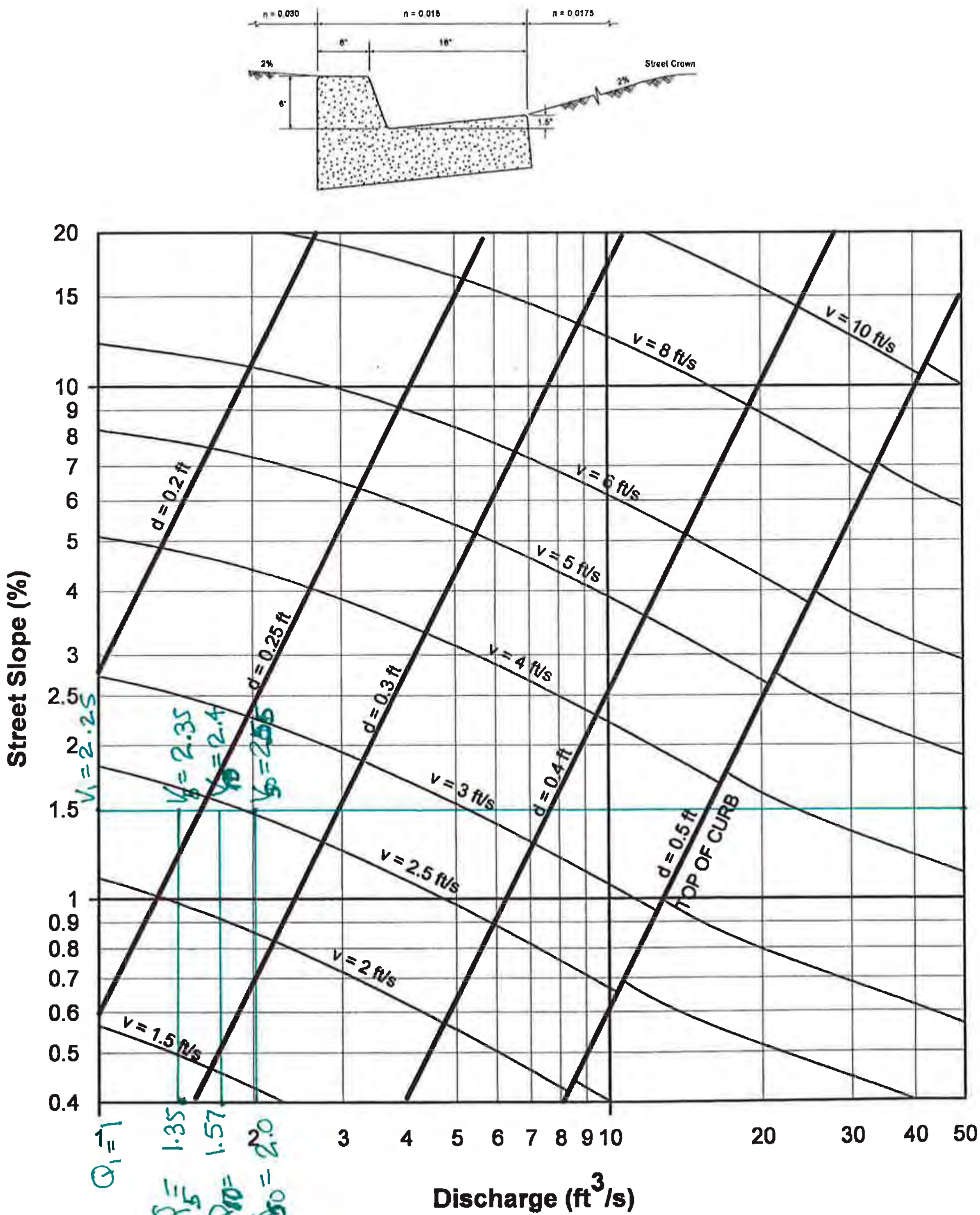


Figure 2-2 6-inch Gutter and Roadway Discharge-Velocity Chart

B6

Figure 2-2

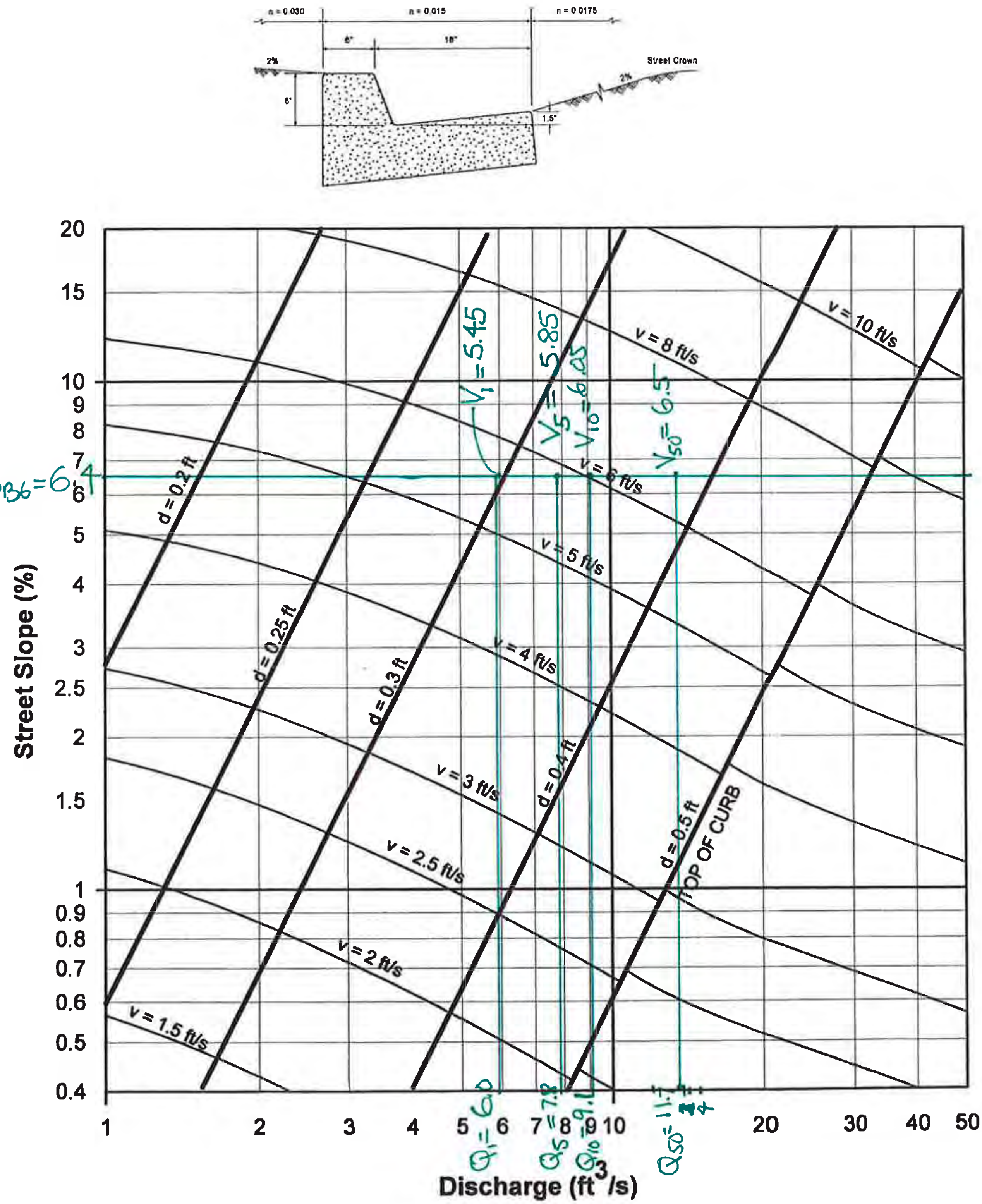


Figure 2-2 6-inch Gutter and Roadway Discharge-Velocity Chart

C1

Figure 2-2

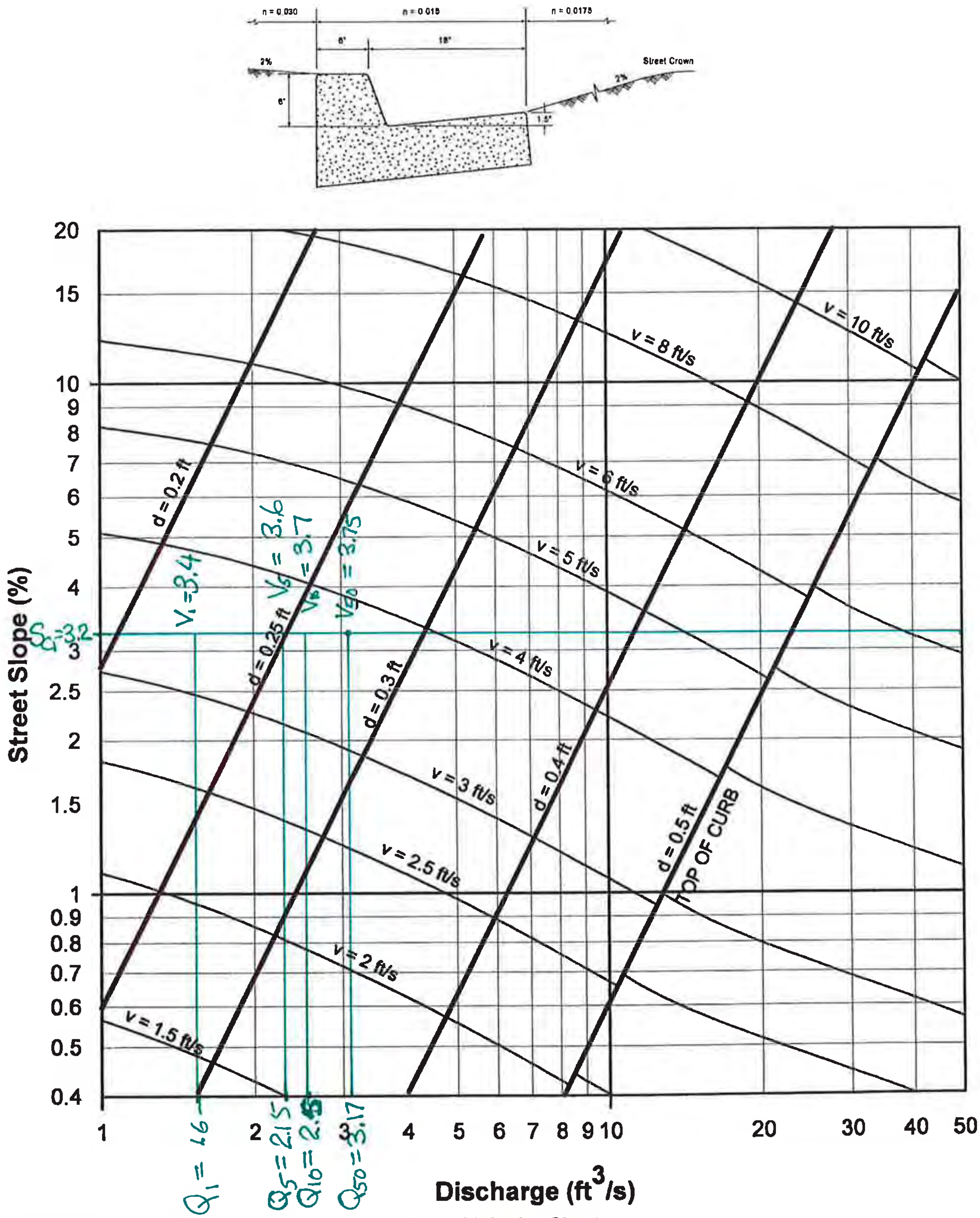


Figure 2-2 6-inch Gutter and Roadway Discharge-Velocity Chart

C2

Figure 2-2

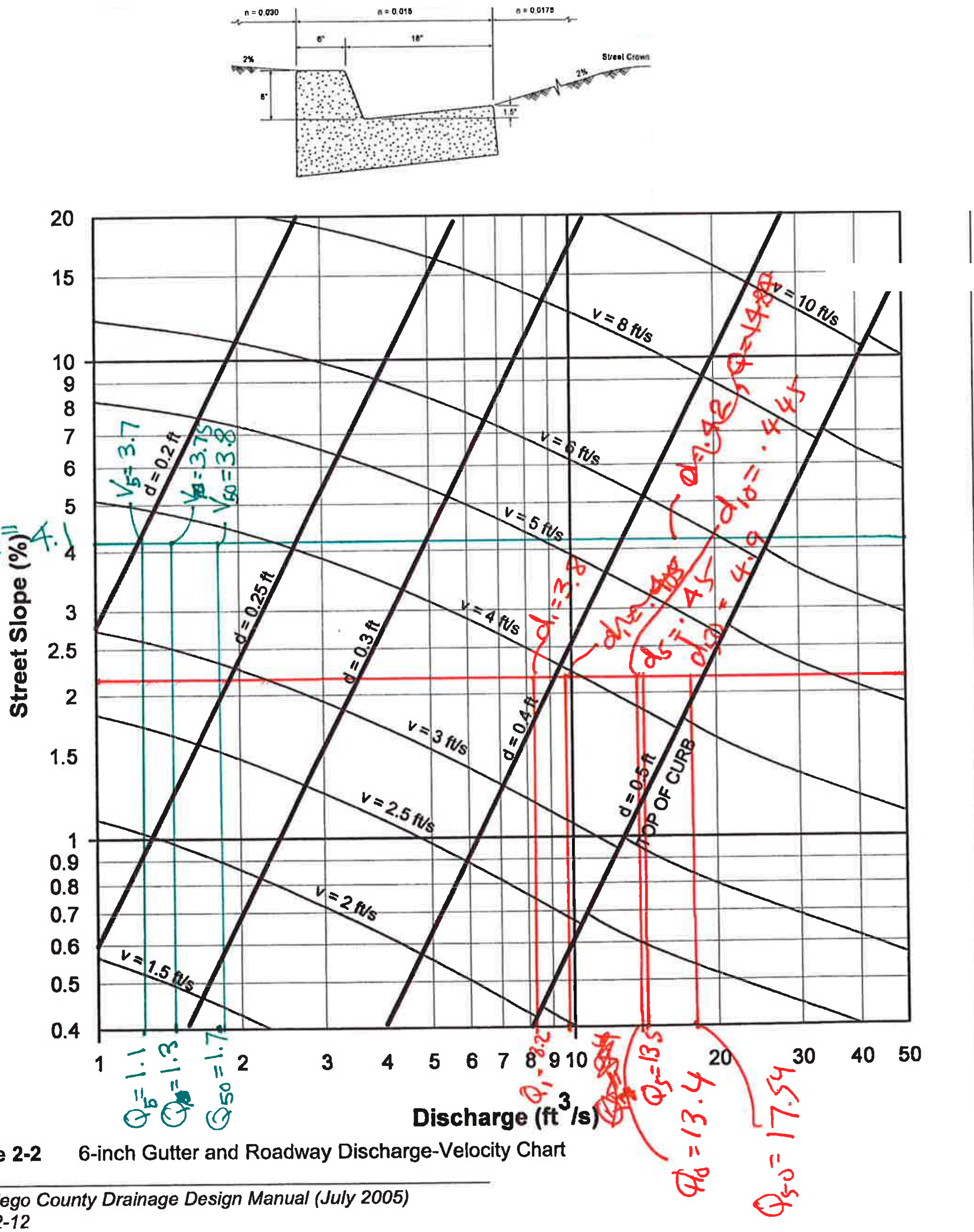


Figure 2-2 6-inch Gutter and Roadway Discharge-Velocity Chart

C3

Blue text is for analysis of the flow at the curb inlet. S ≈ 0.91%

Figure 2-2

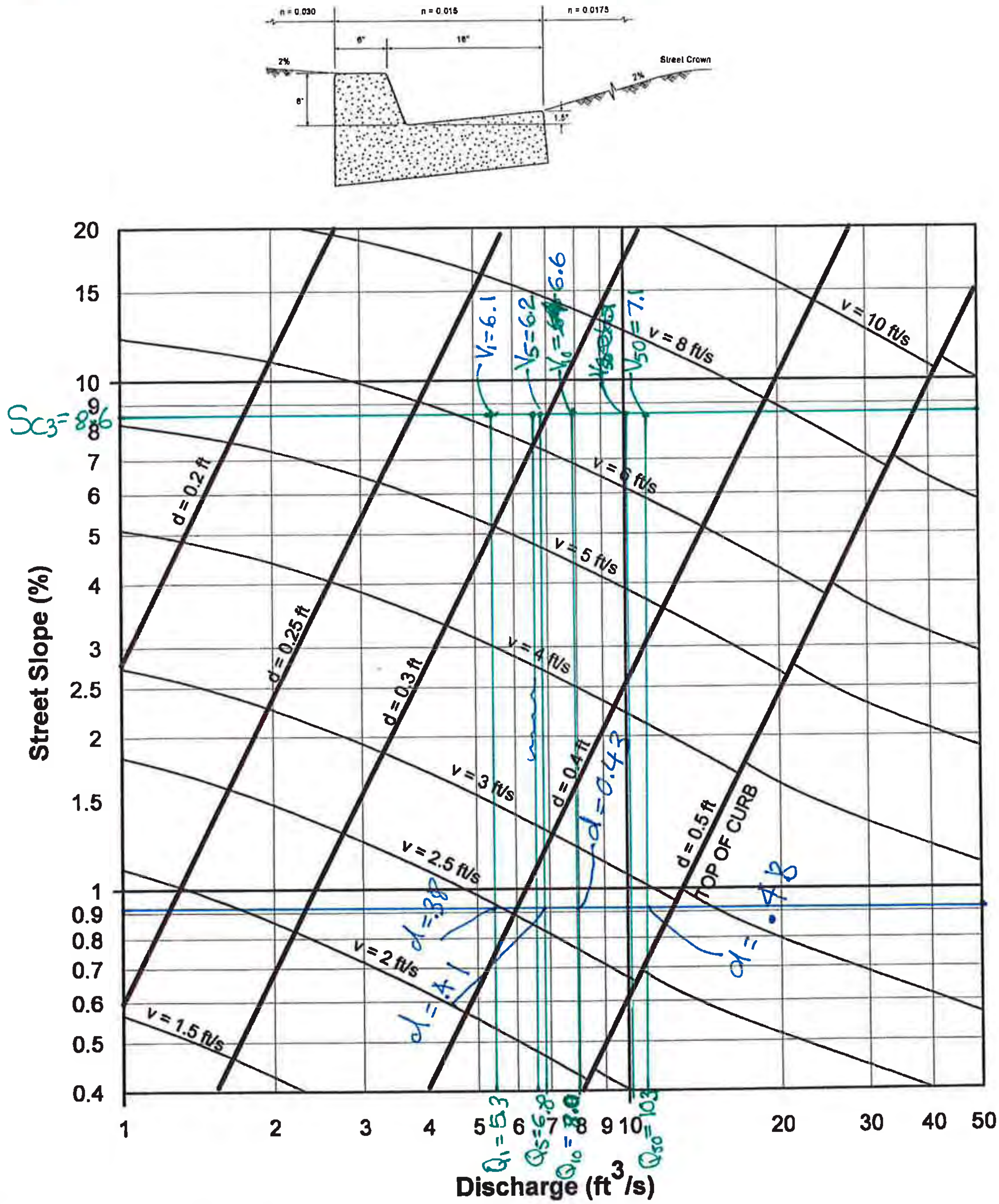


Figure 2-2 6-inch Gutter and Roadway Discharge-Velocity Chart

C4

Figure 2-2

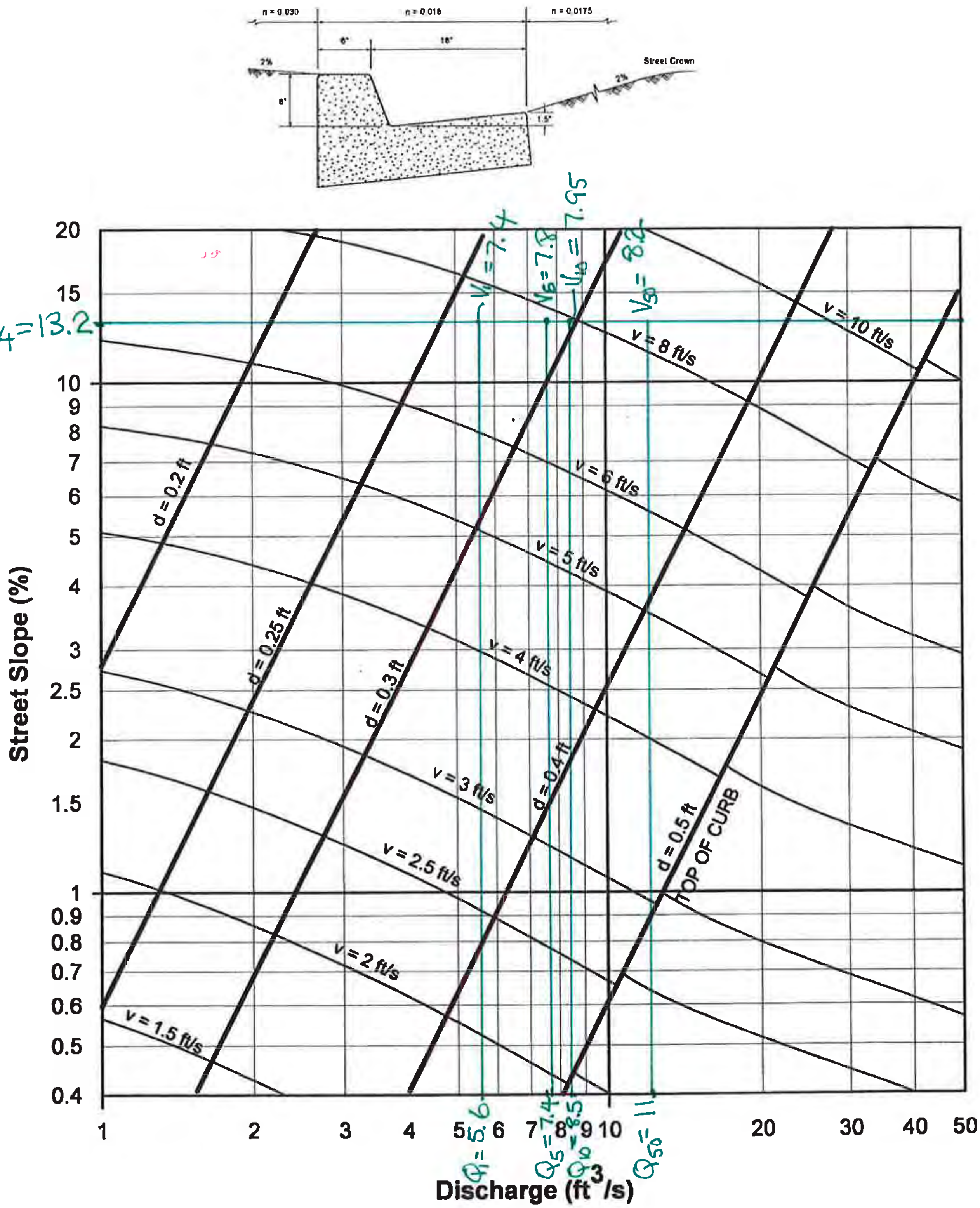


Figure 2-2 6-inch Gutter and Roadway Discharge-Velocity Chart

D1

Figure 2-2

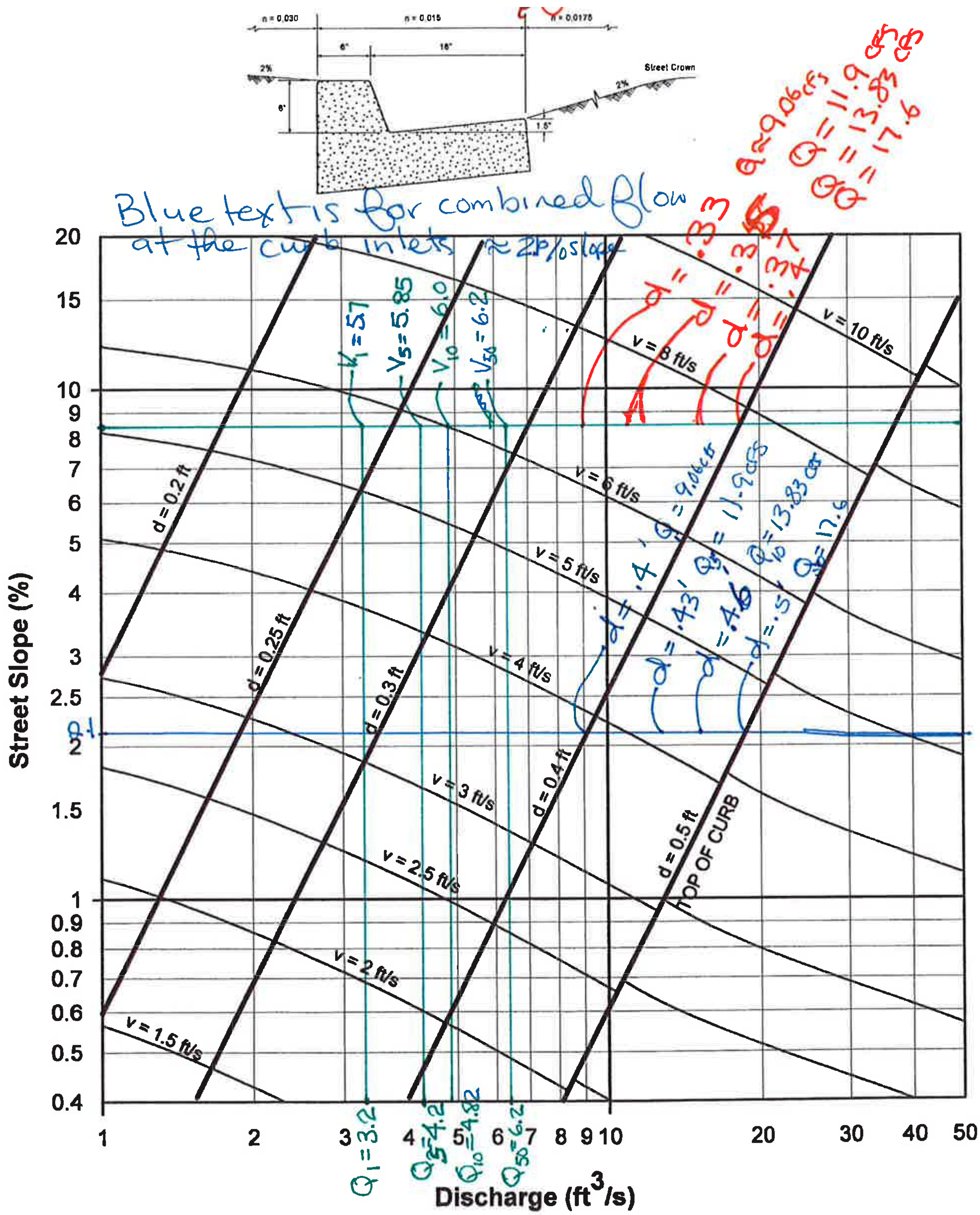


Figure 2-2 6-inch Gutter and Roadway Discharge-Velocity Chart

D2

Figure 2-2

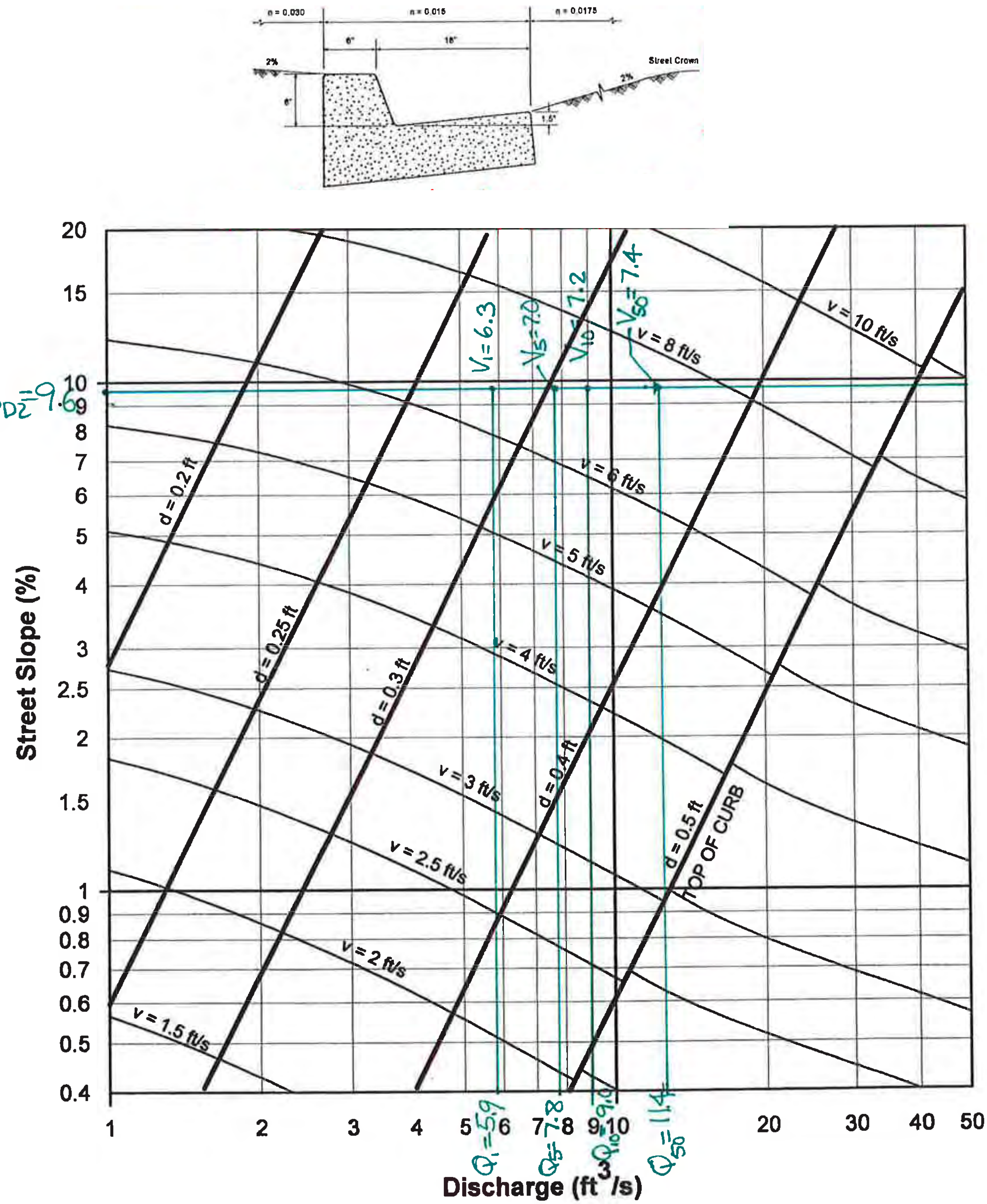


Figure 2-2 6-inch Gutter and Roadway Discharge-Velocity Chart

E

Figure 2-2

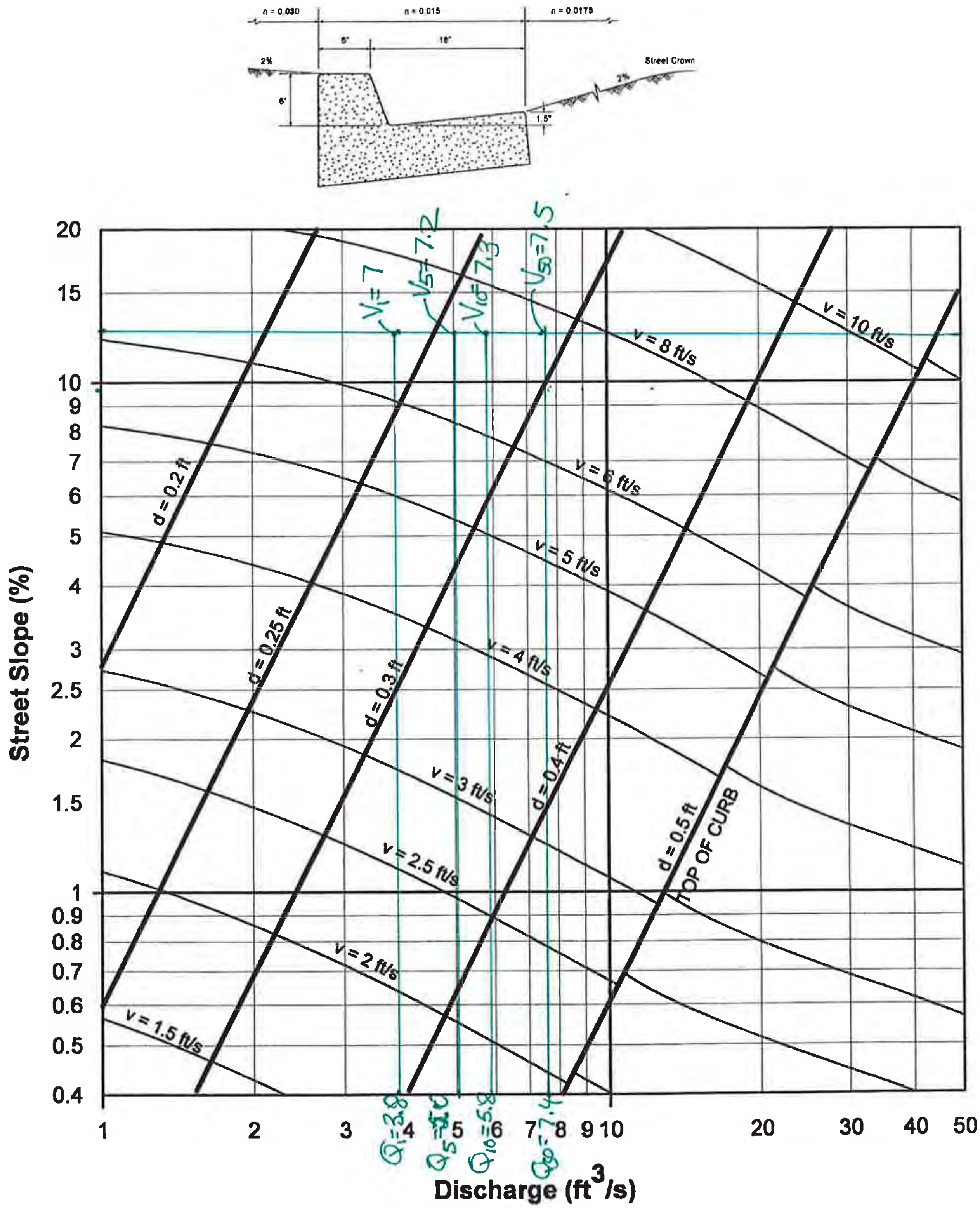


Figure 2-2 6-inch Gutter and Roadway Discharge-Velocity Chart

Basin M2

Figure 2-2

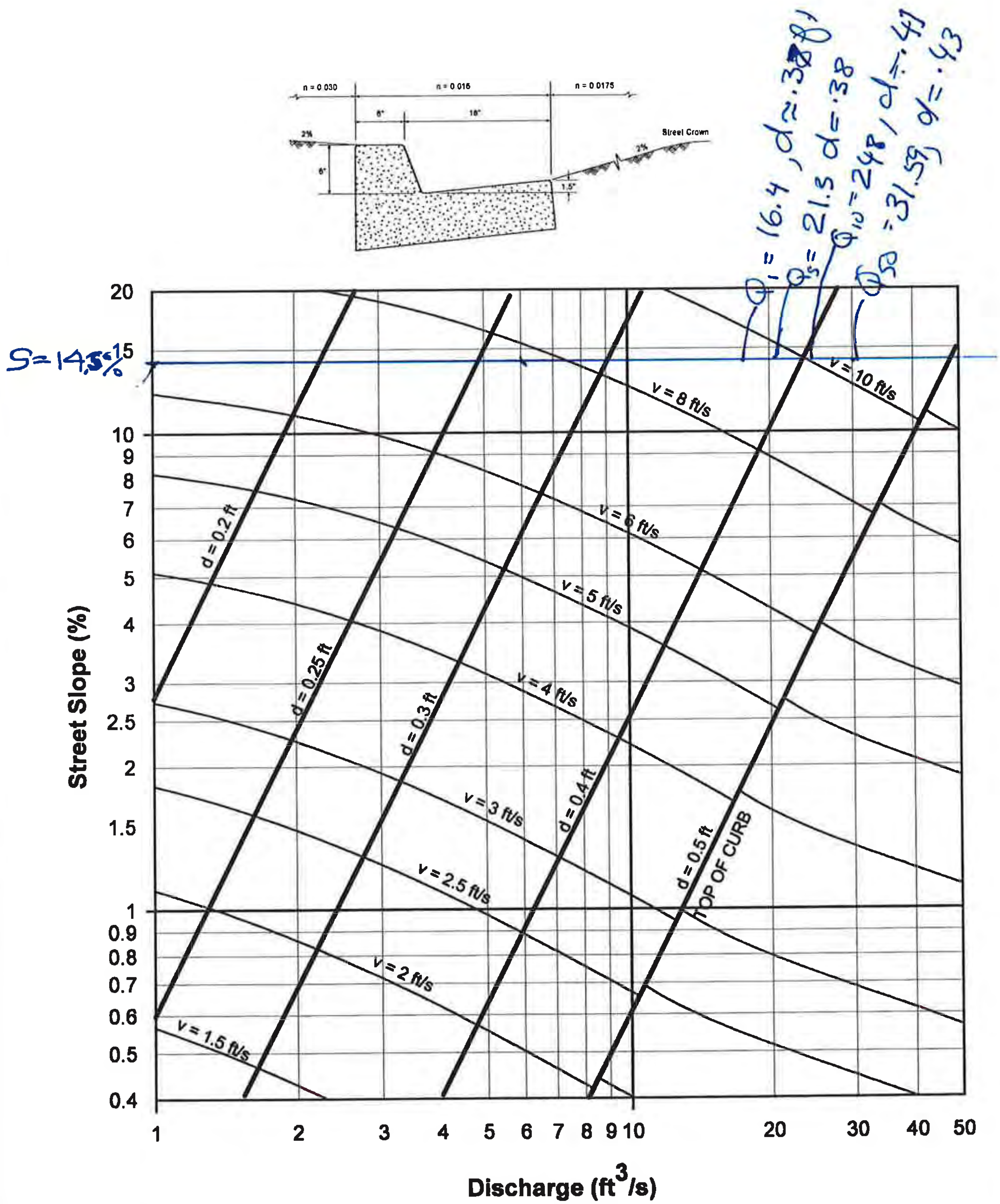


Figure 2-2 6-inch Gutter and Roadway Discharge-Velocity Chart

Basin Q

Figure 2-2

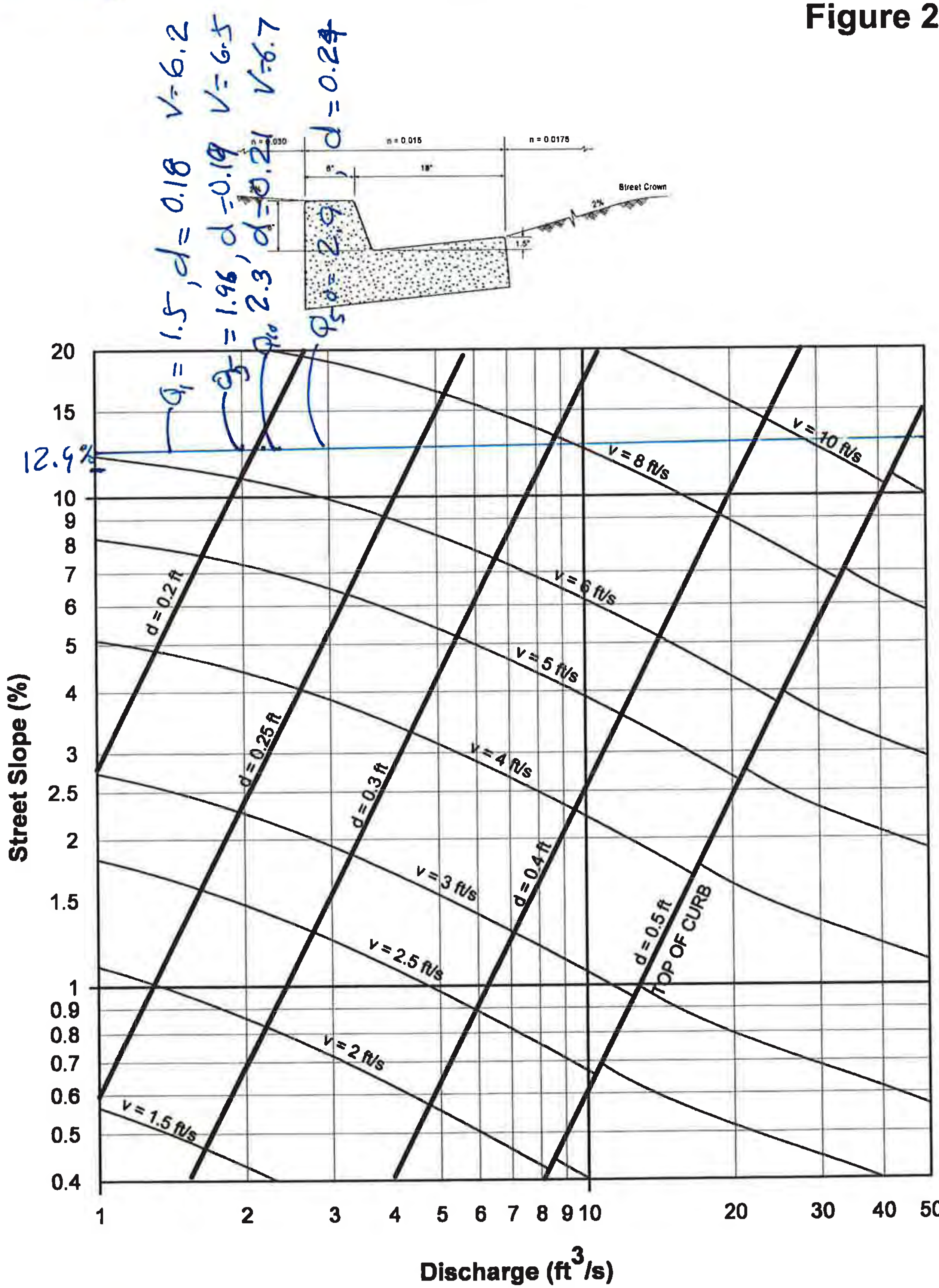


Figure 2-2 6-inch Gutter and Roadway Discharge-Velocity Chart

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APPENDIX C

Shoreline and Bluff Erosion Protection

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SUNSET CLIFFS NATURAL PARK
DRAINAGE & EROSION CONTROL

SHORELINE AND BLUFF EROSION PROTECTION

DRAFT REPORT

Prepared for

DUDEK & ASSOCIATES, INC.
605 Third Street
Encinitas, CA 92024

Prepared by

MOFFATT & NICHOL
1660 Hotel Circle North, Suite 200
San Diego, CA 92108

July 2007

M&N File: 5837

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EXECUTIVE SUMMARY

The City of San Diego has prepared and adopted a Master Plan (MP) and Master Environmental Impact Report (MEIR) for Sunset Cliffs Natural Park (SCNP). The MP and MEIR present recommendations for improving SCNP that follow the vision statement to "*create a Park where people can enjoy San Diego's natural coastal environment as it once was, free from the effects of man and intended to inspire the user to reflect on the grandeur of the sea, and the beauty of the cliffs that are Point Loma*."

The following planning principles were developed as guidelines for the Park planning decisions, and include:

- Do no harm; protect, conserve and enhance.
- Maintain focus on the unique coastal resources.
- Allow public access with minimal environmental impacts.
- Maintain planning integrity/strategy for resource preservation.
- Restore areas of neglect and damage to their previous condition and visual quality.

The first prime task of the MP and MEIR is to conduct a Drainage Study to assess the issues and prepare recommendations for improvements to the drainage system at and surrounding SCNP. As part of the Drainage Study, a shoreline and bluff erosion alternatives analysis is needed to assess current shore protection devices and provide a range of alternatives for other areas of bluff erosion protection. This report presents this segment of the Drainage study. Also included are alternatives for the storm drain outfalls that are recommended as part of the Drainage Study. All alternatives investigated are presented in this study. However, not all alternatives will be suited for the entire park and may require a composite of several alternatives along the SCNP to achieve the goals. Some of the alternatives fail to meet the planning principles listed above and may not be suited for any location within the park.

The shore protection alternatives presented in this study include:

- No Project: This alternative has no proposed changes to the existing conditions at the site. The existing structures, if any, would remain and natural wave-induced erosion would continue. If the upland drainage issues are resolved, then the bluff erosion from substandard upland drainage would decrease.
- Remove Existing Shore-Protection Devices: This alternative considers removing some of the shore-protection devices along SCNP that are failing or will no longer be needed after the storm drain system is redesigned and current outfall structures can be eliminated.
- Beach fill: This alternative consists of placing beach fill sand within existing pocket beaches to provide additional buffer from the waves and tides reaching the bluffs. Any beach fill project would require significant maintenance and renourishment.

- Nearshore Reef: A protected beach area using a low level rocky reef could lessen the erosion problems sustained in a beach fill and lower overall costs. A reef and fill concept would require maintenance over the project life.
- Perched Beach: The perched beach is a variation of the reef concept except that the crest of the reef is higher and the beach is narrower.
- Tie-Back Seawalls: This alternative consists of a cast-in-place concrete wall tied into the slope with rods and concrete anchors. To maintain a natural appearance, the seawall can include a colored and textured surface to aesthetically match the adjacent bluff face. Existing seawalls that are not going to be removed could be modified to include a similar textured surface.
- Riprap revetment: Additional rock structures placed at the toe of the eroding bluff. This alternative is the most unlikely protection alternative since it fails to meet the planning principles of the MP and MEIR.

These alternatives are evaluated in Chapter 5 and site specific applications are presented in Chapter 6. The riprap revetment alternative is the least desirable alternative and its use is only suggested at one small cave location along the entire Sunset Cliffs Natural Park. No structures are proposed in the Hillside Park area of the Park.

Detailed survey information in this area is relatively sparse. The current level of information is adequate to portray conceptual shoreline alternatives; however, initial engineering will require more accurate survey detail. Studies including biological surveys, hydrographic surveys, and detailed engineering will be required prior to implementing any of the proposed alternatives.

The storm drain outfall structure alternatives presented in this study include:

- Internal Dissipation: This alternative includes internal dissipation within the outfall pipe to reduce the velocity of the flow before it exits the pipe. A reduction in velocity can decrease the size of the external dissipating structure. However, internal dissipation can increase the costs of the outfall pipe.
- External Dissipating Rock or Riprap: This alternative consists of a rock or riprap dissipating apron in front of the outfall. The size of the apron will depend on flow velocity and volume. The existing riprap in the areas of the proposed outfalls could be reused to create the apron. The apron can be textured to blend with the adjacent seascape.
- External Dissipating Concrete Baffle Box: This alternative consists of a concrete baffle structure that would dissipate the flow. The size of the structure will depend on design flow velocity and volume. If riprap exists in the proposed outfall location, some may have to be removed to accommodate the structure. The box can be textured to blend with the adjancent seascape.
- Extend into the Nearshore: The proposed outfall pipe can be extended out to the nearshore area, below the mean low water elevation. The pipe could be buried under

the beach sand or existing rubble, or can be textured to blend with the natural rocky nearshore reefs in the area. This alternative could minimize aesthetical impacts, but could increase impacts to public safety and lateral beach access if not placed in the appropriate location. This alternative is probably the least-cost alternative.

A balance between cost and structure size between the outfall pipe and external dissipation structure is needed.These alternatives are presented and evaluated in Chapter 7, Section 7.2 and site specific applications are presented in Section 7.3.

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SUNSET CLIFFS NATURAL PARK
DRAINAGE & EROSION CONTROL

SHORELINE AND BLUFF EROSION PROTECTION

DRAFT REPORT

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1.0 INTRODUCTION

1.1 Study Authorization

The City of San Diego has prepared and adopted a Master Plan (MP) and Master Environmental Impact Report (MEIR) for Sunset Cliffs Natural Park (SCNP). The MP and MEIR present recommendations for improving SCNP that follow the vision statement to "*create a Park where people can enjoy San Diego's natural coastal environment as it once was, free from the effects of man and intended to inspire the user to reflect on the grandeur of the sea, and the beauty of the cliffs that are Point Loma*."

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- Restore areas of neglect and damage to their previous condition and visual quality.

The first prime task of the MP and MEIR is to conduct a Drainage Study to assess the issues and prepare recommendations for improvements to the drainage system at and surrounding SCNP. As part of the Drainage Study, a shoreline and bluff erosion alternatives analysis is needed to assess current shore protection devices and provide a range of alternatives for other areas of bluff erosion protection. This report presents this segment of the Drainage study. Also included are alternative protection devices for the storm drains that are recommended as part of the Drainage Study.

This report will present several alternatives for shore protection; however, not all alternatives will be suited for the entire park and may require a composite of several alternatives along the SCNP to achieve the goals. Some of the alternatives fail to meet the planning principles listed above and may not be suited for any location within the park. Also, it is important to note that the alternatives and analyses presented in this study assume that the existing storm drain system is reconstructed to substantially eliminate existing storm water surface runoff flows over the face of the bluffs and redirected to specific outfall locations along the reach at the base of the bluffs.

1.2 Purpose and Scope

The purpose of this study is to evaluate current shoreline and bluff conditions and develop conceptual beach and bluff protection improvements to mitigate future erosion, consistent with the California Coastal Act, for the protection of Sunset Cliffs Boulevard and proposed storm drain outfall relocations.

The scope of this study includes the following tasks:

1. Review of existing information of historic and recent coastal studies, environmental documents, shoreline assessments, and other information made available by the local community.
2. Conduct a site inspection of the construction access and toe conditions, noting areas of deterioration for possible erosion remediation.
3. Prepare narrative, concept plans and cost estimates describing four concept-level shore-protection alternatives.
4. Attend meetings and community workshops on this project.

1.3 Data Acquisition

Data and reports used in this investigation were obtained from the City of San Diego and Dudek. These include the SCNR Master Plan, Master EIR, City of San Diego Coastal Assessment, SCNR Hydrology and Hydraulic Analysis, Geotechnical reports, reports and documents prepared by other stakeholders, and other miscellaneous reports.

1.4 Site Visits

There were three separate site visits conducted along the park. These include: a May 9th site inspection of Linear Park with staff from Moffatt & Nichol (M&N), Dudek, and a local resident; a May 10th site inspection of Hillside Park with a local resident and staff from M&N and Dudek; and a May 21st site inspection of both Hillside and Linear Park with M&N staff. The first two site inspections were to gather information from the local residents and to gain an understanding of issues from their perspective. The latter site visit was conducted for two main purposes. First to provide feedback on the proposed locations of the storm drain outfalls regarding location, constructability, and access. Secondly, the site visit was to document existing conditions and proposed outfall locations to assist in determining protection alternatives. Appendix A provides meeting minutes and photographs from these site visits.

2.0 EXISTING SITE CONDITIONS

2.1 Study Area Description

The complete study area extends from the northern limits of the park at Adair Street to the southern limits at the US Navy Fort Rosecrans Military Reservation. The study area is broken into two main segments, Linear Park and Hillside Park. Linear Park is the long, narrow portion of the park that extends approximately one mile between Adair Street and Ladera Street. It is bordered on the east by Sunset Cliffs Boulevard and on the west by the Pacific Ocean. Hillside Park begins at Ladera Street and extends south to the US Navy property. Hillside Park is a wide, 50-acre parcel bounded on the east by Point Loma Nazarine University and the Pacific Ocean on the west. Figure 2-1 illustrates the study area.

2.2 Oceanographic Conditions

2.2.1 *Tides and Sea Level*

Tides along the Southern California coastline are of mixed semi-diurnal type. Typically, a lunar day consists of two highs and two low tides, each of different magnitude. Tide gage observations at La Jolla Scripps Institution of Oceanography Pier have been conducted since 1924. Tidal characteristics from the La Jolla gage with reference to a datum of Mean Lower Low Water (MLLW) equal to 0.0 feet are shown in Table 2-1. Storm surge is relatively small (less than one foot) along the Southern California coast when compared to tidal fluctuation.

Table 2-1 Tide Data For La Jolla

Tides	Elevations relative to MLLW datum (feet)
Highest observed water level (11/13/1997)	+7.65
Mean Higher High Water (MHHW)	+5.33
Mean High Water (MHW)	+4.60
Mean Sea Level (MSL)	+2.75
Mean Tide Level (MTL)	+2.73
Mean Low Water (MLW)	+0.91
Mean Lower Low Water (MLLW)	0.00
Lowest observed water level (12/171933)	-2.87

Note: Datums are referenced to the current tidal epoch (1983-2001) and are obtained from the NOAA website at << http://co-ops.nos.noaa.gov/>>.

In Southern California, the highest tides of the year usually occur in the winter months. Typically, this season produces the majority of the storms that cause beach erosion. In 2001, a statistical analysis of annual extreme water elevations were conducted near the project site for the Sunset Cliffs Road Protection project at Adair Street (M&N 2001a). The annual extreme high water elevations versus recurrence interval for the Sunset Cliffs shoreline that were developed for the M&N 2001 study are shown in Table 2-2 and are applicable for this project along the entire reach of SCNP.

Barometric pressure changes in water surface elevation caused by the passage of intense low-pressure systems or storm surges are relatively small (less than a foot) along the Southern California coast as compared to tidal elevations. Although small, these storm surges must be considered and are included in tide data information listed above when designing coastal structures to limit overtopping to acceptable amounts.

Table 2-2 Water Elevation Vs. Recurrence Interval

Interval (Years)	Water Elevations (feet, MLLW)
5	7.32
10	7.40
25	7.53
50	7.62
100	7.73

Source: USACE 1991, *Weibull formula

2.2.2 *Waves*

Ocean waves off the coast of Southern California can be classified into four main categories: northern hemisphere swell, tropical swell (Chubascos), southern hemisphere swell, and seas generated by local winds.

1. Northern Hemisphere swell represents the category of the most severe waves reaching the California coast. Deep-water significant wave heights rarely exceed 10 feet, with wave periods ranging from 12 to 18 seconds. However, during extreme Northern Hemisphere storm events, wave heights may exceed 20 feet with periods ranging from 19 to 22 seconds.

2. Tropical cyclones develop off the West Coast of Mexico during the summer and early fall. The resulting swell rarely exceeds 6 feet, but a strong Chubasco in September 1939 passed directly over the Southern California area and caused one of the highest waves on record at 26.9 feet. A major storm in January 1988 caused waves that were measured at over 30 feet.

3. Southern Hemisphere swell is generated by winds associated with storms of the austral winter in the South Pacific. Typical Southern Hemisphere swell rarely exceeds 4 feet in height in deep water; but with periods ranging up to 18 to 21 seconds, they can break at over twice the swell height.

4. Sea is the term applied to steep, short-periods waves which are generated from either storms that have invaded the Southern California area, strong pressure gradients over the area of the Eastern Pacific Ocean (Pacific High), or from the diurnal sea breezes. Wave heights are usually between 2 and 5 feet with an average period of 7 to 9 seconds.

A wave exposure diagram is shown in Figure 2-2. The Sunset Cliffs are directly exposed to ocean swell entering from two main windows. The more severe northern hemisphere storms enter between azimuths 289 and 299 degrees relative to true north (0 degrees). The Channel Islands (San Miguel, Santa Rosa, Santa Cruz, and Anacapa) and Santa Catalina Island provide some sheltering from these larger waves depending on the approach direction. The other major exposure window opens to the south between 180 and 276 degrees, allowing swell from Southern Hemisphere storms and tropical storms (Chubascos).

For shore protective devices, the design waves will be depth-limited. As waves enter shallow water they become unstable and break. The large deep-water waves will break offshore and then reform. The water depth fronting a structure controls the design wave height. If a structure is in deeper water, the waves breaking on that structure may not be depth limited. Table 2-3 contains wave height/return-interval data (USACE 1991) at the Scripps Pier, which could be similar to conditions at Sunset Cliffs if an offshore structure (like a breakwater) were to be proposed.

Table 2-3 Wave Height Vs. Recurrence Interval

Interval (Years)	Significant Wave Height (feet)
Mean Hs	4.9
5	9.3
10	10.5
25	12.1
50	13.2
100	14.3

Source: USACE 1991

2.3 Littoral Processes

The project site lies along the Mission Bay Littoral Cell extending approximately 14 miles from Point La Jolla to Point Loma (USACE 1991). The littoral cell is further divided into sub cells, in which the Sunset Cliffs is included with the Point Loma sub cell. The Point Loma sub cell is characterized by rocky cliffs with fairly stable formations. The cliffs along Point Loma are steep and tall, reaching approximately 300 feet in some areas. Historic accretion/ erosion profiles indicate that material has accreted at the tip of Point Loma and eroded for portions of the cliffs over 2 feet. The sediment source in this area is limited to the cliff erosion. Kelp removal operations have inadvertently contributed to the loss of sand sources over time. Longshore transport of sediment in the littoral cell moves both to the north and to the south, with a net transport to the north (USACE 1991).

A wide range of erosion rates has been reported for the Sunset Cliffs area. A recent study (Hapke and Reid 2007) reported the erosion rate the bluffs along Sunset Cliffs to be between 0.1 and about 0.8 m/year (about 2.6 feet/year), the maximum retreat was almost 330 feet over the study's 70-year analysis, and the highest rate of cliff retreat was measured near the Point Loma Nazarine College (5.3 feet/yr). However, bluff erosion tends to be localized and can be highly variable due to the episodic nature of the coastal erosion process.

2.4 Geology

The coastal bluff in the project area consists of two different geologic units with different strength and erosion characteristics. The upper bluff (above about elevation +23 feet MLLW) consists of Pleistocene sand and gravel (Bay Point Formation). The upper bluff "terrace deposits" are generally susceptible to erosion from runoff and tend to form moderate slopes. The lower seacliff is underlain by Cretaceous sandstone and shale of the Point Loma Formation. The Point Loma Formation is relatively resistant to wave erosion, and tends to form sea caves, surge channels and overhangs, often along fracture and fault zones. Groundwater seepage can influence the rate of coastal erosion, although groundwater seepage in the area appears minor (GPI 2006). The typical erosion process is that the seacliff toe becomes undercut leading to periodic blockfalls. Progressive undercutting may create sea caves. The blockfall process and/or sea cave collapse eventually undermines the terrace deposits, which quickly slough back to a flatter slope inclination.

Original shoreline construction along the reach include rock revetments and rock fill placed by the US Army Corps of Engineers in 1971, two seawalls constructed by the USACE in the 1980s, and miscellaneous rubble fill and concrete structures. Much of the Corps rock revetments were placed in an attempt to slow the bluff erosion and protect Sunset Cliffs Boulevard, and to slow the erosion of the existing cave structures. These revetments vary in dimensions and elevations and most appear to be graded large riprap.

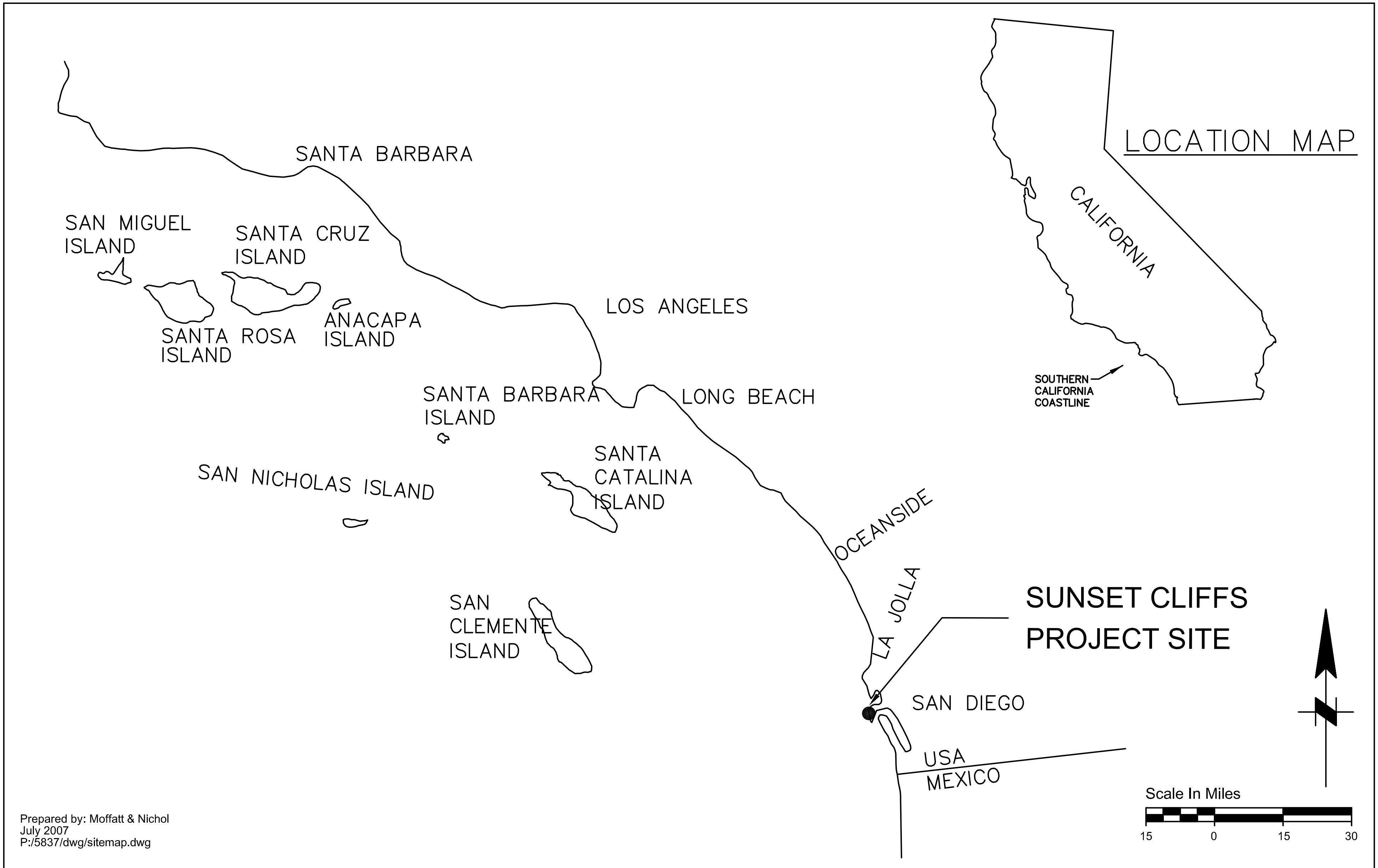
The two seawalls, located between Osprey and Adair Streets, are S-shaped Reinforced Earth Walls™. The seawalls consist of stacked, rectangular-shaped, interlocking precast concrete panels with horizontal galvanized steel reinforcing strips located within the granular backfill zone. The seawalls were founded on a combination of quarry run and rock rubble over the eroded formational terrace platform. The seawalls are fronted by a rock revetment consisting of graded large riprap. The ends of the seawalls were originally keyed into adjacent, near vertical bluff areas consisting of competent formational materials. However, bluff erosion has caused progressive collapse of the bluff. At some point in time, an additional limited amount of riprap was placed along the southern end of the northern seawall in an effort to mitigate the erosion. In some places, the riprap was grouted in-place.

2.5 Biological Resources

Describe underwater reefs and kelp beds offshore…

2.6 Existing Utilities

Investigations from the 2001 Sunset Cliffs Road Protection project at Adair Street project show that there is an existing storm drain and trunk sewer within the Sunset Cliffs Boulevard right of way. The storm drain in the area of consideration is an 18-inch reinforced concrete pipe (RCP) located just east of the street centerline. The trunk sewer is a 10-inch vitrified clay (VC) pipe located east of the storm drain. Additional utilities investigations should be conducted to identify potential electric, communication, freshwater, or gas conduits if modifications are required to the west road edge.



Prepared by: Moffatt & Nichol
July 2007
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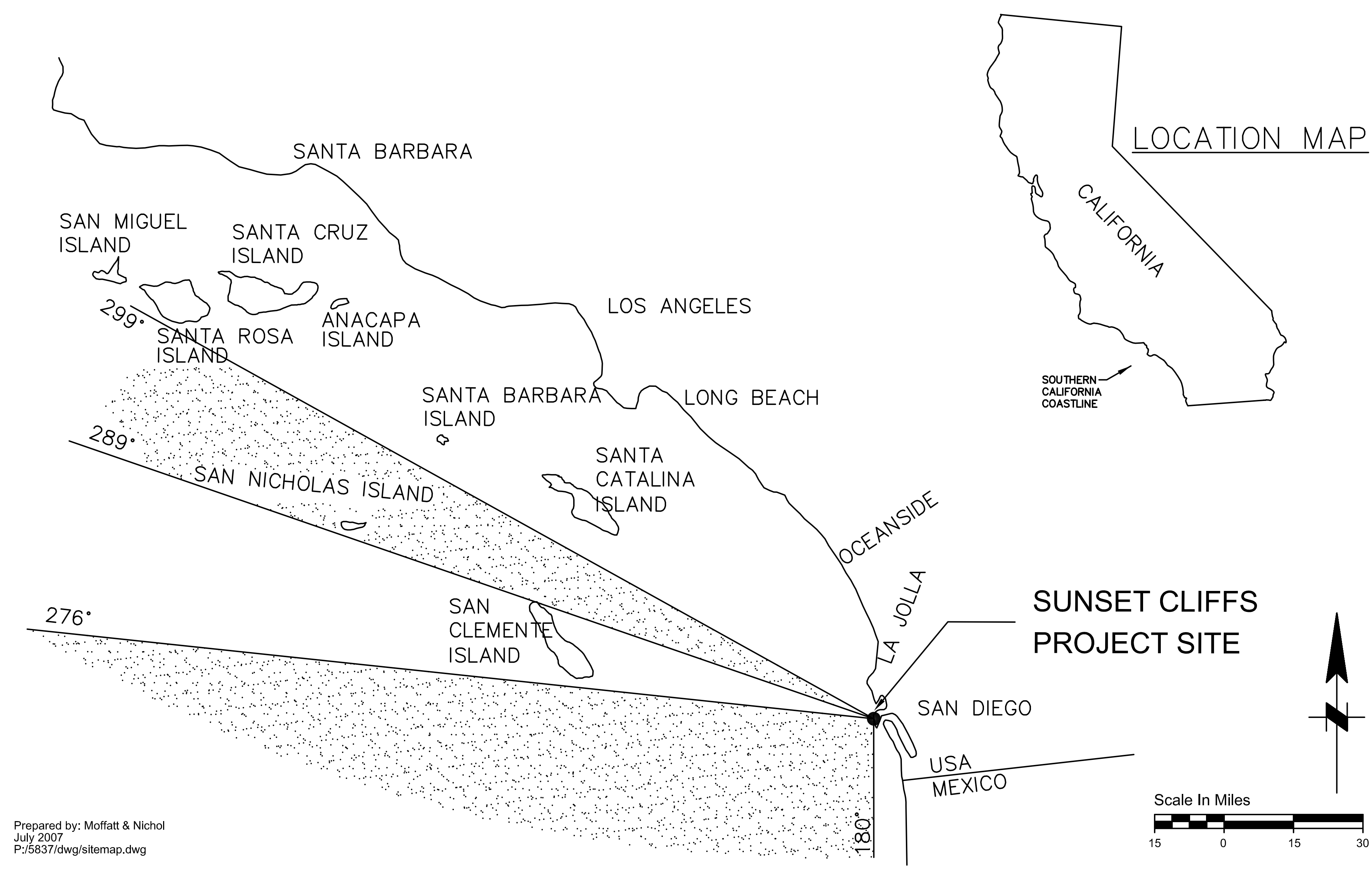
SUNSET CLIFFS NATURAL PARK
SHORELINE AND BLUFF STABILITY STUDY

Site Map

Figure
2-1



SUNSET CLIFFS NATURAL PARK
SHORELINE AND BLUFF STABILITY STUDY

Wave Exposure

Figure 2-2

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3.0 DESIGN CONSTRAINTS AND CRITERIA

3.1 Design Constraints

The selection of alternatives to protect the Sunset Cliffs Natural Park shoreline is limited by site related constraints. Physical limitations include the following:

1. Minimize encroachment of structures on the rocky foreshore.
2. Minimize structure crest elevations for aesthetics and access.
3. Do not impair use or structural integrity of the existing walkways or roadway.
4. Protect underwater reefs and kelp beds offshore, which are sensitive ecosystems.
5. Do not impair existing recreational uses of the crest or foreshore areas.
6. Do not impair public access
7. Protect, conserve, and enhance the natural coastal environment

3.2 Bluff Wave-Erosion Protection (Preliminary Design Criteria)

A description of preliminary design criteria were presented in the Sunset Cliffs Road Protection at Adair Street Study (Moffatt & Nichol 2001a). Since this site is representative of the entire study area and the text presented in the study report is relevant to the entire Sunset Cliffs Natural Park, the entire section is duplicated here.

Oceanographic data are required for the design of structures and protective devices subject to waves and currents. Design parameters to be considered are illustrated in Figure 3-1 and are described below.

1. Design and still water elevation, H_w, and future elevations considering anticipated long-term changes in sea surface elevations.
2. Extreme anticipated scour elevation, H_{sc}.
3. Nearshore slope, m.
4. Wave characteristics, including breaking wave height, h_b; and wave period, T.
5. Maximum wave runup elevation, H_r, which is equal to H_w+R, where R is the wave runup distance above the still water elevation.
6. Volumetric rate of wave overtopping, Q.

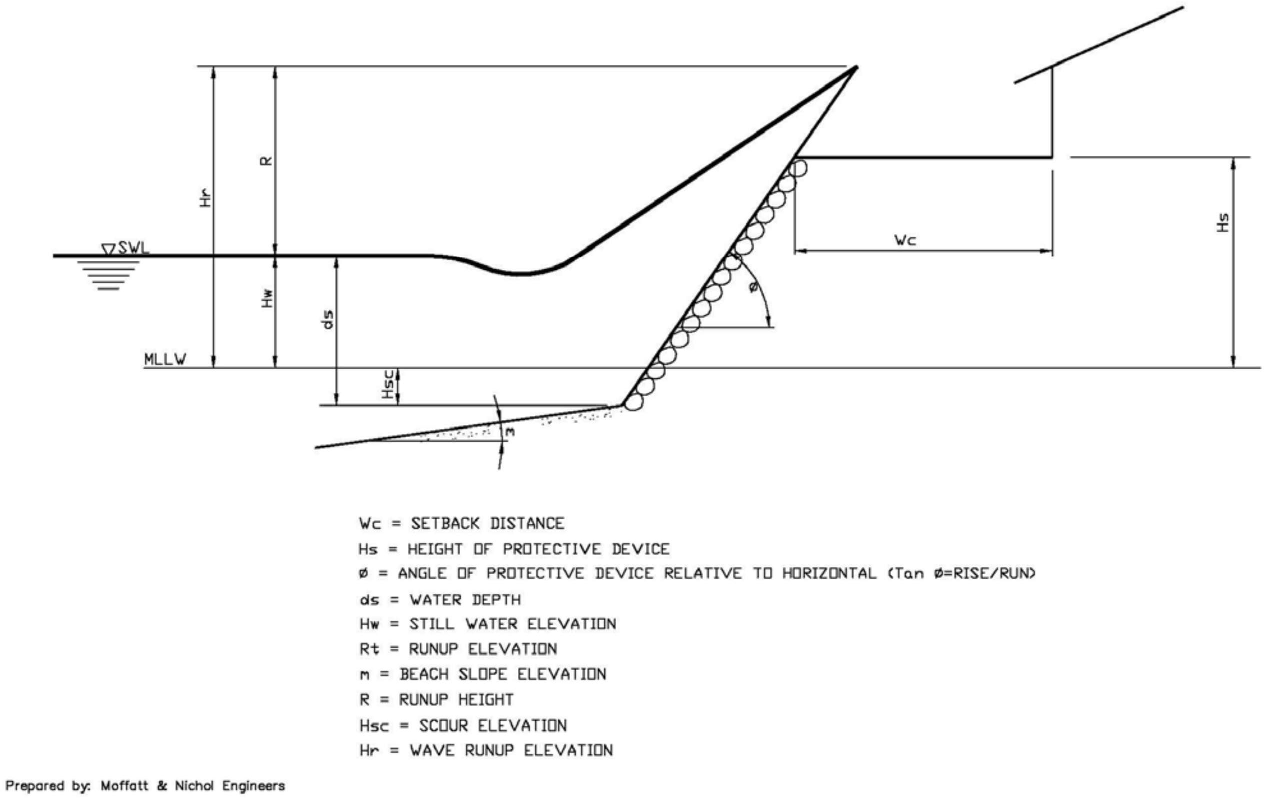

Figure 3-1 Design Parameters

3.2.1 *Design Water Level*

A statistical evaluation of extreme water elevations was conducted for the Adair Street project site (Moffatt & Nichol 2001a). The result includes annual extreme high water elevations versus recurrence interval. However, in addition to the short-term fluctuations in the sea surface, the effects of a progressive change in sea level must be considered for the life of the structure or protective device. Recent studies have documented increasing sea levels, which should be considered in the design of permanent and temporary structures.

A study by the National Research Council Marine Board (NRCMB 1987) predicts a rate increase of 1.3 feet per century recommended for 25-year design projects. The historical rate of sea level rise has been 0.4 to 0.5 per century. For purposes of this study, an average of the historic sea level rise and the rate predicted by NRCMB is used. Table 3-1 summarizes the rise in sea level associated with the various intervals under consideration.

Table 3-1 Sea Level Rise For Design

Interval (Years)	Future Sea Level Rise (Feet)
5	-
10	0.1
25	0.2
50	0.5
100	0.9

The maximum design water level for these projects designed for a life of 50 years takes into account the water level for the 50-year recurrence interval (7.62 feet) and the predicted rise in sea level after 50 years (0.5 feet). In this instance, the design still water level will be 8.12 feet.

3.2.2 *Nearshore Slope*

The nearshore slope must be known to determine the maximum wave height and runup distance on a protective device. It is assumed that nearshore slopes during storm conditions will be somewhat flat due to the relatively hard and flat surface of the geological formation. No profile data is available for the project site beyond the –2-foot contour. Based on the limited available data, slopes of approximately 20:1 (horizontal:vertical) are assumed to be appropriate for maximum scour conditions.

3.2.3 *Scour Depth Potential*

The design scour elevation is anticipated to be minimal due to the presence of resilient formation materials. This parameter along with the design still water elevation must be established to determine the maximum water depth at the structure. The maximum water depth determines the design wave height, runup elevation, and overtopping rate. The design elevation is also required to determine the toe depth of the shoreline protective device to minimize the potential for undermining. This site could erode to a depth of +1 feet MLLW, based on geotechnical observations.

3.2.4 *Design Wave Height*

Extreme wave conditions must be predicted for the design of shore protection structures. Furthermore, these design wave conditions are used to estimate the wave runup and overtopping rates, which determine the structure, crest elevations to limit flooding of the backlands and associated damages.

Deep-water wave heights in excess of 20 feet can be expected to occur during the life of a shore protection device along the coastline. However, design waves, which will act upon these structures fronted by a shallow rocky foreshore, will be depth dependent. Waves exceeding the maximum depth-limited wave height will break farther offshore and dissipate much of their energy before they reach the structure.

Table 3-2 Water Elevation And Breaking Wave Height vs. Recurrence Interval

Recurrence Interval (Years)	Water Elevation (feet, MLLW)	Design Breaking Wave Height (feet)
5	6.3	7.0
10	6.5	7.3
25	6.7	7.5
50	7.1	7.9
100	7.6	8.5

Table 3-2 shows the relationship between the depth-limited breaking wave height based on water elevation including sea level rise at the base of the structure, scour, and recurrence interval associated with that water depth. The design water depth is calculated by adding the appropriate year sea level rise to the predicted extreme water elevations (i.e., 50-year conditions for 50-year design water depth, etc.). Breaking wave calculations are based on methods presented in the USACE Shore Protection Manual (1984).

3.2.5 *Design Wave Runup Elevation and Overtopping Rate*

Wave runup can be an important design parameter because it establishes the vertical height above the still water level to which water from an incident wave will run up the face of the structure. Runup depends on the shape of the structure, the roughness of the structure slope, water depth at the toe of the structure, bottom slope in front of the structure, and wave and water level characteristics. If the runup elevation exceeds the crest elevation of the structure, wave overtopping will occur. These rates would apply only to a structure seaward of the bluff face that protects beach nourishment alternatives, since the height of the bluff is much higher than any potential runup scenario. Shore protection structures commonly allow a certain amount of overtopping. Extensive wave overtopping can subject shore protection structures and backlands to damage. Overtopping can erode the area behind the structure, negating the purpose of the structure. Soil supporting the top of the structure can be removed leading to failure of the structure.

The existing bluffs extend to elevations at 30+ feet above the water surface, virtually blocking runup and overtopping, although occasional spray extends above the bluff line. Overtopping and runup would be a consideration only for lower crest elevation alternatives fronting the beach protection alternatives.

3.3 Storm Drain Outfall Preliminary Design Criteria

The selection of alternatives for the proposed storm drain outfalls along Sunset Cliffs Natural Park is limited by similar site related constraints as the bluff protection alternatives. These include:

1. Minimize encroachment of structures on the rocky foreshore.
2. Minimize structure elevations for aesthetics and lateral beach access.
3. Protect underwater reefs and kelp beds offshore, which are sensitive ecosystems.
4. Do not impair existing recreational uses of the foreshore areas.
5. Do not impair public access.
6. Protect, conserve and enhance the natural coastal environment.
7. Minimize beach and bluff erosion from outfall drainage.
8. Minimize maintenance requirements.

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4.0 BLUFF PROTECTION ALTERNATIVES

This section provides a general description of potential shore protection alternatives that could be applied along SCNP coastline. These include no project, beach fill, nearshore reef, perched beach, tie-back seawall, and rock revetment. These are all of the alternatives considered, but it is important to note that not all will be applicable or recommended. Chapter 5 discusses the evaluation of the alternatives and Chapter 6 discusses the site specific applications for each alternative along the Park.

Also, these alternatives are presented under the assumption that the existing storm drain system throughout the Park area will be reconfigured to significantly reduce surface water runoff over the tops of the bluffs from low-flow and storm events. The proposed storm water drainage systems will collect the water at catch basins along the western curb at Sunset Cliffs Boulevard and the outfall structures will be located at the base of the bluffs. Once the surface runoff is significantly reduced, future bluff erosion will be greatly reduced since this is a major contributing factor to bluff instability along the Park. The majority of the upper bluff erosion along the Park is the result of surface runoff, overwatering of the bluff top, pedestrian traffic, and burrowing by animals. Coastal processes contribute to a portion of the lower bluff erosion.

4.1 No Action Alternative

This alternative does not include any major beach of bluff protection structure, but may include minor repairs to the existing structures and minor restoration of failing bluffs that are undermining the existing road. This may include minor grading at the bluff top, removing small portions of the concrete/rubble debris, seawall patching, and minor revetment redistribution.

With the assumption that a significant portion of the storm water runoff will be directed through a reconfigured storm drain system and away from draining over and through the bluff material, most of the upper bluff erosion will be greatly reduced.

4.2 Remove Existing Structures

All along SCNP are areas of rock revetments and rubble fill that were, for the most part, placed by the US Army Corps of Engineers in the 1970s and 1980s. There are two seawalls located along the northern section of linear park that were constructed in the mid-1980s. This alternative proposes to remove existing structures, where practical, and return the bluff face to a more natural condition. Some segments are unstable and highly erosive and removing any existing structures in these areas is not recommended. Other areas may be more stable from coastal erosion and existing shore-protection structures could be removed. Also, there may be areas that could implement this alternative, but access to retrieving the protective structures may make this an unconstructible alternative.

4.3 Beach Fill Alternative

The objective of a beach fill is to directly increase the level of shoreline protection and recreational opportunities by widening the beach area. Neither the City of San Diego nor SANDAG have indicated a possibility for beach nourishment along Sunset Cliffs due to the lack of access, potential to impact sensitive marine resources, and poor characteristics for sand retention. Most areas along SCNP would be especially difficult to implement a beach fill because of the lack of substantial existing beach and proximity to sensitive marine resources of kelp beds and rocky reef. There are some small pocket beach areas that could be better suited for a beach fill. Most of these small pocket beaches contain riprap and a beach fill could be constructed over the riprap, burying the rock, however, the longevity of any beach fill along this coast is difficult to estimate.

A pilot beach fill project could provide an opportunity to monitor the movement of the sand and supply resource agencies with sufficient information to determine the level of impact to sensitive marine resources. Monitoring data could be analyzed by the City and resource agencies to refine the design of future beach replenishment projects (i.e., quantities, placement locations, timing of placement, beach fill gradation, etc.).

General guidelines for beach nourishment, as stated by SANDAG for shore protection purposes, are to provide a minimum width of 200 feet. This would require an estimated minimum quantity of approximately 150,000 to 200,000 cubic yards of fill per 1,000 feet of coastline, although survey data would need to be used to calculate an accurate total quantity for each beach fill area. Any beach fill placed along SCNP would likely be eroded by ocean forces and coastal processes, such as longshore and cross-shore transport, within approximately one year of installation, which would require constant replenishment no more than every two years. Figure 4-1 and Figure 4-2 show the Beach Fill Alternative plan and profile views.

Beach fills temporarily offer protection to the bluff, but over the long-term it is not a viable solution because of the naturally occurring erosion along the Park. However, the shoreline protection aspect of beach nourishment can be better accomplished by implementing subsequent nourishment projects in the long-term (renourishment cycle). Future projects could progressively increase in scale and quantity from initial projects, and be more refined/tailored based on monitoring data.

The performance of beach fills depends greatly on the sand grain size and overall volume of material. The 2001 Regional Beach Sand Project (RBSP) conducted by SANDAG consisted of 2 million cubic yards of sand placed on 11 beach fill sites. Monitoring results found that the beaches with coarser material and larger volumes of sand retained the sand for longer than beaches with finer material and/or smaller volumes (Coastal Frontiers 2002). Coarser material provides a different equilibrium profile than finer material. Fine sands will be deposited further offshore in deeper water, forming a more gentle beach slope, resulting in a narrower beach berm. Coarser sands tend to form a steeper slope, with more of the sand staying on the higher portions of the beach profile. The berm width formed with coarser sand is therefore wider and tends to provide a greater degree of protection to areas behind the beach. Steeper beaches experience higher wave runup than flatter beaches and are less desirable for recreational users.

Adjustment of the beach profile after construction of the project will occur as waves rework the sand. This condition is referred to as profile equilibrium. The rate of beach profile adjustment, or equilibration, depends on the wave climate during and following the fill. The exact rate of berm recession cannot be accurately determined without more data and analysis, but may be complete by the end of the wave season occurring during beach nourishment activities. Aesthetically, a beach fill provides a natural setting over a hard-scape shoreline protective structure.

4.4 Nearshore Sand Retention Reef Alternative

This alternative consists of an individual nearshore reef constructed of either large geotextile containers filled with sand or quarried stone. Along SCNP, this alternative would best be suited in small pocket beach areas. A concept plan and section for the retention reefs are shown in Figure 4-3 and Figure 4-4 and consist of an arrowhead-shaped reef structure with the point facing nearshore. The reef is proposed to be shallow enough that waves would break over the crest, expend energy nearshore, and create an energy lee near the beach. Sand would potentially accumulate in the lee of the structure producing a salient and widening of the beach. For this example, the reef is placed at -2 feet MLLW and extends landward (See Figure 4-3 and Figure 4-4). The reef is approximately 120 feet wide by 100 feet long and oriented at a 45-degree angle to the beach. This figure shows a beach fill placed behind the structure as part of the alternative design. A side benefit of the retention reef could be the creation of a surfing opportunity.

Although the nearshore reef alternative may not be as effective as offshore breakwaters, the benefits of the reef are that beach widening may occur while surfing and recreational opportunities at the structure are maintained. Also, the reef may be aesthetically more appealing than a surface-piercing offshore breakwater. Retention reefs have not yet been constructed in the San Diego region and only a few have been constructed worlwide. These structures are still very experimental and require more research such as physical and numerical model testing. In addition, access by construction equipment may be problematic.

The nearshore reef concept presents potential benefits as described in the recent study for SANDAG (Moffatt & Nichol 2001). If a demonstration project could be constructed and monitored for its effectiveness, the results could potentially be applied in this location. Monitoring results could be used to reevaluate the performance of the reef concept. The structure could then either be modified as needed and possibly used more widely, replaced with a permanent structure, or be removed and eliminated from future consideration.

A nearshore retention reefs would not significantly change aesthetic conditions compared to natural conditions because it would be submerged. The reef would be designed to create recreational surfing conditions in addition to retaining sand. The reef could however, present a navigation hazard because it would exist near navigational areas.

4.5 Perched Beach Alternative

A perched beach is formed above the existing beach as a result of a submerged retaining structure, similar to the nearshore reef alternative, which traps sand on the landward side of the structure. The main differences between the perched beach alternative and the nearshore reef alternative is the elevation and width of the structure. The perched beach structure is approximately 20 feet wide and is elevated to +15 feet MLLW, similar to an offshore submerged breakwater, as shown in Figure 4-5 and Figure 4-6. In comparison, the nearshore reef alternative described above is a diamond shape with dimensions 120 feet by 100 feet at -2 feet MLLW). The area behind the perched beach retaining structure would be filled with sand.

The resulting perched beach that forms landward of the submerged structure has many of the same qualities as natural beaches and does not block the ocean view. Perched beaches are appropriate erosion control measures where a beach is desired and sand loss is too rapid for convenient or economical replacement.

Construction materials and design considerations for perched beach structures are generally similar to those for fixed breakwaters. For example, the sill can be constructed of a range of materials including quarrystone. Smaller-scale structures could use large geotextile bags filled with sand, but would not be as large as shown in the schematic figures.The sand for a perched beach may be trapped by the sill after being carried inshore by the normal wave action, or it may be transported from another site as beach fill.

4.6 Tie-back Seawall Alternative

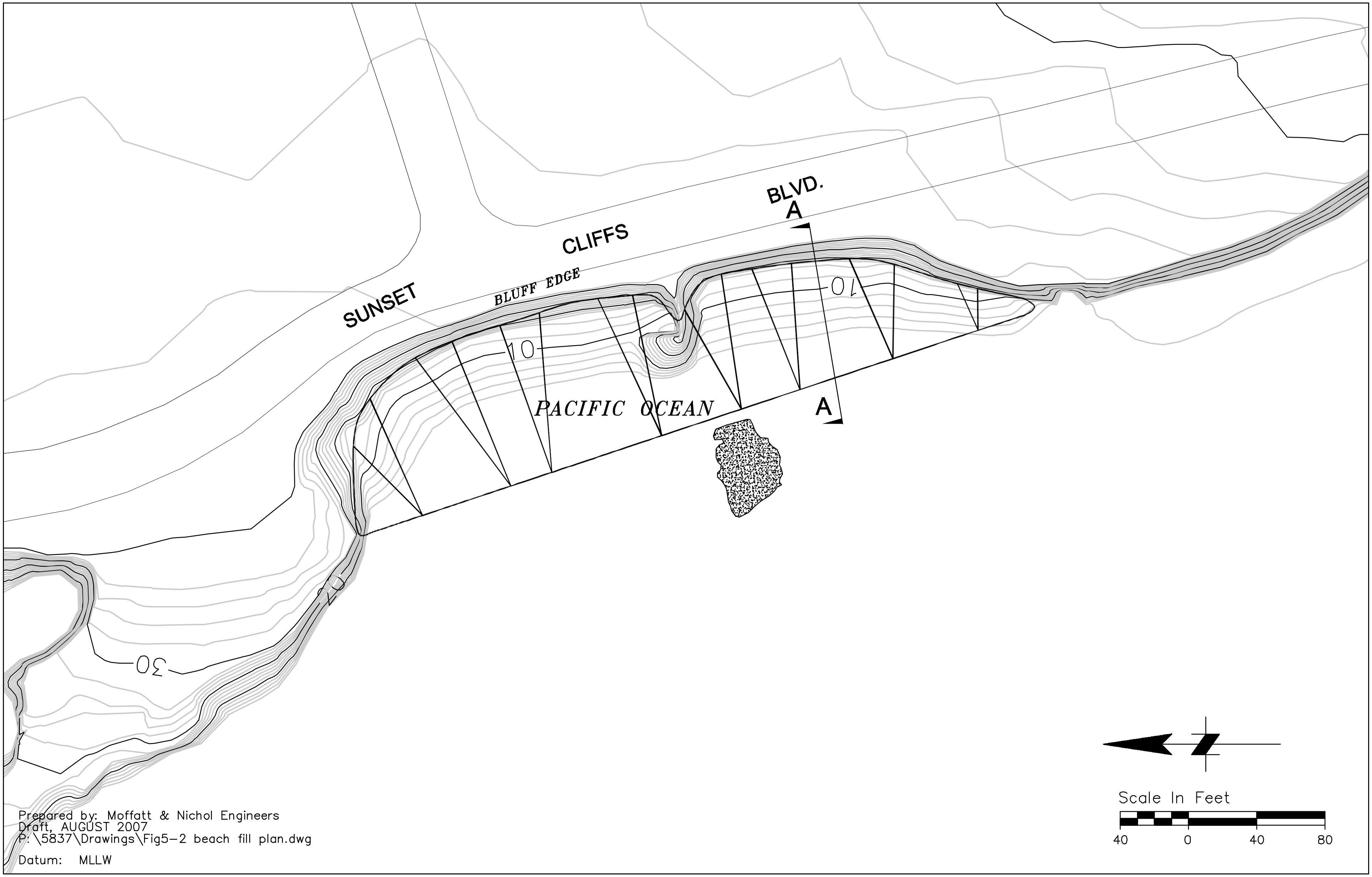
The tie-back wall alternative is a vertical retaining wall, designed to prevent the upland soil from sliding seaward and to minimize undercutting of the cliffs below from wave attack. Tie-back walls are braced by cables or rods tied to anchors in the fill behind them. Tie-back walls are usually constructed of cast-in-place concrete or precast elements. The existing seawall stabilizing a portion of this reach of shoreline is a type of tie-back wall that employs steel strips embedded in compacted earth fill to hold precast concrete blocks in place. Recent seawall installations in this area and others in California include textured or sculpted walls, which use colored grout material and shaped surfaces to mimic the adjacent bluff face to provide a better aesthetic quality to the structure. The Tie-back Seawall Alternative is shown in Figure 4-7 and Figure 4-8.

These vertical wall structures will, in most cases, transmit hydrodynamic forces produced by waves to the soil behind it; the soil must therefore be compacted and retained. Most seawall and bulkhead failures in southern California have occurred because the backfill material was lost and the wall failed in shear or inward bending moments (Moffatt & Nichol Engineers 1985). Seawall failures are less likely to occur where the backfill is properly placed, compacted, and retained. Another mode of failure is from inadequate design of tie-backs and where scour occurs at the toe or flank of the wall. This failure mode will likely occur to the existing seawall without some form of protection of the eroded flank face. Additionally, tie-back walls can fail after the connections corrode after long-term exposure to saltwater environments if not properly designed. The existing seawalls along SCNP will likely experience this failure over time.

4.7 Revetment Alternative

A stone revetment is a common type of structure used for shore protection in Southern California, although it has been perceived negatively in recent years due to aesthetics and shore hardening objections. A stone revetment is composed of one or two layers of large armor underlain with smaller stones and either a graded stone filter or geotextile filter fabric. Underlying geotextile filter fabric is often used to relieve the hydrostatic pressure and retain backfill soils from escaping through voids in the rock. A stone revetment can adjust and settle to a minor degree after construction without causing structural failure. They are flexible, so that damage from waves that exceed the design wave is usually progressive and can be repaired. The displacement of several armor stones will usually not result in the complete loss of protection. Stone revetments typically cost less than vertical walls. Along SCNP, a revetment would armor the bluff toe from increased erosion but would not extend to the top of the bluff.

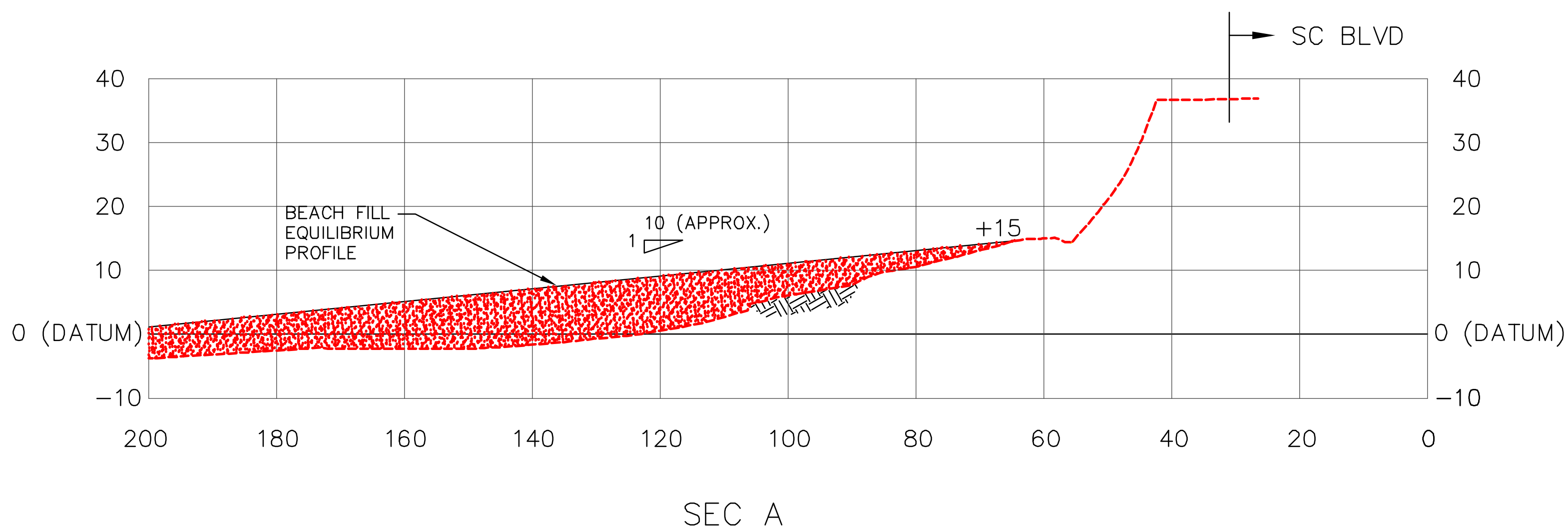
A proposed revetment plan and section for shore protection is shown in Figure 4-9 and Figure 4-10. Existing revetment could be incorporated into this concept. For aesthetics, the exposed face can be covered with a textured surface using colored grout and shaped surfaces.



SUNSET CLIFFS NATURAL PARK SHORELINE AND BLUFF PROTECTION STUDY

BEACH FILL ALTERNATIVE PLAN VIEW (EQUILIBRIUM PROFILE)

Figure 4-1



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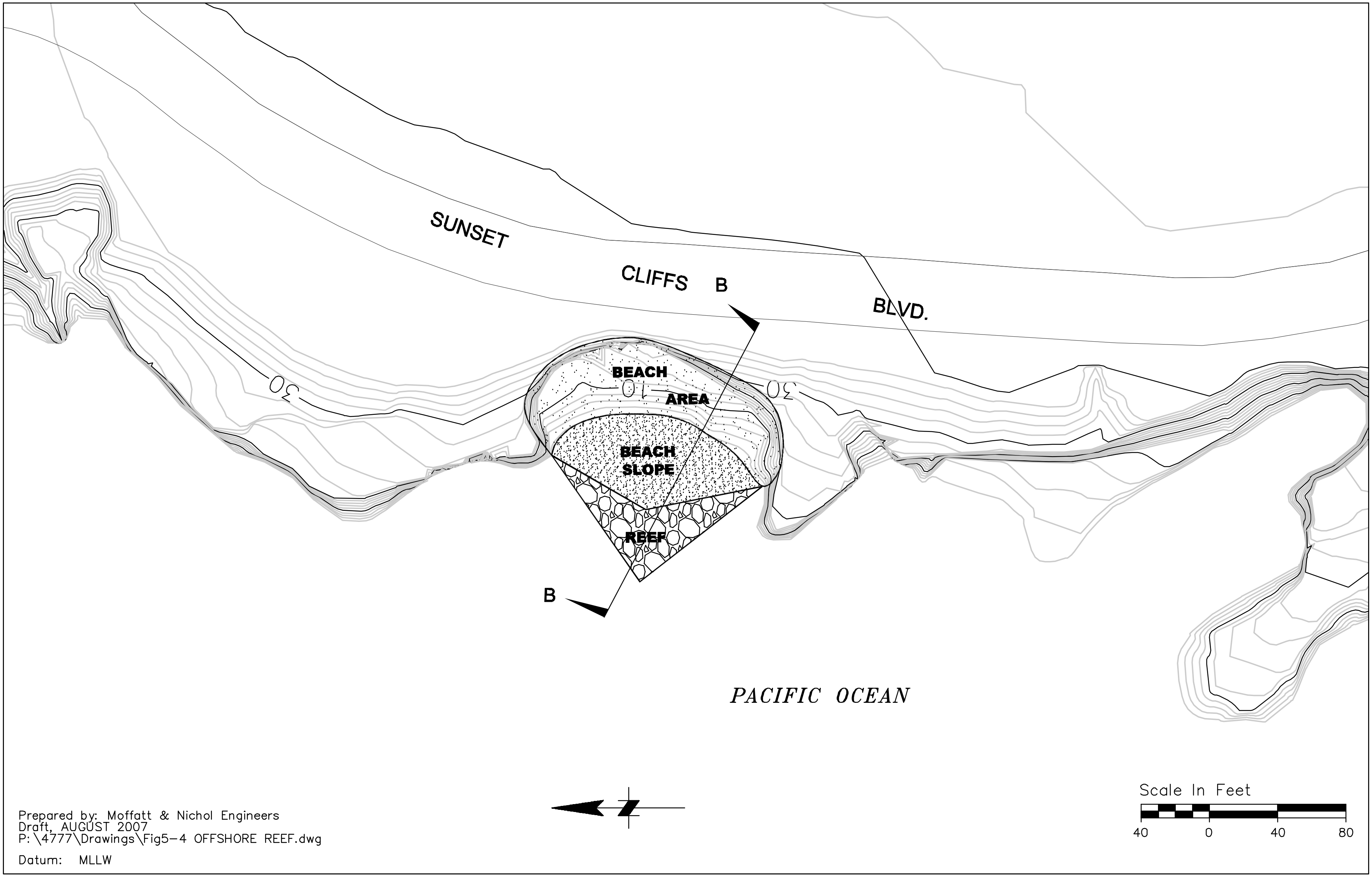
SUNSET CLIFFS NATURAL PARK SHORELINE AND BLUFF PROTECTION STUDY

BEACH FILL ALTERNATIVE PROFILE VIEW (EQUILIBRIUM PROFILE)

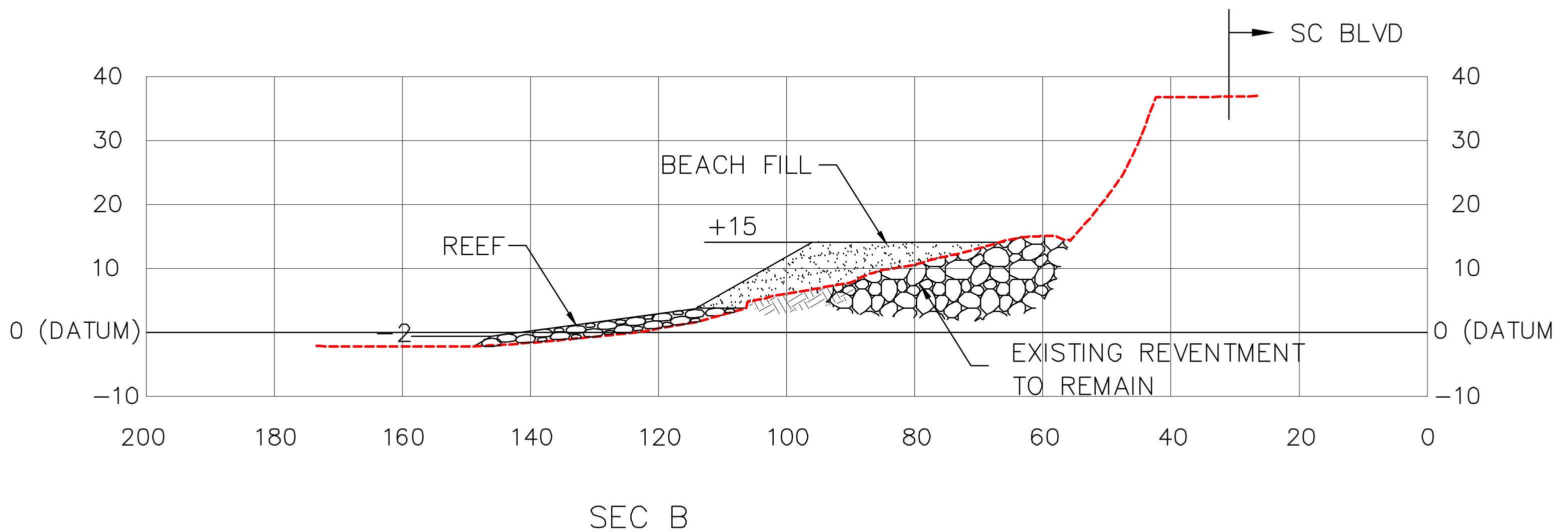
Figure 4-2



SUNSET CLIFFS NATURAL PARK SHORELINE AND BLUFF PROTECTION STUDY

OFFSHORE REEF ALTERNATIVE PLAN VIEW

Figure 4-3



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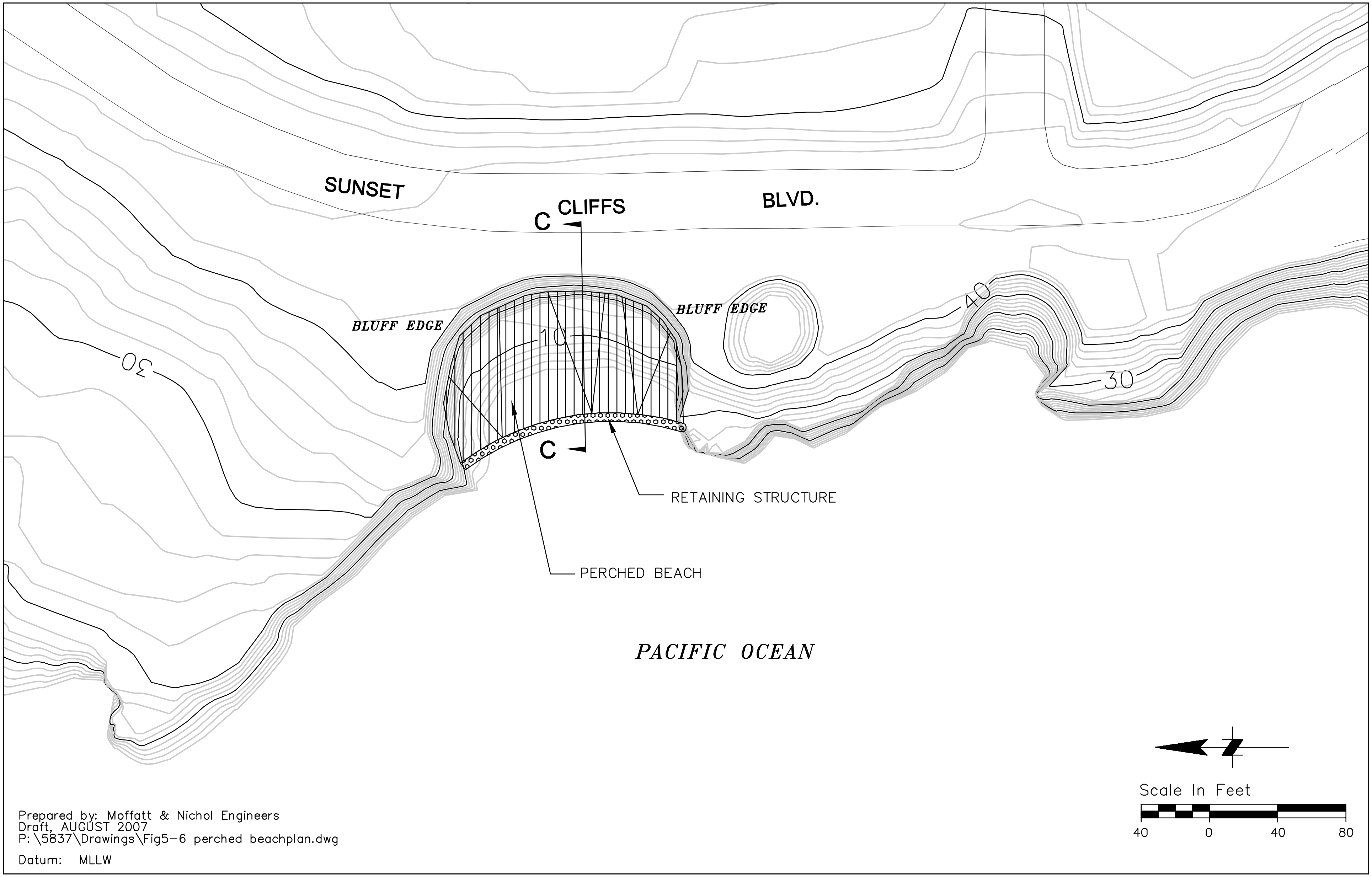
SUNSET CLIFFS NATURAL PARK SHORELINE AND BLUFF PROTECTION STUDY

OFFSHORE REEF ALTERNATIVE PROFILE VIEW

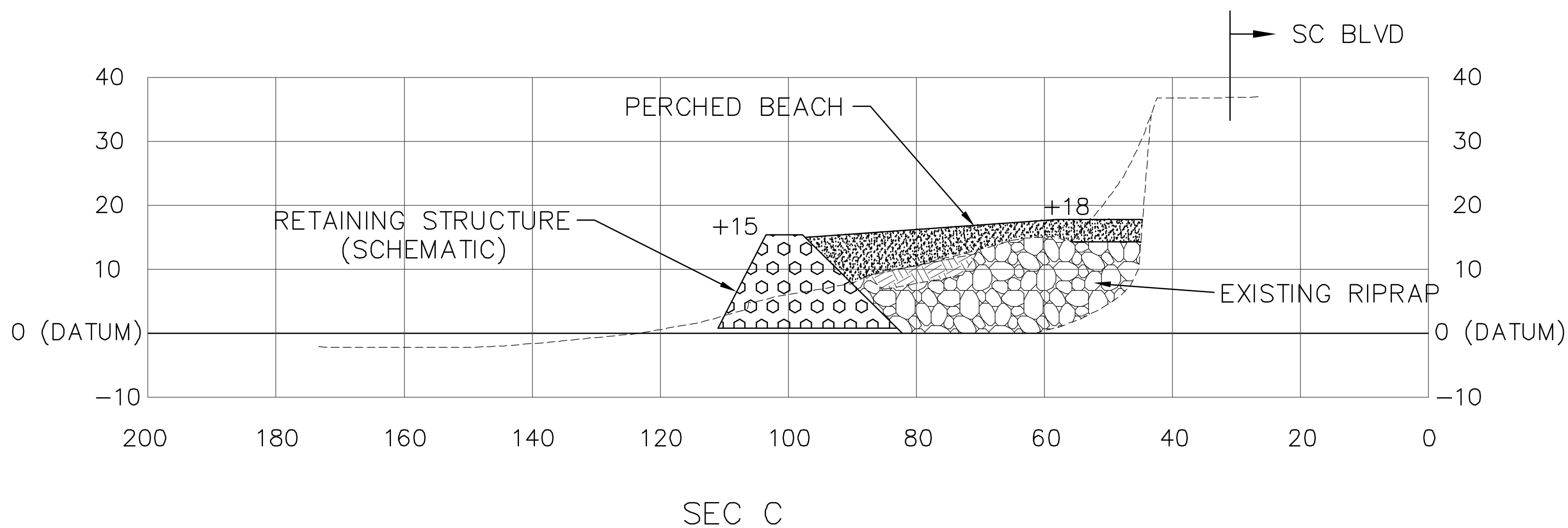
Figure 4-4



SUNSET CLIFFS NATURAL PARK SHORELINE AND BLUFF PROTECTION STUDY

PERCHED BEACH ALTERNATIVE PLAN VIEW

Figure 4-5



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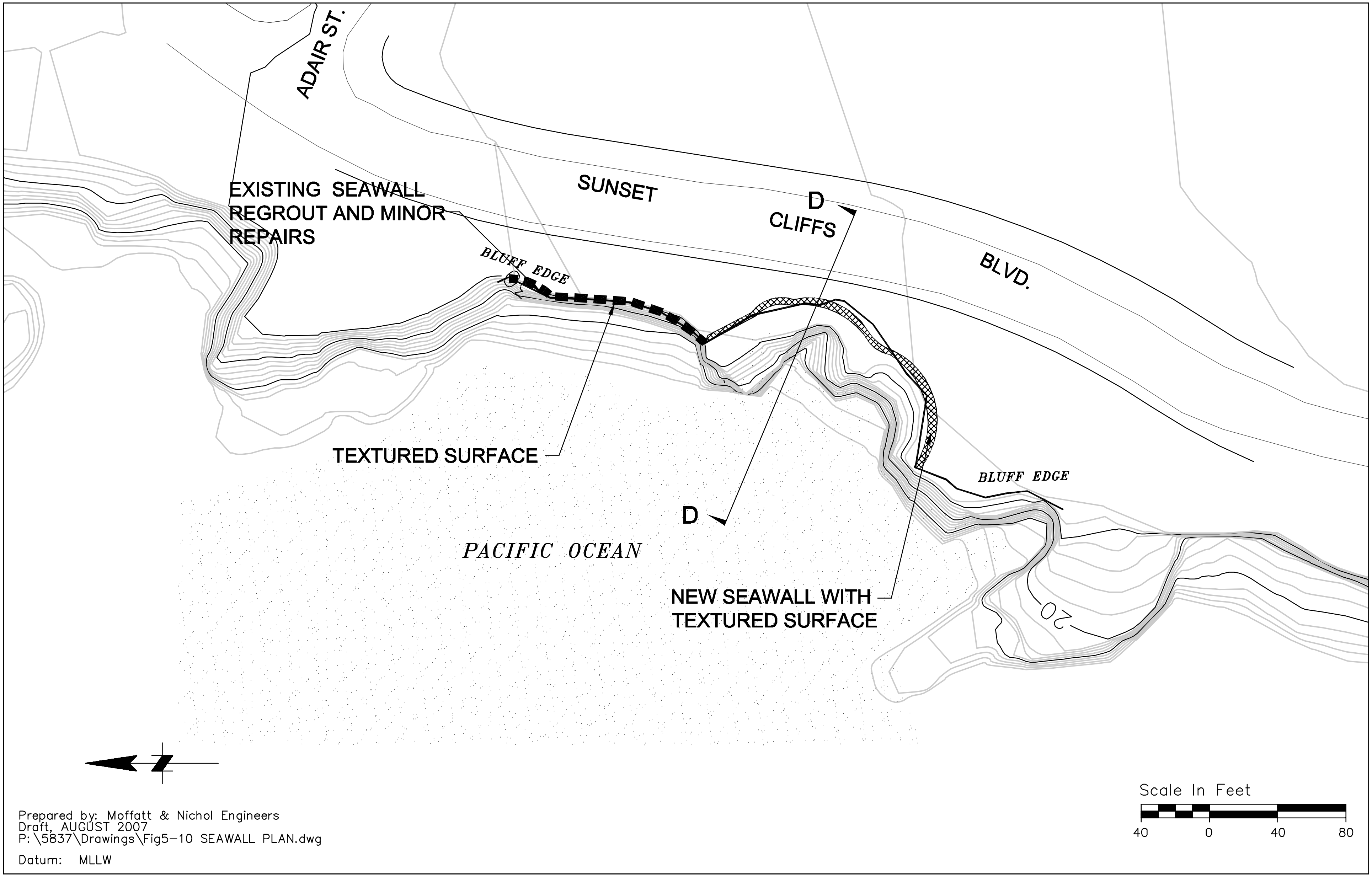
SUNSET CLIFFS NATURAL PARK SHORELINE AND BLUFF PROTECTION STUDY

PERCHED BEACH ALTERNATIVE PROFILE VIEW

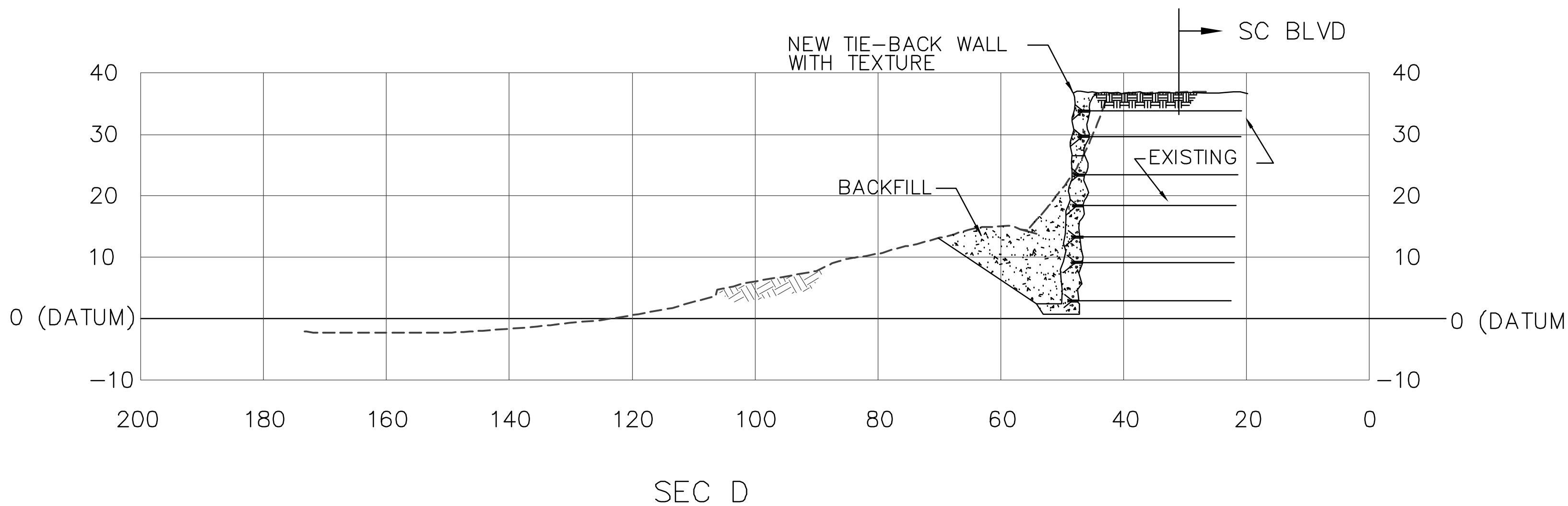
Figure 4-6



SUNSET CLIFFS NATURAL PARK SHORELINE AND BLUFF PROTECTION STUDY

TIE-BACK SEAWALL ALTERNATIVE PLAN VIEW

Figure 4-7



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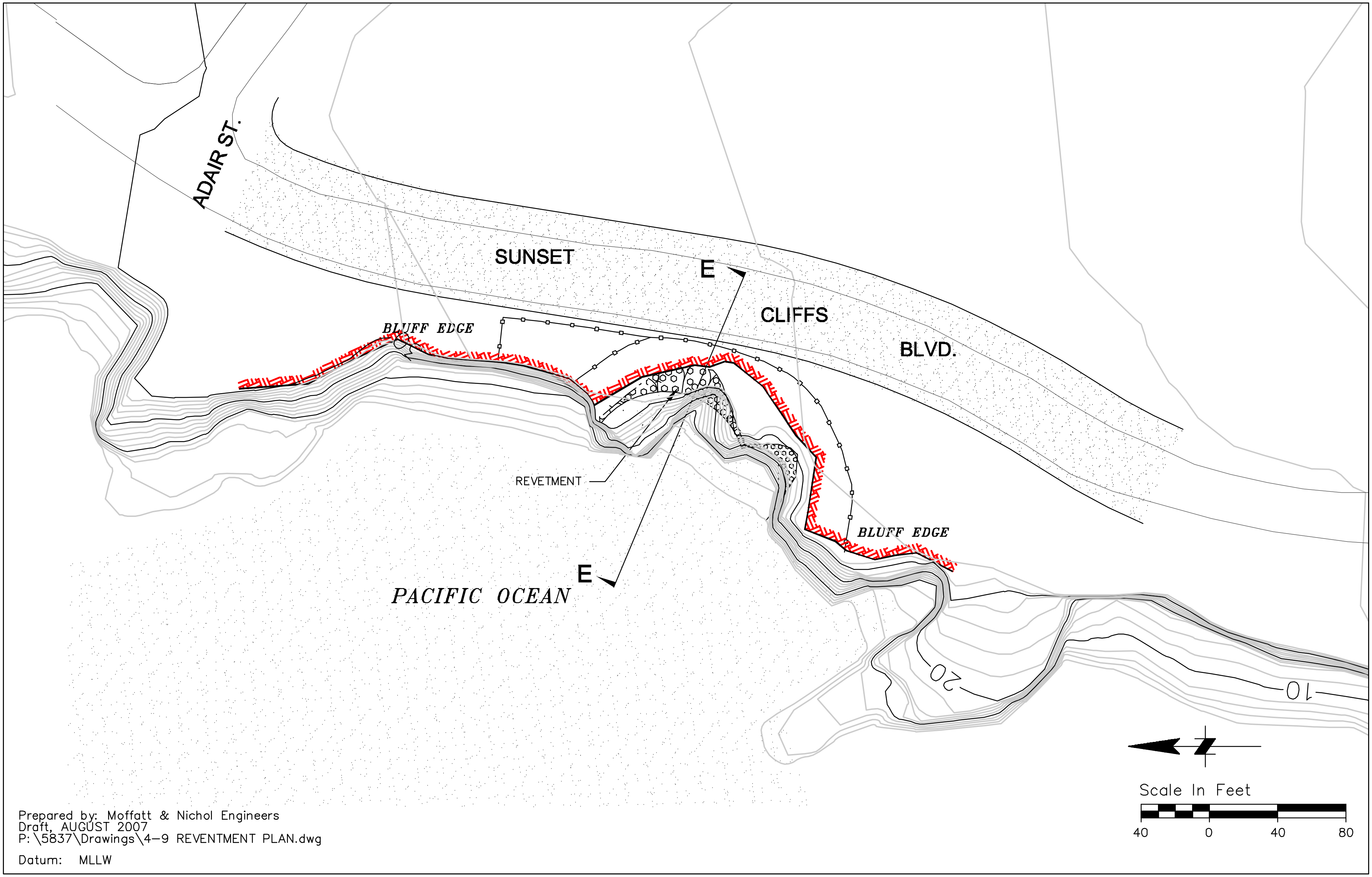
SUNSET CLIFFS NATURAL PARK SHORELINE AND BLUFF PROTECTION STUDY

TIE-BACK SEAWALL ALTERNATIVE PROFILE VIEW

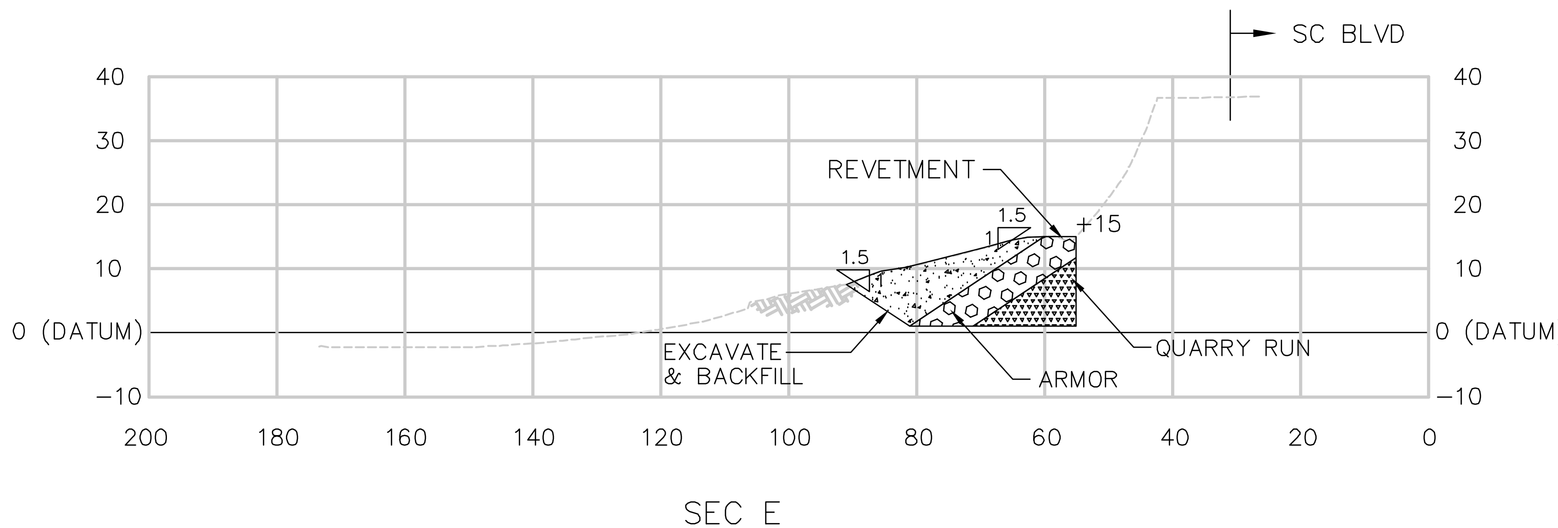
Figure 4-8



SUNSET CLIFFS NATURAL PARK SHORELINE AND BLUFF PROTECTION STUDY

REVETMENT ALTERNATIVE PLAN VIEW

Figure 4-9



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SUNSET CLIFFS NATURAL PARK SHORELINE AND BLUFF PROTECTION STUDY

REVETMENT ALTERNATIVE PROFILE VIEW

Figure 4-10

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5.0 EVALUATION OF BLUFF PROTECTION ALTERNATIVES

This section provides a general evaluation of the shore protection alternatives. Criteria used to evaluate the alternatives include constructability, permitability, cost, aesthetics, preserving public access, and public acceptance. All of these alternatives have the assumption that the storm drain system will be completely reconfigured to significantly reduce surface water runoff over the tops of the bluffs from low-flow and storm events. Once the surface runoff is significantly reduced, future bluff erosion will be greatly reduced since this is a major contributing factor to bluff instability along the Park.

5.1 No project

This alternative does not include any major beach of bluff protection structure, but may include minor repairs to the existing structures and minor restoration of failing bluffs that are undermining the existing road. The bluffs will continue to slowly erode from the coastal processes and from minor amounts of surface run off. Any minor repairs to the existing structures, such as minor grading at the bluff top, removing small portions of the concrete/rubble debris, seawall patching, and/or minor revetment redistribution could be completed. In some locations where Sunset Cliffs Boulevard is being undermined from surface erosion, the bluffs may continue to erode and the road may fail if no repair or protection is offered to the upper bluffs. Although the costs associated with the No Project Alternative are much lower than the other alternatives, it does not create or preserve public access to the park as there are already areas where the pedestrian path is directed into Sunset Cliffs Boulevard.

The No Project Alternative does not restore the aesthetic potential of the Park in that all of the riprap and the existing seawalls will remain generally as is. Future repairs will be needed as the bluffs continue to erode and this may result in a piece-meal patching approach that has been the major contributor to the aesthetic, public access, and public acceptance problems. It is foreseen that most minor repairs would not require extensive permits from the resource agencies in that there is minimal impacts to the environment factors (e.g., biology, traffic, recreation, air quality, noise, etc.).

5.2 Remove Existing Structures

This alternative includes removing existing shore-protection structures, where appropriate. It is important to note that this alternative is not feasible for the entire SCNP. Most areas are difficult to access and any removal would require an extensive effort and cost. Most locations where there is placed riprap at the base of the bluff would require a crane to pick up each rock individually. The equipment needed is generally very heavy and could cause bluff instability. There would also be the issue of local traffic impacts associated with the large equipment that would be needed to remove any structures.

Any area considered for removing the existing structures needs to be carefully evaluated. Immediately following the removal, the aesthetic character of the site will be improved, but the bluffs will be subject to coastal erosion from waves and tides and could erode more quickly than they have with the protective structures in place. The long-term consequences of any structural removal needs to be carefully considered.

5.3 Beach fill

The beach fill alternative will provide some initial protection of the bluffs along SCNP, however, periodic nourishment of at least every two years will be needed to maintain the beach area. Sand needed for beach fills could be trucked in or dredged and pumped from offshore borrow sites and further investigations would be needed to identify such sites. Grain size comparison would need to be factored in determining the best offshore borrow site. A nourished beach along this coastline is subject to more risk of erosion without any retention structures to help protect from erosion caused by waves and currents.

Beach fills would probably perform better in the smaller pocket coves along the reach, rather than a uniform beach fill along the entire SCNP. In these small pocket beaches, the beach fill could be placed directly over any existing shore protection structure (i.e., riprap revetments). This would aesthetically create a natural coastal setting, while preserving the protective element that the revetment structures provide. This alternative preserves public access and could create additional areas for the public to reach the shore below the bluffs. However, additional public access can jeopardize the stability of the bluffs.

Beach fills are a soft-solution, which are generally more acceptable to the general public and resource agencies. But the initial costs and re-nourishment cycle costs would make it a more expensive alternative. Also, because of the high density of offshore reefs and kelp beds, this alternative may be difficult to permit. As discussed in Section 4, a small-scale pilot project, followed by monitoring, may be the best approach to implement a beach fill along SCNP.

5.4 Nearshore Reef with Beach Fill

The nearshore reef alternative may provide some surfing opportunities as well as providing some protection to the bluffs along SCNP. The beach behind the structure would likely require periodic fills to provide erosion protection from wave action, however at a decreased rate compared to just a beach fill without any retention structures. Initial costs will likely be higher than the beach fill alternative, but periodic maintenance costs will be much lower because the renourishment cycle required will be over longer intervals. Access to the beach may be difficult in some areas, making constructability an issue. The sand used for the beach fill behind the reef structure would likely be pumped in from an offshore dredge operation.

Aesthetically, the nearshore reef can create a more natural looking setting with a sandy beach fronting the bluffs. Similarly to the beach fill alternative, the nearshore reef alternative would best be suited in smaller pocket coves than long stretches of beach. Also, the sandy beach area could be created over any existing revetment to preserve the last line of defense from severe erosive forces, while creating a aesthetically appealing sandy beach. The nearshore reef will generally preserve public access, however, in some locations it may create a public safety issue with the potentially changed wave climate. This alternative may be difficult to obtain permit approvals from the resource agencies for similar reasons as the beach fill alternative.

5.5 Perched Beach

The armor structure of the Perched Beach Alternative could be a safety hazard to surfers and other water users because it creates a steep drop-off from the upper beach area. Similarly to the

Nearshore Reef Alternative, the beach would require periodic beach fills to maintain erosion protection. The sand would likely be pumped to the beach from an offshore dredge operation. The armor structure could be difficult to construct because of the wave climate in the area and equipment that would be needed to place the armor units.

Aesthetically, the beach could cover the existing riprap revetments and create a dry beach area, but the seaward edge of the beach area would be bound by a hard armor structure. It could be possible to color and texture the seaward armor structure to closely match the existing nearshore reef system making it more visually appealing. This alternative has similar permit issues as the Beach Fill Alternative and Nearshore Reef Alternative and resource agency approvals will be difficult to obtain. The armor structure may also collect debris from wave action and be trapped on the landward side of the armor. However, the widened beach provides greater recreational opportunities with the increased access.

5.6 Vertical Tie-Back Sea Wall

The Vertical Tie-Back Wall Alternative has much lower total costs (initial and periodic maintenance) than the beach fill alternatives (with and without retention structures). Since no additional sand will be placed on the beach with this alternative, it may be less obtrusive to sensitive resources and acquiring agency approvals may be easier than the neashore reef and perched beach. This is mainly because of the understanding of how seawalls, revetments, and beach fills perform in this environment (nearshore reefs and perched beaches are not common alternatives and not understood as well). However, permitting will still be difficult.

The wall can be colored and textured to look like a natural bluff face and blend with the surrounding cliffs. A seawall can be constructed and backfilled to widen public access along the crest of the bluffs, if needed. Generally, a seawall will not decrease public access along the top of the bluffs, but may limit public access to the beach below the bluffs. Also, public acceptance of seawalls along SCNP has not been favorable, but it may be the best-preferred option in some specific locations.

5.7 Revetment

The revetment is the least cost alternative of all of the alternatives listed (except for the No-Project Alternative and potentially the removing of existing structures). The revetment will provide long-term protection to the upper road and bluffs and many locations along SCNP are already fronted by a revetment structure. The greatest impact is a reduced access to the small pocket beaches the aesthetic impacts to the Park, and public acceptance.

Permitting a revetment along this area would be extremely difficult in that there is already so much armoring along the Park, is the least desirable public alternative, will block public access, and the environmental impacts. Also, this alternative is the least desirable alternative for local residents because of the un-natural aesthetics of the design. The surface of the revetments could be treated with a colored and sculpted surface to minimize aesthetic impacts, however, this may impact wave runup on the structure and change the level of protection. Also, any sculpted surface will increase the costs of the structure.

5.8 Design Tasks

The following tasks should be completed prior to implementation of any alternative or as part of draft and final design:

a. Perform bathymetric surveys of the nearshore region.
b. Perform biological monitoring of marine resources nearby and adjacent to the Park.
c. Perform geotechnical explorations to determine limits of existing structures.
d. Initiate preliminary engineering and environmental studies.

6.0 SITE SPECIFIC APPLICATIONS

This section will describe site specific applications of the bluff protection alternatives. Because the revetment alternative is the least desirable in terms of aesthetics, cost, public acceptance, and permitability, it is not considered as a viable alternative for most locations along SCNP. The San Diego Coastal Erosion Site Assessment site designations are used in this section to describe alternatives that could be applied to specific locations along SCNP. These site designations are currently used by the City of San Diego to provide an on-going review of the City's coastal areas, reassess ongoing changes of the coast line, provide recommended actions, and assess the overall risk rating. The risk ratings are based upon field observations and conditions that present potential public hazards.

Site specific applications are illustrated in Figures 6-1 through 6-6.

6.1 Site 10 – Adair Street to Point Loma Avenue (to Northern boundary of SCNP)

Potential Alternatives Considered – Remove existing structures, Beach Fill, Nearshore Reef, Perched Beach

This section extends from Adair Street to Pt. Loma Avenue and is illustrated in Figure 6-1, however for this study only the section at Adair Street is considered because this is the northern limit of the SCNP. The Coastal Assessment Risk Rating for this area is Low. The site contains rubble fill placed in a small pocket beach area.

A new outfall structure is proposed in this location, and removal of the existing rubble may be needed to properly install the outfall. This could be an opportunity to restore this site to a more natural appearing coastline.

A small beach fill, nearshore reef, or perched beach could also be implemented in this small pocket beach. The beachfill could be used to cover some of the rubble, creating a natural looking pocket. Because of the pocket beach shape, the sand may be retained longer during normal wave conditions. However, if a strong storm came, the cross-shore transport could erode the beach fill material and expose the reef or rock riprap structure of the perched beach.

Photo 1 –Looking north at the Adair Street street end rubble, Site 10.

Photo 2 –At northern boundary of SCNP, looking south, Site 10.

6.2 Site 9A and 9B – Osprey Street to Adair Street

Potential Alternatives Considered – Remove existing Structures, Beach Fill, Nearshore Reef, Perched Beach, Seawall

This site extends from Osprey Street to Adair Street and includes Spaulding Point, two bluff-top parking areas, two seawalls fronted by rock revetments constructed by the Corps in the 1980s, and other rock riprap at the bluff toe placed by the Corps in the 1980s. Sites 9A and 9B are illustrated in Figures 6-1 and 6-2. The City's Coastal Assessment Risk Rating for this area is High along the northern reach where the two existing seawalls are located and Low at the Osprey Street street-end. Site 9B represents the section south of Spaulding Point to Osprey Street and Site 9A represents the area from Spaulding Point to Adair Street.

Much of the revetment at the base of the Osprey Street street end has been placed because of the large storm drain outfall. When the existing storm drain collection system is reconstructed a new outfall is proposed at this same location. However, all of the new outfalls will be located at the base of the bluffs to reduce the amount of storm water runoff over the top of the bluffs. Removal of the existing revetment could be achieved during the construction of the new outfall structure. Some type of dissipation system will be required with any new outfall (discussed in Section 7), and could potentially reuse some of this rock.

North of Spaulding Point, no other structures are recommended to be removed. Along this reach, beach fill, nearshore reef, perched beach and seawalls could be implemented. Similar to Site 8, the beach alternatives could be constructed in between the headlands in the pocket beach areas. The sandy beach could be placed directly over the existing revetment, creating a sandy pocket beach. The beach fill and fill behind the reef or perched beach structures would offer protection to the bluffs. However, creating a sandy beach may increase foot traffic to the beach. The pocket beach areas would help retain the material from normal longshore transport, but any significant storm could transport the sand cross-shore and it would need to be renourished.

Two seawalls currently exist along the reach. Generally, the walls are performing well. Some repair needs to be considered along the abutments. These existing seawalls could be modified and tiered to a lower elevation, and colored and textured to appear more like the natural bluff formation. A new seawall could be constructed in the area just south of the northern seawall. These two walls could be tied together to create a more uniform structure. The seawall could be colored and textured to blend with the natural bluff face.

Photo 3 –Southern seawall, just north of Spaulding Point, Site 9.

Photo 4 –Northern seawall showing location where a new seawall could be implemented, Site 9.

6.3 Site 8 – Froude Street to Osprey Street

Potential Alternatives Considered – Beach Fill, Nearshore Reef, Perched Beach

Site 8 extends from Froude Street to Osprey Street and contains two existing parking areas on the top of the bluff and is illustrated in Figures 6-2 and 6-3. The overall coastal risk rating for the site is High. An extensive seacave exists under the southern parking area. The bluffs are near vertical along this reach and are mostly fronted by revetment placed by the Corps in the early 1970s. The revetment is providing erosion protection to the bluffs.

Alternatives considered for this site include beach fill, nearshore reef, and perched beach. These alternatives are proposed in the pocket beaches between the two bluff-top parking areas. The beach fill and fill behind the reef or perched beach structures would offer protection to the large southern sea cave. Similar to other sites, creating a beach may increase foot traffic down the bluffs. The pocket beach areas would help retain the material from longshore transport, but any significant storm could transport the sand cross-shore and it would need to be renourished.

No seawalls are proposed along this reach, but upper bluff stability is recommended. This could include an injected grout or resin product as described in the previous sections.

Photo 5 – From northern parking area, looking south along Site 8.

Photo 6 –At southern parking area, looking landward toward cave entrance, Site 8.

6.4 Site 7 – Guizot Street to Froude Street

Potential Alternatives Considered – Beach Fill, Nearshore Reef, Perched Beach, Seawall

Site 7 extends from Guizot Street to Froude Street and has an overall Coastal Risk Assessment rating of High (Figure 6-3). About half of the site is fronted by a riprap revetment placed by the Corps in the 1970s. In April 2007, a bluff failure occurred on this reach (Site 7A), at about the mid-point of the site. Because of the High Risk Assessment Rating, removing any structures is not recommended.

Potential beach alternatives considered for the site include a beach fill, nearshore reef, and perched beach. However, at the northern end of No Surf Beach, only the beach fill alternative could be implemented. The other beach alternatives, nearshore reef and perched beach, are not recommended in this area for public safety concerns (see Section 6.5 for further discussion of Alternatives in northern No Surf Beach).

The beach area north of No Surf Beach and seaward of Froude Street is a shallow pocket beach and any of the beach fill alternatives (with or without retention structures) could be considered at this location. The revetment is providing protection to the base of the bluffs along this reach, and should not be removed. A beach fill could be constructed over the revetment along this site. The nearshore reef alternative and perched beach alternative could also be implemented along this reach of Site 7. Creating a beach along this reach may increase foot traffic from the public trying to gain access to the site.

A seawall is another alternative that could be considered along this site because of the critically eroded upper bluff face. Similar to Site 6, there are areas where the bluff has eroded under Sunset Cliffs Boulevard, and public access is directed into the road. A seawall could reclaim a little of that needed public access route along the bluff top. The seawall would need to extend from the base to the top of the bluff and could be colored and textured to blend with the natural bluff face. A less intrusive alternative to a seawall would be to provide some upper bluff stabilization, such as an injected grout or resin product. This would slow the upper bluff erosion, but would not restore the bluff face. This product would need to be reapplied every one to two years and would not hold up well to extensive foot traffic.

Photo 7 – Looking South at Site 7.

Photo 8 – Top of Bluffs, showing damage to curb, Site 7.

6.5 Site 6 – Hill Street to Guizot Street

Potential Alternatives Considered – Beach Fill, Seawall

This area extends from Hill Street to Guizot Street and contains No Surf Beach (Figures 6-3 and 6-4). This beach is one of the few sandy beach areas in SCNP and is a very popular recreational beach site. The City's Coastal Assessment Risk Rating for this site is High because of the upper bluff stability issues along the site. Much of this bluff erosion appears to be from storm water runoff over the top of the bluffs. This should be reduced significantly with the reconstruction of the storm drain system. Access to the beach is via an unimproved foot path that extends down the bluff face from an area at Sunset Cliffs Boulevard that is critically eroded. Foot traffic in this area is another significant contributor to the bluff erosion. The SCNP Master Plan does not include an access stairway at No Surf Beach, but unless foot traffic is stopped or an adequate access route is provided, continued erosion of the bluff face at this point will continue.

Alternatives for this site include a beach fill and seawall. Since there is already an existing sandy beach area, creating additional sandy beach is feasible. It would be recommended to extend the sandy beach area to the north, over the existing revetment riprap and into Site 7. Extending the beach fill to the south is not recommended due to the extensive nearshore rocky reef area. The nearshore reef and perched beach alternative are not recommended for this site because they can create a public safety hazard since this beach site is used extensively by the public and because they can impact the rocky reef habitat area.

Seawalls are another potential alternative because of the critically eroded upper bluff face. There are many areas where the bluff has eroded under Sunset Cliffs Boulevard, and public access is directed into the road. A seawall could reclaim a little of that needed public access route along the bluff top. The seawall would need to extend from the base to the top of the bluff and could be colored and textured to blend with the natural bluff face. A less intrusive alternative to a seawall would be to provide some upper bluff stabilization, such as an injected grout or resin product. This would slow the upper bluff erosion, but would not restore the bluff face. This product would need to be reapplied every one to two years and would not hold up well to extensive foot traffic.

Photo 9 – No Surf Beach, Site 6.

Photo 10 – North end of Surf Beach, Site 6.

6.6 Site 5 – Monaco Street to Hill Street

Potential Alternatives Considered – Remove existing structures, Beach Fill, Nearshore Reef, Perched Beach

Site 5 extends from Monaco Street to Hill Street and includes Luscomb's Point and is illustrated in Figure 6-4. The Coastal Assessment Risk Rating is Low for this stretch because of the relative stability of the bluffs and minimal erosion noted over the last 10+ years. There are extensive riprap revetments on the north side of Luscomb's Point and in the caves and coves to the south of the point.

Due to the lack of upland infrastructure on Luscomb's Point, the riprap on the north side of the point could be removed. Removing these structures may increase erosion on the north side of Luscomb's Point, but would return the setting to a more natural state. It is not recommended to consider removing the riprap located at the bluff toe at the end of Hill Street, since the bluff crest here is extremely narrow and encroaching into Sunset Cliffs Boulevard. None of the revetment structures to the south of Luscomb's Point should be considered for removal because of the risk of erosion to the existing seacaves.

The small pocket beach area south of Luscomb's Point is an applicable location for the beach fill, nearshore reef, and perched beach alternatives. The beachfill could be used to cover the revetment riprap, creating a natural looking pocket beach area. Because of the pocket beach shape, the sand may be retained longer during normal wave conditions. However, if a strong storm came, the cross-shore transport could erode any beach fill material and the rock riprap would become exposed.

The smaller pocket beach area south of the collapsed sea cave whole could also support a beach fill, nearshore reef, or perched beach. However, the sea arch cutting through the southern headland would need to be filled with rock or concrete to prevent any sand placed in the pocket beach area from being swept out by waves and tides through the arch.

Photo 11 – Small pocket beach just south of Luscomb's Point, Site 5.

6.7 Site 4 – Carmelo Street to Monaco Street

Potential Alternatives Considered – Remove existing structures, Beach Fill, Nearshore Reef, Seawall

The site from Carmelo to Monaco Streets is designated as a Moderate Risk Rating in the City's Coastal Assessment (Figures 6-4 and 6-5). Most of the bluff erosion is resulting from storm water run off over the crest of the bluffs. Once a new storm drain system is constructed, the rate of bluff erosion should slow considerably. The section is fronted by a long revetment constructed by the USACE in the 1970s. There is a gabion basket structure that was placed to accommodate the bluff-top drainage in this area. (Note: A gabion is a wire cage structure filled with cobble and/or stone.) This gabion structure could be removed after the reconstruction of the storm drain system in the Park. It is not recommended that the revetment structure be removed because of the narrow access between the bluff top and Sunset Cliffs Boulevard.

A beach fill or nearshore reef could be constructed along the reach over the existing revetment structure. Because of the long and straight coastal alignment, the beach fill alternative would not be expected to retain sand for long periods and frequent renourishment would be required. A nearshore reef along this area would be extensive, but could create a good surfing environment. Either alternative, although feasible, would be difficult to permit due to the extensive nearshore rocky reef that exists just offshore. A perched beach is not recommended at this site because of the site's long reach. Constructing a perched structure would be extremely expensive for such a long reach.

A seawall could be constructed along this reach, mainly because of the very narrow bluff top. In some areas, the bluff top is encroaching within the road way and public access is directed into the road. A seawall could reclaim a little of that needed public access route along the bluff top. The seawall would need to extend from the base to the top of the bluff and could be colored and textured to blend with the natural bluff face. A less intrusive alternative to a seawall would be to provide some upper bluff stabilization, such as a injected grout or resin product. This would slow the upper bluff erosion, but would not restore the bluff face. This product would need to be reapplied every one to two years and would not hold up well to extensive foot traffic.

Photo 12 – Site 4 view south of Monaco Street

6.8 Site 3 – Ladera Street to Carmelo Street

Potential Alternatives Considered – Revetment or Concrete Fill

Site 3 extends from Ladera Street north to Carmelo Street and includes a high and narrow bluff top fronted by armor and rock at the toe (Photo 4). Figure 6-5 illustrates the extent of the reach. This site has a risk rating of Low and the only recommendation is to fill the sea cave just south of Carmelo Street with either riprap or concrete, as recommended in the City's Coastal Assessment. Although no other alternatives are considered for this reach, the recommendation of the Coastal Assessment should still be considered because this seacave extends under Sunset Cliffs Boulevard. However, this is low-priority because of the slow rate of marine erosion. No other alternatives are considered viable for this section of coast. This is the only location where a revetment is considered applicable.

Photo 13 – Site 3 view north from Ladera Street Stairway

6.9 Site 2 – Ladera Street Access Stairway

Potential Alternatives Considered – Remove existing structures, Seawall

This site is the small area directly around the Ladera Street stairway (Figure 6-5). A new stairway is proposed as part of the SCNP Master Plan and therefore the existing stairway would need to be demolished and removed. Any new stairway may require a small, site-specific protective device to ensure its design life and stability. This could be a small seawall structure that could be incorporated into the stair structure design and not a stand-alone structure. The stairway external face could be colored and textured to match the existing bluff face, but this is probably not practical at this location since the prime vantage point of the stairs is from the top of the bluff.

Photo 14 – Site 2, Access Stairway at Ladera Street

6.10 Site 1 – Sunset Cliffs Park (South of Ladera Street)

Potential Alternatives Considered – None

This section extends from Ladera Street to the southern boundary of SCNP (Figures 6-5 to 6-7) and has an overall risk rating of Low. Photographs 1 and 2 are from Site 1. The City's Coastal Assessment does not contain any recommended action items. The majority of the erosion of the bluffs along this reach is from storm water discharge. Coastal processes are present, but do not contribute to the majority of the erosion issues present. Once the upland storm water system is redesigned and constructed, the bluff erosion will slow significantly. No shore- or bluff-protection alternatives are recommended for this site. The only structures that should be incorporated into this site are public access and properly designed storm drain outfall structure(s). The public access should be designed to prevent further bluff erosion from foot traffic.

Photo 15 - View North at Garbage Beach, Site 1.

Photo 16 - View South of Site 1

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ALTERNATIVES				
REMOVE EXISTING STRUCTURES (RM)		YES	NO	(RM)
BEACH FILL (BF)		YES	NO	(BF)
NEARSHORE REEF (NR)		YES	YES	(NR)
PEARCHED BEACH (PB)		YES	YES	(PB)
SEAWALL (SW)		NO	YES (EXISTING SEAWALL)	(SW)

SCALE: 1"=100'

6–1

(RM)	NO		NO	N/A	YES	NO			(RM)
(BF)	NO		YES	N/A	YES	NO	YES	YES	(BF)
(NR)	YES	YES	YES	N/A	YES	NO	YES	YES	(NR)
(PB)	YES	YES	YES	N/A	YES	NO	YES	YES	(PB)
(SW)	YES	YES (EXISTING SEAWALL)	NO			NO			(SW)

SCALE: 1"=100'

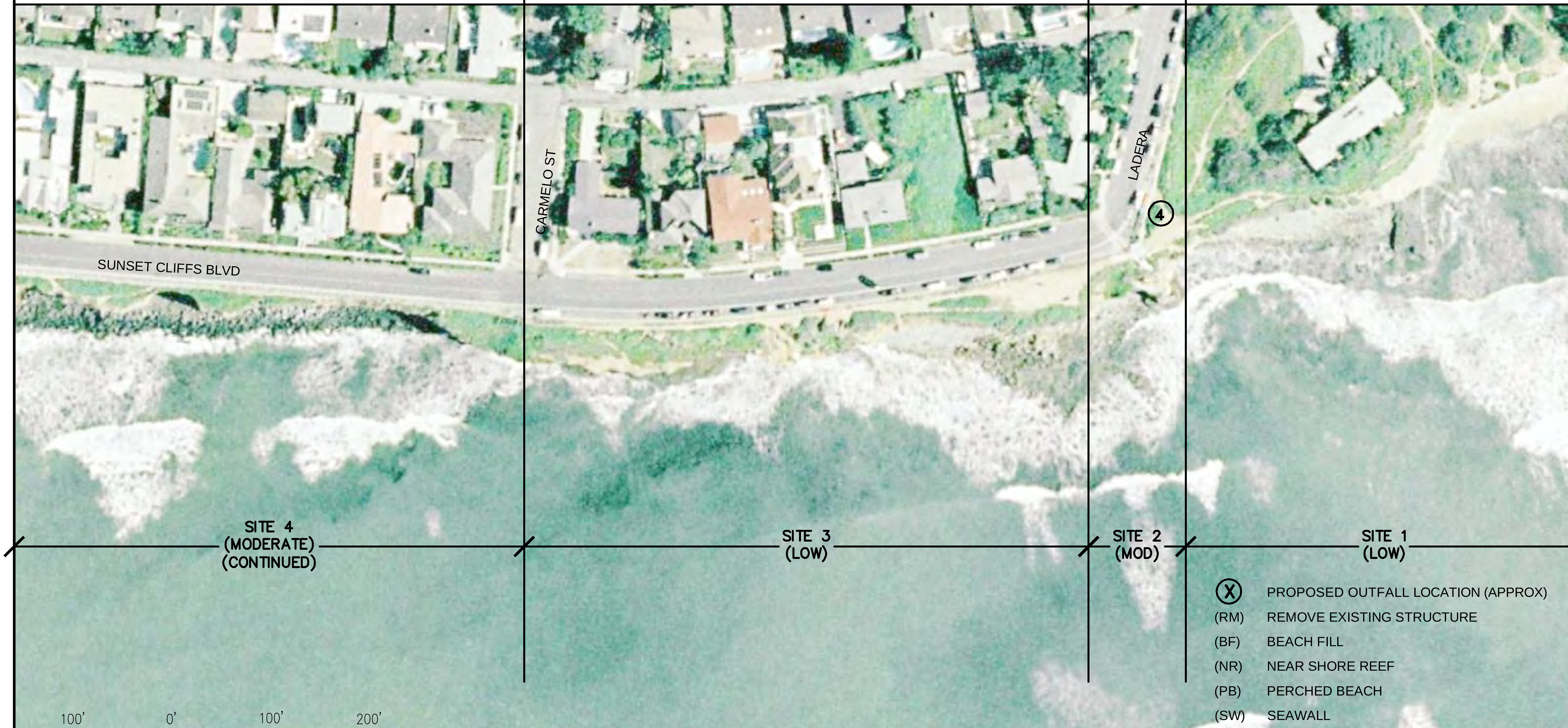
(RM)	NO			NO			NO	(RM)
(BF)	YES	N/A	YES	YES	NO	YES	YES	(BF)
(NR)	YES	N/A	YES	YES	NO	NO	NO	(NR)
(PB)	YES	N/A	YES	YES	NO	NO	NO	(PB)
(SW)	NO		YES	YES	NO	YES	YES	(SW)

SCALE: 1"=100'

(RM)	NO	YES	N/A	NO			NO			(RM)
(BF)	NO	N/A		YES	NO	Y	Y	NO	YES	(BF)
(NR)	NO	N/A		YES	NO	N	N	NO	YES	(NR)
(PB)	NO	N/A		YES	NO	Y	Y	NO	NO	(PB)
(SW)	YES	NO					NO		YES	(SW)

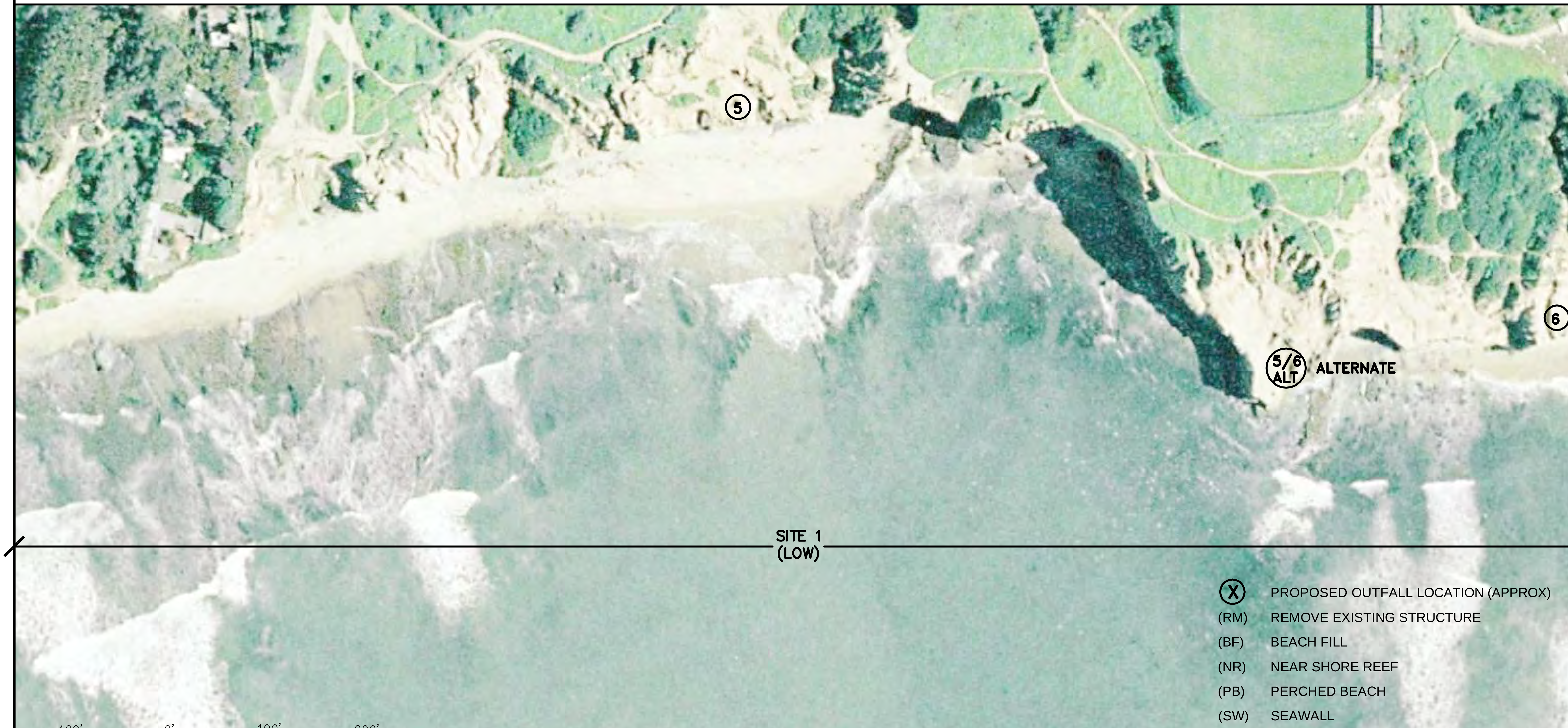
SCALE: 1"=100'

(RM)	NO	YES (ACCESS MAY BE DIFFICULT)	NO	NO	(RM)
(BF)	YES	NO	NO	NO	(BF)
(NR)	YES	NO	NO	NO	(NR)
(PB)	NO	NO	NO	NO	(PB)
(SW)	YES	NO	YES	NO	(SW)



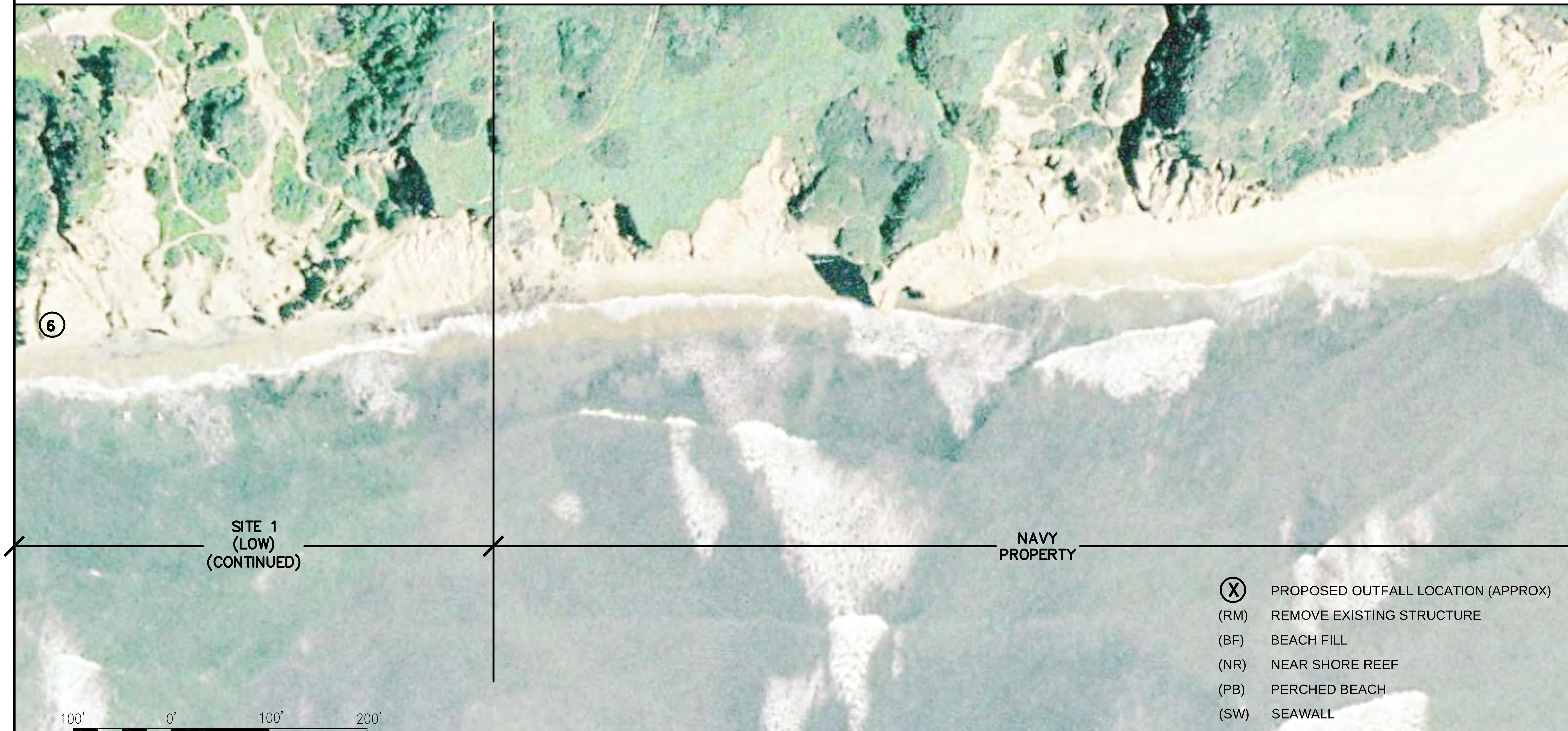
SCALE: 1"=100'

(RM)		(RM)
(BF)		(BF)
(NR)	NO ALTERNATIVES RECOMMENDED FOR THIS REACH	(NR)
(PB)		(PB)
(SW)		(SW)



SCALE: 1"=100'

(RM)		(RM)
(BF)		(BF)
(NR)	NO ALTERNATIVES RECOMMENDED FOR THIS REACH	(NR)
(PB)		(PB)
(SW)		(SW)



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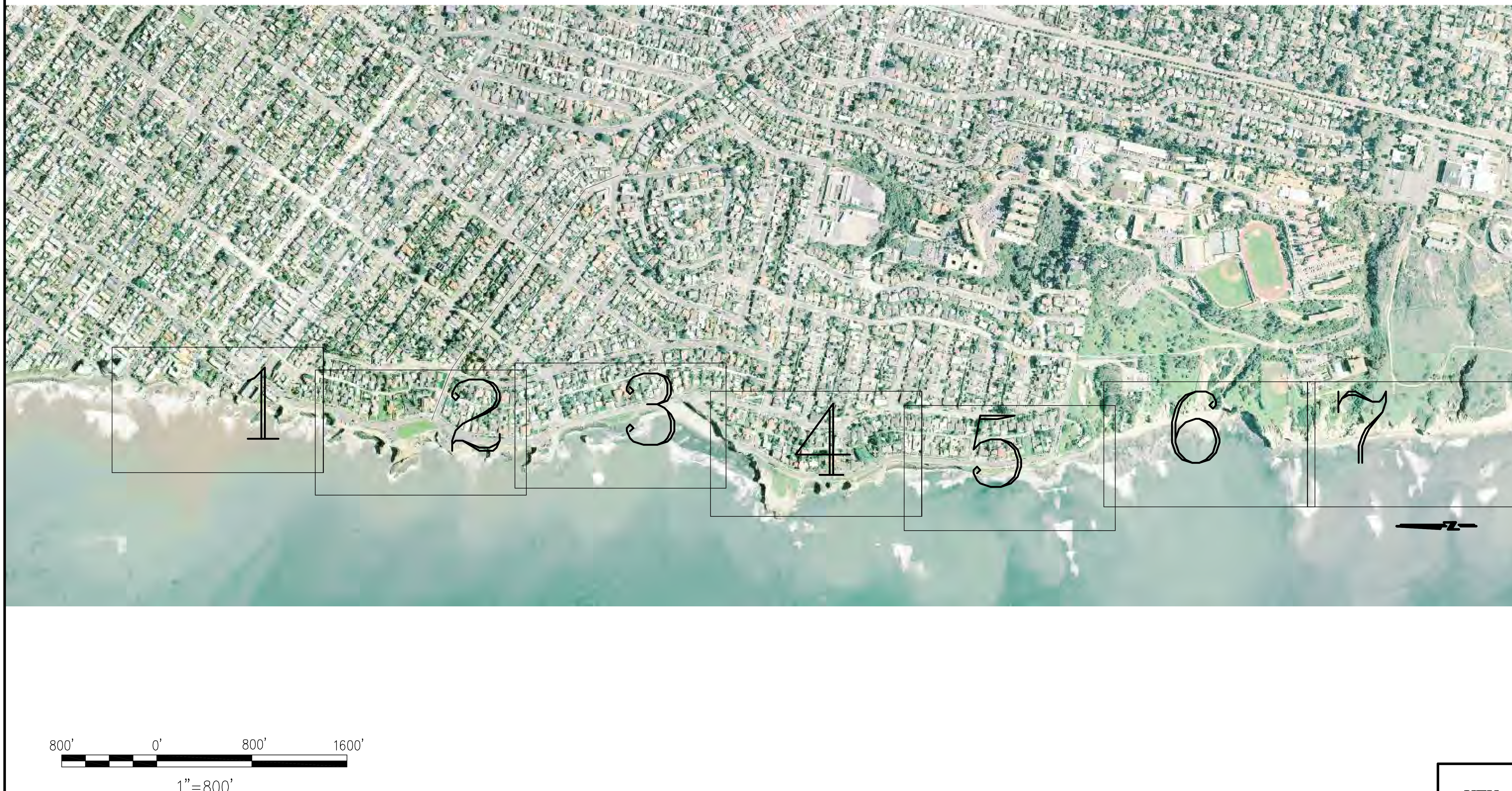

KEY

7.0 STORM DRAIN OUTFALL ALTERNATIVES AND EVALUATION

The storm water collection system located in SCNP is proposed to be redesigned and reconstructed to greatly reduce the amount of run off that occurs over the crest of the bluffs. This existing drainage and runoff is one of the major contributors of upper bluff erosion and instability along the Park. The majority of the upper bluff erosion along the Park is the result of surface runoff, overwatering of the bluff top, pedestrian traffic, and burrowing by animals. Coastal processes contribute to a portion of the lower bluff erosion.

Along Linear Park, the proposed storm water drainage systems will collect the water at catch basins along the western curb at Sunset Cliffs Boulevard and the outfall structures will be proposed at the base of the bluffs. The collection systems proposed in Hillside Park will also be directed to outfall locations at the base of the bluffs.

Currently, six outfall locations are proposed; with four in Linear Park and two in Hillside Park. These approximate locations are (1) at the Adair Street street end, (2) at the Osprey Street street end, (3) at the Monaco Street street end, (4) at the Ladera Street street end, (5) toward the southern end of Garbage Beach, and (6) approximately 400-500 feet north of the SCNP southern property boundary. An alternate outfall is proposed that would combine the flows from #5 and #6 is located at the headland point at the southern end of Garbage Beach. Figures 6-1 through 6-6 indicate the locations of the proposed outfall structures.

This section will present outfall alternatives that could be implemented, evaluate the alternatives, and discuss site-specific applications for each of the outfall locations.

7.1 Design Criteria

The outfall structure is an essential element to any storm drain system, especially along the open coast. It is not just the outfall structure, but the energy dissipating design and outlet protection that are the key components of the outfall. The purpose of the outfall structure along SCNP is to direct the storm flow away from the bluffs and prevent major erosion of the beach and bluff. The main method for reducing and preventing erosion is to lower the excessive flow velocities and direct the flow away from the bluff face.

Design parameters to be considered include:

1. H = Energy head to be dissipated, feet (can be approximated as the difference between channel invert elevations at the inlet and outlet).
2. Q = Design discharge, cubic feet/second
3. v = Theoretical discharge velocity determined from 2 g H, feet/second
4. A = Flow area, Q / v, feet^2
5. d = Flow depth entering the basin, ft
6. Fr = Froude number = v / (g d)0.5, dimensionless
7. g = Gravitational constant = 32.2 feet/second^2

7.2 Storm Drain Outfall Alternatives and Evaluation

The outfall culvert or pipe, energy dissipator, and any erosion-protection structure should be designed as an integrated system. Each will play an important role on the other. For example, lowering the flow in the pipe may change the design requirements of the energy dissipator and possibly the protective structures. It is important to note that the type and size of an outfall system is primarily dependent on the flow velocity. Because of the narrow area fronting the bluffs along SCNP, lowering the flows in the pipe before it exits to the dissipating structure may be key in limiting the size of the overall outfall and dissipating structures. There will be a balance between the cost and size of the pipe (size and internal dissipation) and the cost and size of the outfall structure.

Outfall structures are generally constructed of concrete and dissipating structures can be constructed of concrete or rock riprap. Erosion protection should be incorporated into the design of the outfall to prevent the structure from failing due to coastal processes acting on the structure and/or from storm water flow velocities.

Criteria used to evaluate the outfall alternatives include constructability, permitability, cost, aesthetics, preserving public access, minimizing beach and bluff erosion, and public acceptance. It is important to note that the new storm drain system at SCNP will greatly improve the bluff erosion problems by limiting the amount of storm water flow over the top of the bluffs.

Alternatives presented in this section to lower flow velocities can include channel or pipe linings, a dissipating structure, or flow barrier.

7.2.1 *Internal/Integrated Dissipators*

One method for reducing the flow velocity is by increasing the roughness inside the discharge pipe. The roughness elements placed inside the culvert barrel can decrease the drainage velocity by creating a series of hydraulic jumps and can be an optimum dissipator on steep slopes. The series of hydraulic jumps and overfalls within the culvert maintain the flow at approximately critical velocity even on slopes that would otherwise be characterized by high supercritical velocities. A major concern with this alternative is that silt may accumulate in front of the roughness elements, making them ineffective. A schematic of this is presented in Figure 7-1.

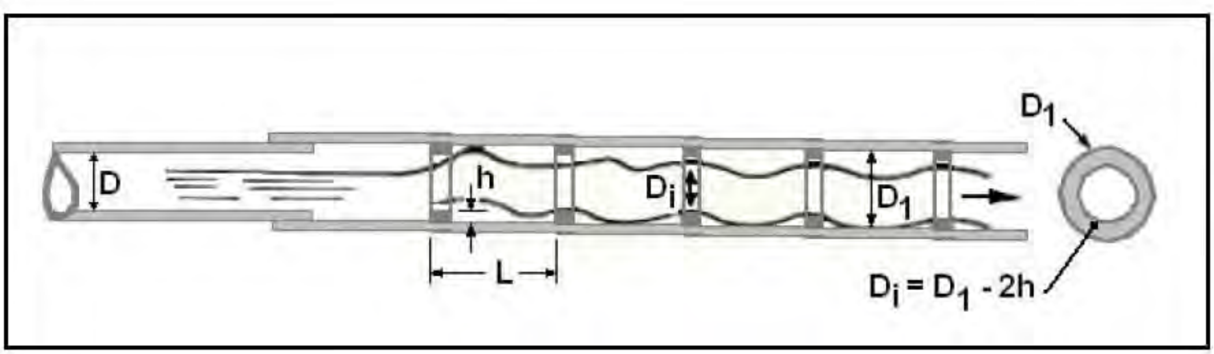

Figure 7-1. Tumbling Flow in Circular Culvers (USDOT 2006)

This alternative should be investigated for each of the outfall locations as it may slow the storm water flow before it reaches the external dissipating structure. A slower flow may result in a small, less obstructive structure at the bluff base, and may result in a more aesthetically appealing and publicly acceptable system. However, slowing the flow too much during extreme storm events may cause the pipe to back up and not operate correctly. Proper sizing and design of the pipe is critical and should focus on design flow volume and velocity.

7.2.2 *External Dissipators*

External dissipators can be constructed from stone, riprap, or concrete structures. By relocating the outfalls to the base of the bluffs provides an opportunity to restore the bluff tops and restore public access along the bluffs. Although a structure located at the base of the bluff may limit lateral pubic access along the beach, most areas along SCNP have limited lateral beach access because of the steep bluffs and curvatures in the coast. It is important to place or design the outfall in a location that will not cause or exacerbate beach erosion. This may involve locating the oufalls at or near adjacent headlands.

Generally the use of erosion control stone for energy dissipation is limited to a maximum flow velocity of 19 feet per second (fps). If the flow is greater than this, then a riprap outlet apron or concrete baffled outlets may be required. The riprap outlet reduces the exist velocity of the flow by expanding the flow over the riprap area. A rock or riprap dissipating structure should be designed with an erosion toe to protect from undermining scour at the end of the pad. The size of the rock apron will be dependent on the flow velocity and volume. Schematics of the rock/riprap dissipation structure are illustrated in Figure 7-2.

Baffled Outlets consist of a concrete box structure with a vertical hanging concrete baffle and an end sill. These structures dissipate the flow energy through impact of the water hitting the baffle and through the turbulence that results. This type of outlet protection can be used with outlet velocities up to 50fps and the size of the structure will depend on the volume and velocity of the flow. A schematic is illustrated in Figure 7-3.

Aesthetically, either external dissipating structure (riprap or concrete baffle) will not blend with the natural surroundings of SCNP. Grouting, texturizing and coloring the rock, riprap, and/or concrete can create a surface that can blend with the bluff, beach, and/or nearshore rocky reef surroundings. Using this texturing method will greatly restore the aesthetics of SCNP, especially after the existing storm drain pipes are removed from the top of the bluffs.

The size of the external dissipating structure is dependent on the flow velocity, but should be kept to a minimum to preserve the aesthetic impacts. Any method to reduce the flow velocity before it exits to the dissipating structure will help in reducing its size. Also, the outlet structure should not create an obstacle to public access.

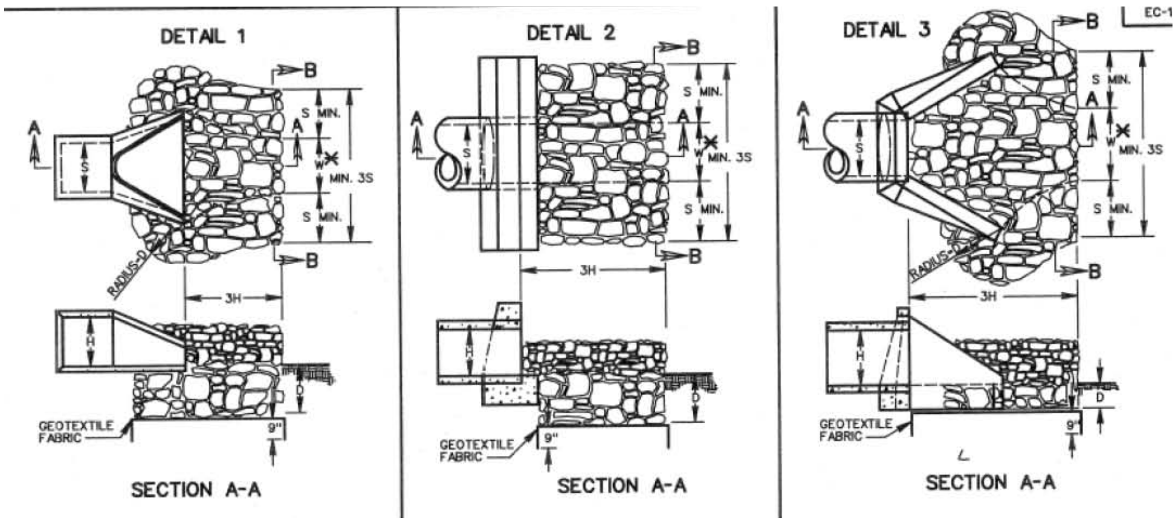

Figure 7-2. Stone/riprap energy dissipator schematic. (County of Roanoke 2006)

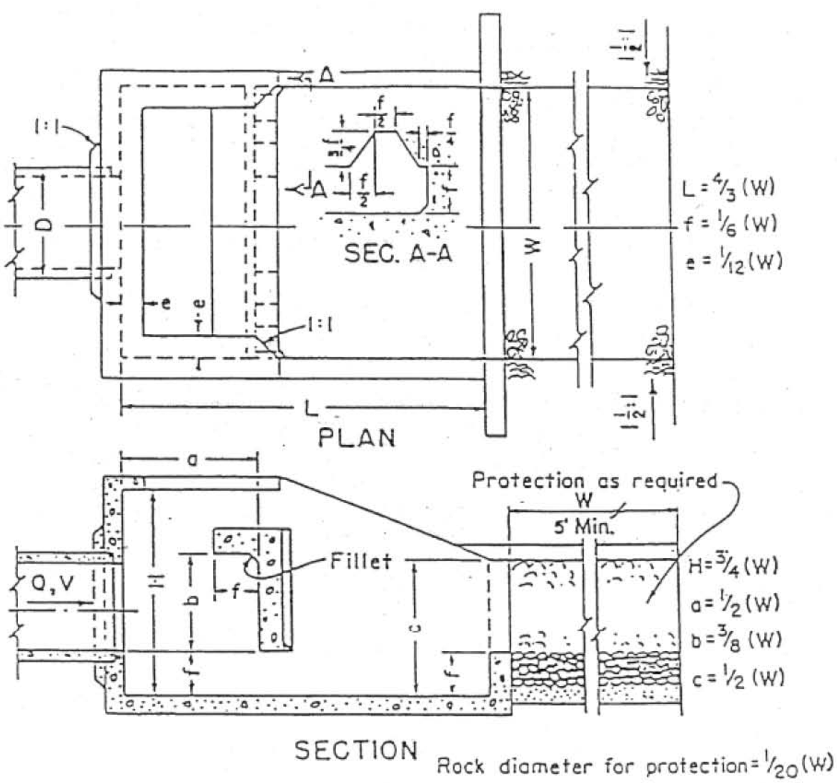

Figure 7-3. Schematic of Baffled Outlet

(County of Roanoke 2006)

7.2.3 *Extend outfall pipe into the Nearshore*

This alternative involves extending the storm water pipe from the base of the bluffs across the shore to the nearshore area. This is illustrated in the figure below (elevations and dimensions not applicable for SCNP). The structure could be buried under existing riprap or sandy beach and protrude out below the Mean Low Water Elevation. Extending it out below the Mean Low Water will minimize its aesthetic impacts of the area since it will be exposed under the water. It is possible for the pipe to become exposed as seasonal shifts in beach width occur. However, the pipe could be covered with a colored and textured surface to mimic the natural nearshore rocks and reefs that exist along the coast.

This alternative may be difficult to construct if the existing beach is eroded down to hard substrate rock. However, it is probably the least expensive alternative from a materials standpoint. Aesthetically, the pipe would be buried and not exposed, resulting in a more-natural coastal setting at the outfall location. An outfall structure may be a public hazard if the beach area is frequented by swimmers and/or surfers and should not be proposed at these publicly used locations.

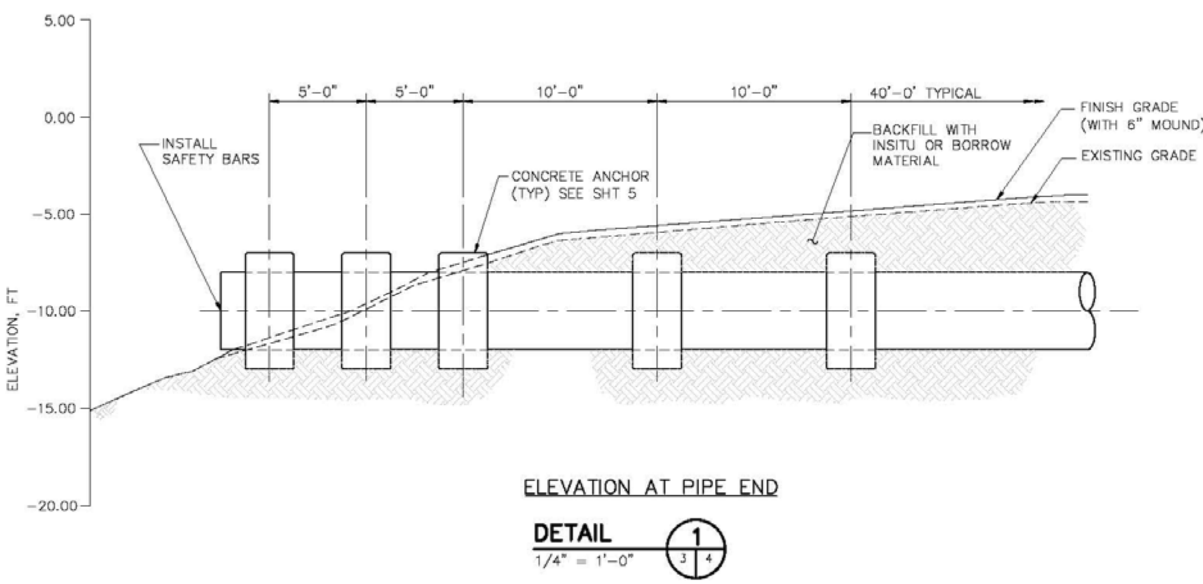

Figure 7-4. Example of pipe outfall extending below the waterline.

7.3 Site Specific Applications

7.3.1 *Outfall #1*

The proposed location for Outfall #1 is located at the Adair Street street end. This site is a small (approximately 80-foot wide) pocket beach at the northern boundary of SCNP that is bound by a natural headland to the south. The back beach and bluff consists of rubble fill. This area is a good location for an outfall structure, because it is confined, does not see a lot of public access at the beach below, has a wide area on the crest for equipment access, and does not have a high vertical drop from bluff top to base. Also, an outfall at this location could provide an opportunity to remove some of the concrete and rubble debris during the construction of the outfall. The area could then be backfilled with a properly designed fill that blends with the natural bluff face just to the south.

Because the pocket beach is narrow, the alternative best suited would be to extend the outfall pipe into the nearshore, past the southern headland. This would prevent any storm water flow to erode the headland to the south. The outfall pipe could be buried under the existing sand and covered with at textured surface to blend with the natural sandy and nearshore rocks.

Riprap or concrete dissipating structures could also be constructed here, but would need to be designed to ensure that the high-energy flow did not contribute to erosion of the headland area.

Photo 17 – Adair Street street end.

7.3.2 *Outfall #2*

Outfall #2 is located at the Osprey Street street end. This site is similar to Adair street, in that it appears to be a good location from a constructability stand point and also may provide an opportunity to remove some of the riprap at the base of the bluffs, depending on the type and design of the outfall structure. The parking area to the south would be an ideal location for staging and equipment storage.

Alternatives that could be applied that this location include a riprap apron or concrete dissipating structure and extending the outfall into the nearshore area. Internal dissipation may be applicable at this site because of the longer length between the catch basin location and the outfall location. Some of the existing riprap could be used to construct a riprap dissipating apron, however the structure should be constructed as far seaward as possible to prevent scour erosion at the base of the bluffs to the north and south. The eroded bluff could be restored with backfill material. Public access impacts resulting from an outfall structure are not an issue at this site, because public access to the beach is currently limited under existing conditions. The outfall should be aligned to minimize aesthetical impacts from viewers up- and down-coast and appropriate color and texturizing would help minimize any visual impacts.

Photo 18 – Osprey Street street end.

7.3.3 *Outfall #3*

The location of Outfall #3 is proposed at the Monaco Street end. This site is very similar to the Outfall #1 and #2, however, access to this site may be more difficult. The bluff top is narrow, higher, and limited staging area is available for equipment access to the beach. Traffic control will be an issue around this site because of the narrow upland working area available. Riprap is located at the base of the bluff and some could be removed during the construction of an outlet at this location. However, it may require a crane operation to move, remove, or realign the riprap. Some of the existing riprap could be re-used for a riprap apron dissipation structure at the base of the bluff.

However, there is concern that an outfall located at the base of the bluff could create flow conditions that can contribute to the erosion of the headland area to the south of the cove (see Photo below). It may be best to situate the outfall structure further seaward, near this headland to minimize erosion from storm water flow. The site may be better suited to extend the outfall pipe into the nearshore, past the southern headland. This would prevent any storm water flow from increasing the erosion to the bluff face and headland. The outfall pipe could be buried and covered with a textured surface to blend with the natural seascape.

Photo 19 – Monaco Street street end.

7.3.4 *Outfall #4*

The Ladera Street Access Stairway is the proposed location for Outfall #4. This is the most-difficult location along Linear Park for construction access and staging. The bluffs here are near vertical and higher than the locations to the north. It would be best to incorporate the outfall structure into the proposed new stair structure to minimize the number of structures in the coastal environment. This site would be best suited with a riprap apron or concrete dissipating structure that can be incorporated into the base of the new stairway design. Some of the existing rock located at the bluff base could be incorporated into the structure, if needed.

Extending the outfall into the nearshore area is not advised at this location because of the public safety impact this may pose to the surfers and other beach users accessing this location.

Photo 20 – Ladera Street street public stairway.

7.3.5 *Outfalls #5 and #6*

Outfall #5 is located toward the end of the sandy beach area of Garbage Beach and shown in Photo 21 and Outfall #6 is located around the headland point, and approximately 450 feet north of the SCNP southern property boundary (Photo 21). Both of these locations are similar in that there is a sandy beach below a very high bluff. Access to the beach would be extremely difficult from either location. A very large crane or helicopter may be required to bring equipment and supplies to the beach for construction of an outfall structure.

An outfall extending into the nearshore is not recommended because of the risk of the pipe being exposed during times of low sand deposition. This could create a public safety hazard and would not be very aesthetically pleasing or publicly acceptable.

A concrete or riprap dissipation structure would be the two better alternatives for an outfall at either of these sites. If a beach-access route is incorporated into either of these locations, then the outfall structure could be incorporated into the design, minimizing the number of structures along the coastline. It is noted, that during times of high flow velocity, beach scour may occur from the outfall across the beach to the waterline. The southern end of Garbage Beach has some exposed bedrock that could help serve as a natural scour prevention surface, but some of the sandy beach area around the outfall structure and to the waterline will scour unless the apron is adequately designed. To minimize the risk of beach scour, the dissipating structure should be designed adequately, which may result in a larger structure along the bluff base. Incorporating internal energy dissipators in the outfall pipe can reduce the velocity of the pipe and may help in reducing the size of the structure.

Photo 21 – Photo showing Garbage Beach (Outfall 5 location on S. end).

Photo 22 – Site for Proposed Outfall 6.

7.3.6 *Outfall #5/6 Alt*

This is a combined outfall to replace #5 and #6 and is proposed at the headland area at the south end of Garbage Beach. Although constructability, cost, and construction access access may be more of an issue because the headland extends further into the Pacific, it may be a more suitable location from environmental, aesthetic, public acceptance, and public beach access aspects.

An outfall structure could be constructed into the base of the headland which would direct the storm flow out toward the ocean and minimize any beach scour impacts from the storm flows. The flows could be slowed using internal dissipating devices inside the discharge pipe before it gets to the outfall structure, which may result in smaller design for the outfall dissipating structure. Either a concrete or riprap apron structure could be incorporated into this outfall location. The structure could be colored and textured to match the existing seascape.

Photo 23 – Site for Proposed Outfall 5/6 Alternative.

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8.0 ENVIRONMENTAL/PERMIT ISSUES

Environmental issues and necessary permits required for any project-proposing beach fill and/or retention measures are summarized below. Permitting for any coastal project is a time-intensive endeavor with many diverse opinions regarding implementation. From beginning to permit issuance generally takes from three to five years to accomplish.

8.1 National Environmental Policy Act (NEPA)

Any proposed project within U.S. waters are under Federal jurisdiction and require the preparation of an appropriate environmental document for environmental review. The document can be either is a Finding of No Significant Impact (FONSI) or an Environmental Impact Statement (EIS), depending on the anticipated scale of environmental impacts. A FONSI pertains to projects with relatively negligible impacts on the environment and an EIS is required if environmental impacts are anticipated to be potentially significant.

8.2 California Environmental Quality Act (CEQA)

State law requires environmental review of all projects within State jurisdiction. This review, prepared by the proponent, can be documented in either a Negative Declaration (ND), Mitigated Negative Declaration (MND), or an Environmental Impact Report (EIR). Similar to the Federal law, a ND and MND pertains to projects with relatively negligible impacts on the environment while an EIR is required if environmental impacts are anticipated to be potentially significant. All government agencies with jurisdiction over the project are required to review and comment on the CEQA document prior to issuing permits. The certified final CEQA document will have to be submitted as part of the permit application packages in order for the applications to be deemed complete.

8.3 Sections 10 and 404 permit from the U.S. Army Corps of Engineers

Projects in navigable waters require review as to their conformance with Federal Guidelines under Sections 10 and 404 of the Rivers and Harbors Act. This requires the submission of a completed permit application form accompanied by a project description (area, material volume, wetlands affected, etc.) and drawings. In addition, the USACE performs an environmental assessment to determine the need for a FONSI or EIS. The appropriate document is prepared and the permitting process extended a minimum of six months to a year.

The issuance of this permit is conditional on obtaining approvals from the local City, California Coastal Commission, and Regional Water Quality Control Board as discussed below. No separate permits are required from the U.S. Fish & Wildlife Service as the USACE consults with this agency prior to issuing its permit.

Beach fills, nearshore structures, revetments, and seawalls will all require approval from the USACE. Potentially, some removal projects may require Federal approval.

8.4 Coastal Development Permit from the California Coastal Commission

The authorizing jurisdiction is responsible for complying with the California Coastal Act. The City has the permit discretion from the Coastal Commission. The Commission requires a Final CEQA document and issuance of local approvals prior to issuing a state-level permit. A public hearing is held by the Commission, requiring a minimum of 6 to 8 weeks prior to issuance of the permit in California. This time frame can be affected by requests for continuance by the applicant or Commission staff.

Beach fills, nearshore structures, revetments, and seawalls will all require approval from the Coastal Commission. Potentially, some removal projects may require Coastal Commission approval also.

8.5 Water Quality Permits

The Regional Water Quality Control Board (RWQCB) will require the following permits.

8.5.1 *Section 401 Certification of the Clean Water Act.*

Any project that requires a Section 10 or 404 from the USACE will undergo review under provisions of the Federal Clean Water Act. An application is to be submitted to the RWQCB that includes a letter describing the project, drawings, steps to be taken to minimize water quality impacts, beneficial uses of the affected water body, and copies of the USACE 404 permit application and CEQA\NEPA document. The RWQCB does not act on the 401 certification until the FEIR/EIS is certified. The application process takes a minimum of 60 days upon which the region makes a recommendation to the State Board and the State issues the certification after several weeks. Application fees are determined on the amount of fill or length of project and can be a minimum of $500 or a maximum of $40,000.

8.5.2 *Storm Water Permits*

Most alternatives presented herein will require both the General Construction Activity Storm Water Permit and General Industrial Activity Storm Water Permit for construction and operation. Both these permits require completion of a Notice of Intent to Discharge (NOI) form, and preparation and implementation a Storm Water Pollution Prevention Plan (SWPPP). Generally, the construction SWPPP is less detailed and mainly requires adequate erosion control measures. The industrial activity SWPPP requires more intensive measures, such as regular sampling and testing of storm water runoff. It is anticipated that storm water drainage will not be incorporated into this project, so a NOI will not be required.

8.6 Lease of State Lands from the California State Lands Commission

A lease of State Lands will likely be required for most alternatives presented herein from the State Lands Commission (SLC) unless the area has already been granted lands to the City of San Diego. The SLC requires a topographic survey of the mean high tide line prior to project implementation to determine the pre-construction state land boundary (seaward of MHW). These surveys may already exist for portions of the City. A $3,025 filing fee is required.

8.7 Monitoring

Monitoring of the alternatives will be required and will consist of construction monitoring and post-construction monitoring. Monitoring may include water quality during construction, beach profiling during and after, biological monitoring pre- and post-construction and potentially, monitoring of surfing conditions.

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9.0 CONSTRUCTION CONSIDERATIONS AND COSTS

This section provides a general discussion of some of the construction assumptions and considerations, and costs for the alternatives. The cost and construction section is somewhat general to allow a range of alternatives or combinations to be considered. It is possible that several alternatives can be combined and implemented at various locations along SCNP. If several small projects are implemented simultaneously along the Park, then some of the costs between the alternatives can be shared, such as the Mobilization and Demobilization and Environmental Documentation costs.

9.1 Construction Issues

Some of the major construction considerations associated with the alternatives are listed below. These topics have direct impacts on construction costs and constructability of the conceptual projects.

A. It appears that shoreline protection construction can be performed from land. If it is later determined that construction can not be completed from the land-side, then project costs will be much greater due to the access restrictions of the narrow beach along much of the Park's coastline.

B. Transport and stockpile of building materials will require a local storage site. If local storage is not available, costs will increase substantially. There are some areas above the bluff that can be used for staging and deployment of equipment (existing parking lots), but other areas that do not have a staging area nearby will require an off-site area that would increase costs.

C. Excavation or grading in the surf zone in submerged conditions will require close coordination with ocean conditions and traffic control and will impact project costs and schedule.

D. Any beach nourishment activity will require location of either dredge pipelines or truck haul routes. Larger volumes of sand will relatively increase the costs of truck delivery, but decrease the costs using a dredge pipeline.

E. Impacts to utility lines by construction equipment and trenching must be accounted for by either temporary protection or relocation.

F. Traffic control for construction equipment such as bulldozers, backhoes, cranes, and large trucks will be required.

G. Existing armor stone, if removed, can be stored nearby off-site, and replaced as either a sacrificial toe, an under or outer armor layer, or used as dissipation for proposed storm drain outfall structures.

H. Construction of any outfall dissipating structure should be completed at the same time as construction of the pipe outfall. It is best to construct the outfall to ensure that the flows from the outfall pipe are diverted or blocked during construction of the dissipating structure to avoid damage to work in progress.

9.2 Bluff Protection Construction Costs

Calculations of probable construction costs for each alternative on a per 100-linear foot basis were generated to assess concept level estimates for economic comparison of the items, and to guide project-funding requirements. A detailed breakdown of quantities and costs for each alternative is included in Appendix B. Table 9-1 summarizes the estimated initial construction costs that are detailed in Appendix B for each shore protection alternative.

These costs are intended to provide a first-order review of project costs. Material and construction costs will increase proportionally to the size of the project, but other costs may not. For example, the mobilization and demobilization costs may be slightly more for a 200-ft long project, but may not be doubled from the 100-ft long project. This is also similar for the engineering, design and permitting, and the construction engineering and management line items that are outlined in the detailed cost estimates in Appendix B. Also, if several alternatives are selected for different locations along SCNP as one inclusive project, then some of these costs can be combined. It is difficult to provide a detailed cost estimate for the entire SCNP until a specific project, or projects, are defined.

Table 9-1 Summary of Beach Fill, Reef, Perched Beach, and Tie-Back Seawall Alternatives. (Cost per 100-foot segment)

Alternative	Construction Total ($ per 100 lf)	Maintenance Costs (1) ($ per 100 lf)	Total ($ per 100 lf)
No-Project	$100,000	--	$100,000
Beach Nourishment	$1,400,000	$10,500,000	$11,900,000
Offshore Reef	$2,100,000	$1,900,000	$4,000,000
Perched Beach	$2,200,000	$1,900,000	$4,100,000
Tie-Back Seawall w/ Texture	$6,300,000	$200,000	$6,500,000

(1) Maintenance costs are present-value costs over the entire 50-year project life.

9.2.1 *No Project Alternative*

This may include minor grading at the bluff top, removing small portions of the concrete/rubble debris, seawall patching, and minor revetment redistribution. The costs include equipment to break the concrete into smaller pieces and to haul the debris away to a concrete recycle yard and the grading.

9.2.2 *Remove Existing Structures*

This alternative is not included in the cost estimate table above because it is difficult to define a cost per linear foot. The cost for removing structures is related to the volume or tonnage of material being removed and not per linear foot. It is estimated that the cost to remove rubble, armor, and debris could be on the order of $60 per ton and would include the operation of removing the material from the base of the bluffs and loading it onto trucks for disposal at an upland landfill.

9.2.3 *Beach Fill Alternatives*

The alternatives with a beach fill include the Beach Fill, Nearshore Reef, and Perched Beach Alternatives. The mobilization/demobilization costs include the setup of large machinery to spread the sand. For each of these alternatives, the cost of the beach fill includes importing the sand by truck from a land source. Dredging sand from an offshore borrow site is also an option, however the costs would increase substantially because of the much higher mobilization/demobilization costs (the price per cubic yard would be lower). If an offshore dredging project was already underway in the San Diego Region, or multiple locations of along SCNP were to implement a beach fill alternative, then it may be possible to offset the mob/demob costs by “piggy-backing” onto this existing project. The costs outlined above only include importing sand by truck from a land source as this is a more practical method for smaller volumes of sand.

The beach fill alternative is estimated to include a renourishment cycle of every two years, which substantially increases the maintenance costs over the 50-year project life. Also, sand sources need to be defined to supply the initial and renourishment sand on this proposed cycle. The nearshore reef and perched beach alternatives include a 10-year renourishment cycle.

9.2.4 *Quarry Stone Alternatives*

The Quarry Stone Alternatives include the Nearshore Reef and Perched Beach Alternatives. For each of these alternatives, the quarry stone costs assume the stone would be transported from local quarries, delivered, and placed at the site. The unit price costs were estimated from other similar projects and costs to place the armor in these difficult conditions.

9.2.5 *Tie-Back Wall Alternative*

The costs for the Tie-Back Wall Alternative include delivery and installation of the walls. The costs include the texture coating to blend the wall with the native bluff face. The costs were estimated from recent similar projects and discussions with contractors.

9.3 Schedule

The schedule includes the final decision to proceed with an alternative, final design, obtaining permits, contracting a construction firm, and construction of the project. The critical path will be preparation of the environmental documents with necessary permits and these can vary significantly depending on the types and numbers of coastal structures that are proposed. Scheduling for environmental documentation and permitting can be as short as one year for a Negative Declaration or Mitigated Negative Declaration for minor project, such as repair or maintenance of existing structures, to well over three years for major beach projects that would require an Environmental Impact Report.

Table 9-2. Schedule

Task	Duration
Review and Select Alternative(s)	12 months
Perform additional Studies and Surveys Bathymetry, geotechnical, biological	6 months
Initial Design	4 months
Environmental Documentation	1 to 3 years
Permitting	1 to 3 years
Prepare Final Design	9-12 months
Prepare Specifications and Estimates	6 months
Prepare Contractor Bids and Select Contractor	3 months
Mobilization	1 month
Project Construction	6 months to 2 years

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Appendix A

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1660 Hotel Circle North, Ste. 200, San Diego, CA 92108
P: 619-220-6050 F: 619-220-6055

M E M O R A N D U M

PROJECT: Sunset Cliffs
M&N Project No. 5837

DATE: Wednesday, May 9, 2007

SITE VISIT ATTENDEES: **Anne-Lise Lindquist, M&N**
Steve Jepsen, Dudek
Dedi Ridenour, resident at 1071 Sunset Cliffs Blvd

Issues raised by Dedi:

- Dedi questions the experience level of the boring company and questions if the borings were in the right places.
- Does the role of M&N include 'preventative maintenance' for bluff erosion/stabilization?, e.g. sand bags on newly formed gullies to prevent further erosion from upland runoff.
- She was concerned about why the Hydrology Study ended at SCB. Steve tried to explain that that is where we want it to end so a system can be designed to handle the water flow up to that point. She was concerned about the water westward of the SCB curb and where it would go.

Tour started at Dedi's house at 1071 Sunset Cliffs Blvd (SCB) and followed north, then we back-tracked toward the south. The recent bluff fall was directly fronting Dedi's house.

Note: Need to document the types of erosion including percolating water through/between formations but cite finding of boring study. (wind, waves, indirect erosion from riprap/seawall, pedestrian traffic, rodents, upland runoff)

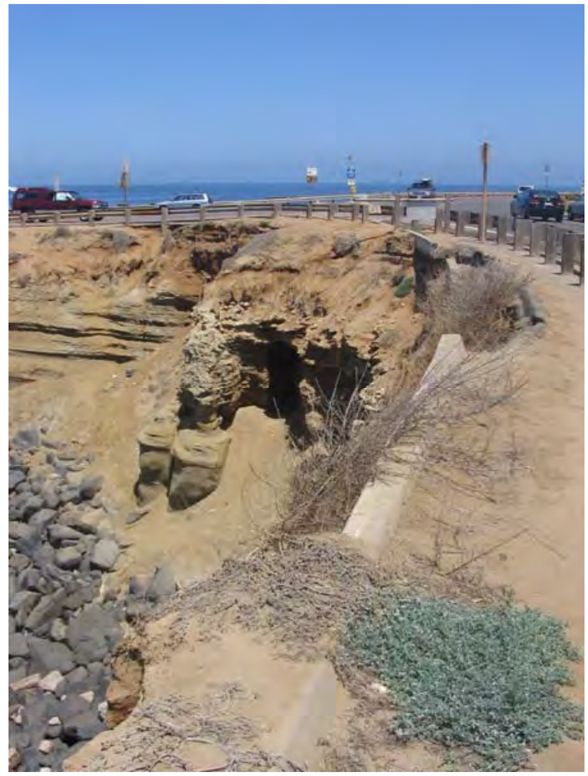
Dedi's recommendations/priorities:

1. Raise level of bluff top higher than the road to reroute rain water back to road and catchment basins.
2. Froude Street: (a) keep pedestrians west of road barrier; (b) restore bluff face to be higher than the road; and (c) re-route road along this area to allow for the new path. This might mean that a section of SCB is one way. (she doesn't agree with protecting the infrastructure <u>and</u> the need for a two-way road.)
3. Parking lots: burlap sandbags for preventative maintenance to keep runoff from running over the side of the road where curb has failed or isn't there. Not fiberglass bags.
4. Ladera Street stairs need to be re-designed.

PHOTOS:

- Froude Street:
  - There is ponding at the end of Froude Street (east side of SCB). Caused by rain and/or surface runoff.
  - There is a cave here and Riprap placed by USACE. Cliff retreat has been caused by uncontrolled drainage in parking area. Note: in 1915 concrete stairs and tunnel to the north fell down.
  - Dedi showed us sand piled on the east site of the street curbs (PH 1). She says this is caused by winds, carrying sand over the curbs. Is wind-blown sand erosion an issue? How can the wind-blown sand be controlled in drainage features (clogging)?
  - SCB is crowned, not sloped inland. Dedi said that she would like to see the road sloped inland. Steve mentioned the catch basins at the western curb.

PH1: Sandbags at the end of Fronde Street to prevent runoff over cliffs



PH2: New (<10 year) cave forming.

PH3: Parking at Froude Street slopes seaward; therefore all drainage runs over cliff. Concrete stairs used to exist to get to the cave and were replaced in the 1940s (these lasted 10-15 years). In 1915, near the corner of the parking, there was a little building there (this is where there was a lot of bluff erosion).

PH4: Bluff Failure caused by 2 caves joining on both sides of bluff protrusion and water impact through these formations.

PH5: Site of the April 2007 Bluff failure fronting Dedi's house.


PH6 &7: Bluff Top erosion. 5 years ago this was very small and now it is large and you cant walk over it anymore. 5 years to create this hole.

- There is a cave under the parking lot (pics 6 and 7). There was a whole looking down into/up through the cave and was filled with concrete on top to cover the whole in cave.
- Squirrels are a big problem and seem to be a large factor in increasing runoff-enduced cliff erosion.
- Note: Need to differentiate between Natural Cliff Retreat and Non-Natural.

PH8: showing concrete and eroded gullies from rain run-off from the parking lot.

PH9: Remnants from a 1915 wooden drainage structure.

PH10/11: Riprap to fill coves. In 1984, the Corps filled in all the crevices and eroded areas along entire SCB stretch.

PH 12: There is un-natural fill material placed on the top of the bluffs from the Corps during the same 1984?

PH13/14: riprap placed by the Corps in 84/86. Parking lot at Osprey.

PH15: (From parking lot at Osprey). Dedi used to walk across these two points as a kid, it used to be straight across. It has since eroded into a small cove area and rip-rap has been placed at foot of bluff in cove.

PH16: Concrete slab placed as a (failed) attempt to control run-off erosion (at Osprey). Flow was supposed to go over it, but as shown, it has scoured around it. Rodents live under these structures as they provide extra protection.

PH17/18: Fill on cliff face the end of Osprey.

PH19: at Osprey street: rubble at the end of the point. Needs to be removed.
Does this increase erosion from the added weight of riprap/rubble on top of the bluff?

PH20: Osprey drain pipe filled below with rip-rap. Drain pipe to be removed with new plan.

PH21/22: Another "failed" attempt to control runoff erosion of bluff, by pouring a concrete slab over the top of the point. These things need to be removed.

PH23: sea arch formed over last >20 years; caused by natural erosion (waves, wind).

PH24: USACE sea wall/riprap. There was a cove behind it that extended close to the road. The Corps built the wall farther seaward and filled behind it. The fill appears to be quarry rock and dirt. ***Dedi would prefer to see this are be used as a parking area, since it is 'man made' and it would be hard to restore this are to natural conditions. Natural vegetation would have a hard time growing over the fill material.***

PH25/26: (Spalding Point) This arch fell about 10 years ago. In 1915 there was a seawall and viewing platform. This collapsed because the seawall failed (Dedi suspects that this is indirect erosion caused by the seawall/shore protection scouring around it).

*****Dedi Recommended that this area could benefit from a newly created cove beach and use the rubble (to the north) to create a small groin on the north end, fill with sand, and provide access to beach (or use the point that we are standing on to get to the two cove beaches). She also said that this is one area that a textured seawall would be good solution and restore the cove beach.***

Adair Street (Fill on top of point.) Seawall

PH27: Dedi recommended 'stair-stepping' the bluff down to restore this area (opposed to hard vertical wall) (Looking back toward area of photo 26. we were out on the seaward point looking landward)

PH28: construction fill, concrete rubble. 1930 whole caved in and rubble fill was placed.

PH29: This shows 'unnatural' erosion caused by failing and settling rubble.
Dedi would like to see a set of terraces/stairs at this point to view the entrance of the park.

PH30/31: historical steps (1915), ended on sand to walk across to large rock. Now there is a big drop down and no sand along this reach. (Man-made 'swimming' hole in rock)

PH32: Construction rubble over bluff.

P33: No surf beach, no recommendations for beach access (no parking/restrooms/etc.) in Master Plan. However, it is a popular spot and foot traffic is causing erosion via paths and seeping water.

Dedi recommended rerouting road, move infrastructure in this area (one way traffic).

P34: Cordova/Guzot, source of erosion? Bluff top is higher than road, so it is not surface water runoff. Waves rarely reach the base of these cliffs, so it is not wave-induced erosion. Wind? Rodents?

Fill crevices with fill on top of bluff for trail? Probably would need compacted soils → can't grow plants on compacted fill very easily.
Monaco: rip rap beach - beach pocket just north of hole and fence.

PH35: South of Monaco: No stratification along cliff face; The road was built on cove and filled it with rock and fill. The gabions need to go before they break and spill small rock all over the beach.

No other issues south of here; except at Ladera.
1904 maps of coastline from Paul at the City.

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MOFFATT & NICHOL
1660 Hotel Circle North, Ste. 200, San Diego, CA 92108
P: 619-220-6050 F: 619-220-6055

M E M O R A N D U M

PROJECT: **Sunset Cliffs**
M&N Project No. 5837

DATE: Friday, May 10, 2007

SITE VISIT ATTENDEES: **Anne-Lise Lindquist, M&N**
Bill Dubbs, M&N
Steve Jepsen, Dudek
Anne Swanson, resident of Sunset Cliffs area

Ladera Street Stairs:

- Storm Drain: shortly after they worked on it in the 1990's, the bluffs collapsed. Coincidence or Cause?
- Cavern forming by the stairs.
- Rocks naturally placed. Not armor
- 1904 maps show a straight shoreline at Garbage Beach (now it is a pocket beach).
- What are the causes of erosion in Garbage Beach; the waves rarely reach bluff face. Runoff from parking lot? Seepage/piping to lower cliff face? Foot traffic? Wind? Incremental wave erosion from when it is exposed?

- Shear face on the bluffs look like they will soon fall.
- Water dripping on lower section of the cliff.
- Sinkhole on upper Park – The City filled it with riprap, and then it sank and disappeared. Was this sinkhole caused by piping? Did this increase the erosion?: riprap settles with the flow of runoff then it is too low to remove it. Does the weight of armor cause/increase failures?
- Are life estates contributing to the problem?

- Public access in mid Garbage Beach near concrete drain. Incorporate public stairs into plan for drainage structure
- This has been used as an Amphitheater area: was it created or used because of its natural configuration?

- University Ball Field on top of cliff is contributing to erosion.

- The pits on the cliffs at the south end of Garbage Beach, are these caused by piping? Or is it another contribution by the ball field/upland irrigation?

- End of the Grand Canyon: can't get through the lower formation and has completely eroded the upper formation.
- In 1988 there was a massive block fall in the south end of Garbage Beach in the curved area near the end.
- What is causing the caves at the end of Garbage Beach? Waves rarely reach here. Could be slow process only occurring during high tides/storm conditions.

- Point Loma Nazarene University – City wastewater effluent line – use this as the ADA access path

  - Surface erosion is coming close to the affluent line. (Sink area to the left of the top photo above)
  - Major increase in the surface erosion over the last 10 years.
  - Sink area: surface runoff. Used fiberglass sandbags by Young Hall Parking lot.
    a. Subterranean seepage? What causes this? Rodent hole causes a spot for water to seep in and through under and top layers.
    b. Soil is too hard for seepage through it without a way in (rodent hole)

c. Coyotes present.
d. Rodent control.

Fence of Navy Property

Shows extend of rodent burrows

Top of "V" that is shown in previous pictures from Garbage beach. This has eroded significantly over the last 5 years.

View of Garbage Beach

Grand Canyon (No man's Land). How to repair this erosion so park can be restored after new pipe is layed in the crevice?

This is the sink hole described on Page 4 of this memo.

Easement for the City's Wastewater Effluent line. Use this as the ADA path.

These eucalyptus trees were planted long ago. Need to be removed and represerved with natural vegetation.

MOFFATT & NICHOL

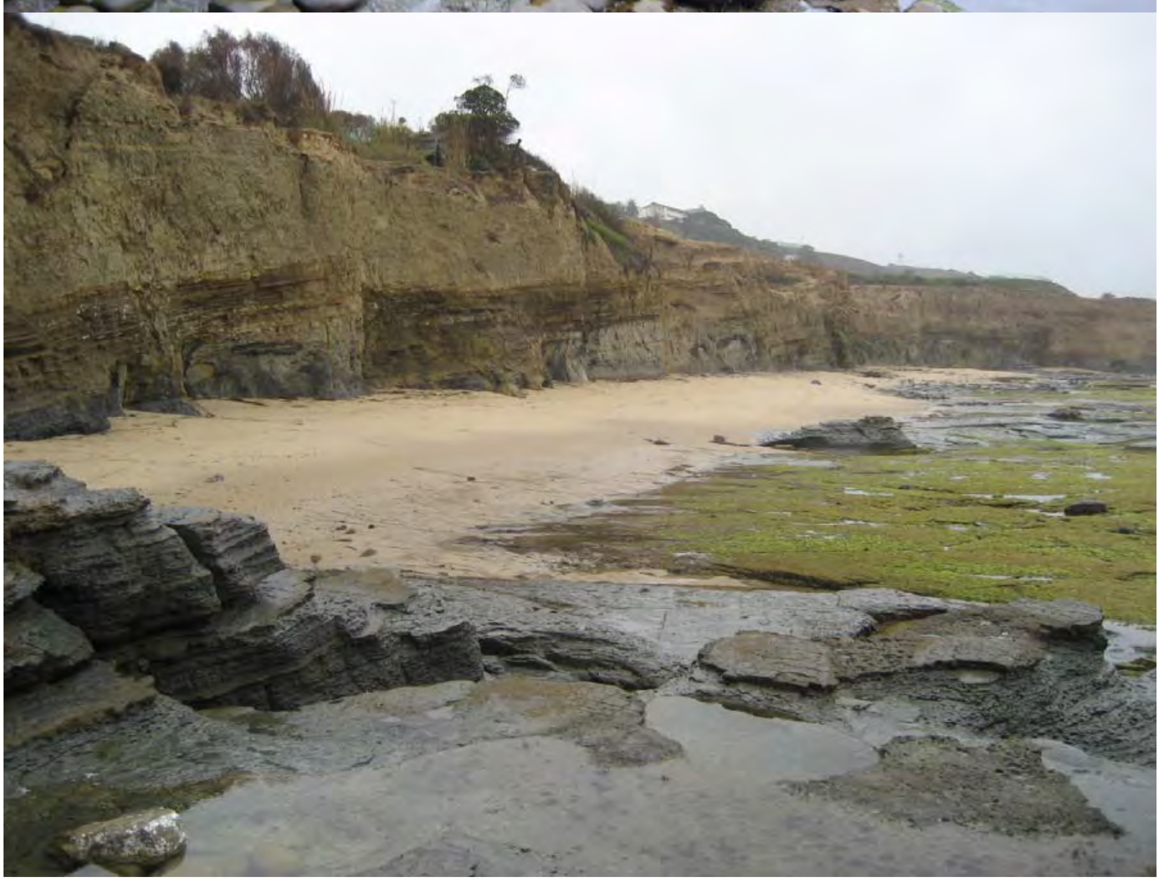
1660 Hotel Circle North, Ste. 200, San Diego, CA 92108
P: 619-220-6050 F: 619-220-6055

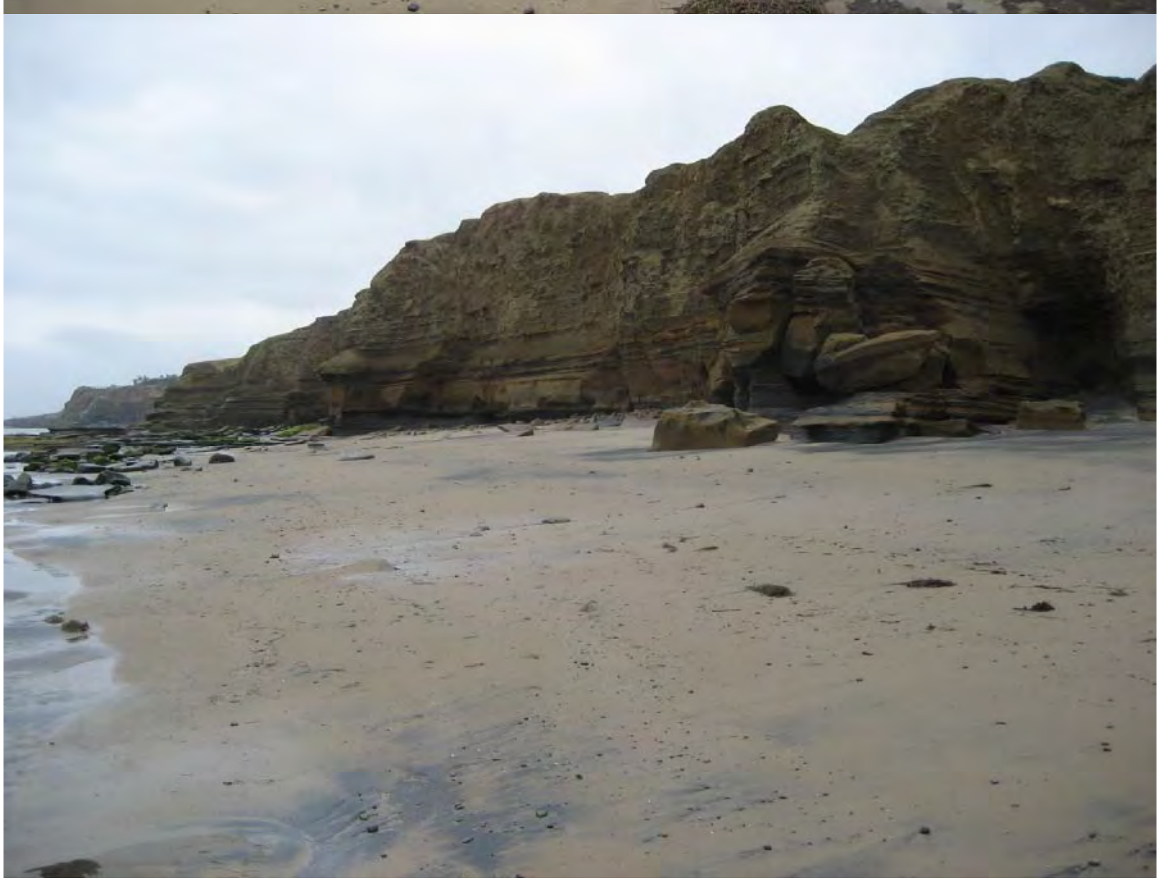
M E M O R A N D U M

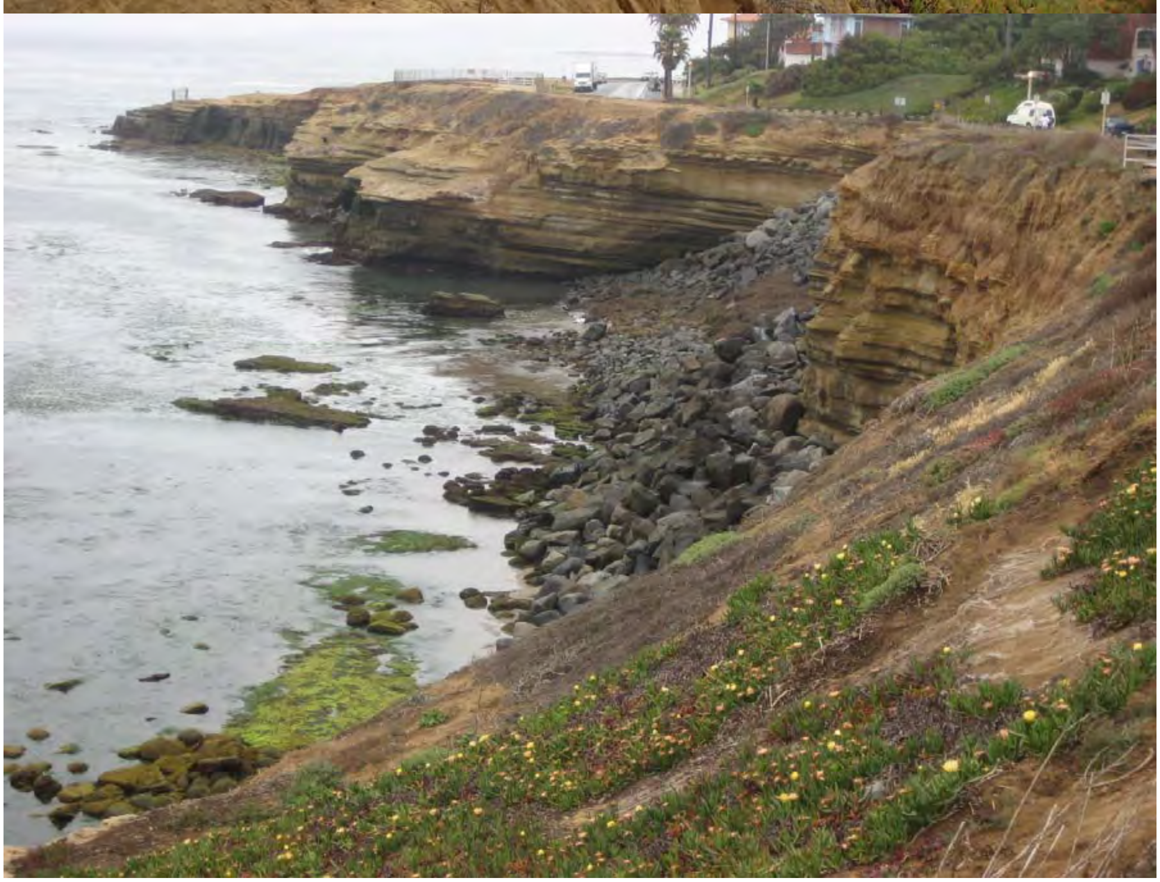
PROJECT: **Sunset Cliffs**
M&N Project No. 5837

DATE: Monday May 21, 2007

SITE VISIT ATTENDEES: **Anne-Lise Lindquist, M&N**
Alan Alcorn, M&N








SWIFT

Appendix B

BEACH NOURISHMENT (For 100-lf segment)

ITEM NO.	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	AMOUNT
	INITIAL FILL				
1	MOB & DEMOB	1	LS	$250,000	$250,000
2	BEACH FILL[1] ((80ft wide x 100ft long) * 1.3 cy/sf * 1.1 overfill)	11,500	CY	$50	$575,000
3	SUBTOTAL (For 100 Linear Feet)				$825,000
4	CONTINGENCY			20%	$165,000
5	SUBTOTAL				$990,000
6	ENGINEERING, DESIGN, PERMITTING	1	LS	20%	$198,000
7	CONSTRUCTION ENG. & MGMT.	1	LS	20%	$198,000
8	INITIAL BEACH FILL COST. (For 100 Linear Feet)				$1,386,000
	PERIODIC NOURISHMENT				
9	MOB & DEMOB	1	LS	$250,000	$250,000
10	BEACH FILL[1] (100% of original fill)	9,000	CY	$50	$450,000
11	SUBTOTAL				$700,000
12	CONTINGENCY			20%	$140,000
13	SUBTOTAL				$840,000
14	ENGINEERING, DESIGN, PERMITTING			20%	$168,000
15	CONSTRUCTION ENG. & MGMT.			20%	$168,000
16	ONE BEACH NOURISHMENT (For 100 Linear Feet)				$1,176,000
	Project Year			FUTURE INFLATED COST	PRESENT WORTH
	2			$1,258,320	$1,086,337
	4			$1,340,640	$999,216
	6			$1,422,960	$915,617
	8			$1,505,280	$836,203
	10			$1,587,600	$761,394
	12			$1,669,920	$691,413
	14			$1,752,240	$626,339
	16			$1,834,560	$566,137
	18			$1,916,880	$510,691
	20			$1,999,200	$459,825
	22			$2,081,520	$413,324
	24			$2,163,840	$370,945
	26			$2,246,160	$332,429
	28			$2,328,480	$297,512
	30			$2,410,800	$265,929
	32			$2,493,120	$237,423
	34			$2,575,440	$211,740
	36			$2,657,760	$188,643
	38			$2,740,080	$167,905
	40			$2,822,400	$149,311
	42			$2,904,720	$132,663
	44			$2,987,040	$117,777
	46			$3,069,360	$104,482
	48			$3,151,680	$92,621
19	TOTAL MAINTENANCE PRESENT COST (For 100 Linear Feet)				$10,535,878
20	INITIAL FILL + MAINTENANCE (For 100 Linear Feet)				$11,921,878
21	ROUNDED (For 100 Linear Feet)				$11,900,000

NOTES:
1. Sand unit costs based on single project costs delivered by truck and offloaded.
2. Annual Interest Rate: i = 7.625%
3. Annual Rate of inflation (From ENR): e = 3.50%
4. Project Life (years) = n: n = 50
5. Future Cost = Present Cost * (1+e*n). Amount paid for the same work n years in the future.
6. Present Worth = Future Cost/$(1+i)^n$. Amount placed in a bank account today.

OFFSHORE REEF (Per 100-lf segment)

ITEM NO.	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	AMOUNT
	REEF CONSTRUCTION				
1	MOB., DEMOB. & PREP. WORK	1	LS	$300,000	$300,000
2	ARMOR STONE	1,500	TON	$75	$112,500
3	UNDERLAYER	1,000	TON	$65	$65,000
4	SUBTOTAL				$477,500
5	CONTINGENCY			20%	$95,500
6	SUBTOTAL				$573,000
7	ENGINEERING, DESIGN, PERMITTING			20%	$114,600
8	CONSTRUCTION ENG. & MGMT.			20%	$114,600
9	REEF CONSTRUCTION COST				$802,200
	REEF MAINTENANCE				
12	MAINTENANCE @ YEAR 25 (PRESENT CONST. RATES)			20%	$160,440
13	FUTURE MAINTENANCE COST WITH INFLATION = Cost*(1+e*n)				$300,825
14	PRESENT WORTH OF MAINTENANCE = Future*$(1+i)^{-n}$				$47,916
	BEACH SAND PRE FILL				
15	SAND MOB & DEMOB	1	LS	$200,000	$200,000
16	BEACH SAND PRE FILL	11,500	CY	$50	$575,000
17	SUBTOTAL				$775,000
18	CONTINGENCY			20%	$155,000
19	SUBTOTAL				$930,000
20	ENGINEERING, DESIGN, PERMITTING			20%	$186,000
21	CONSTRUCTION ENG. & MGMT.			20%	$186,000
22	SAND PRE-FILL COST				$1,302,000
	NOURISHMENT				
23	SAND MOB & DEMOB	1	LS	$200,000	$200,000
24	REEF NOURISH (100% ORIGINAL QTY.)	11,500	CY	$50	$575,000
25	CONTINGENCY			20%	$155,000
26	SUBTOTAL				$930,000
27	ENGINEERING, DESIGN, PERMITTING			20%	$186,000
28	CONSTRUCTION ENG. & MGMT.			20%	$186,000
29	ONE BEACH NOURISHMENT				$1,302,000
	PROJECT YEAR			FUTURE INFLATED COST	PRESENT WORTH
	10			$1,757,700	$842,972
	20			$2,213,400	$509,092
	30			$2,669,100	$294,422
	40			$3,124,800	$165,309
30	TOTAL NOURISH PRESENT COST				$1,811,795
31	TOTAL PRESENT COST = REEF CONST. + MAINT+PRE-FILL+NOURISH				$3,963,911
32	ROUNDED				$4,000,000

NOTES:

1. Annual Interest Rate: i = 7.625%
3. Annual Rate of inflation (From ENR): e = 3.50%
3. Project Life (years) = n: n = 50
4. Future Cost = Present Cost * (1+e*n). Amount paid for the same work n years in the future.
5. Present Worth = Future Cost/$(1+i)^n$. Amount placed in a bank account today.

PERCHED BEACH (Per 100-lf segment)

	REVETMENT CONSTRUCTION				
1	MOB., DEMOB. & PREP. WORK	1	LS	$300,000	$300,000
2	ARMOR STONE	2,000	TON	$75	$150,000
3	UNDERLAYER	1,100	TON	$65	$71,500
4	SUBTOTAL				$521,500
5	CONTINGENCY			20%	$104,300
6	SUBTOTAL				$625,800
7	ENGINEERING, DESIGN, PERMITTING			20%	$125,160
8	CONSTRUCTION ENG. & MGMT.			20%	$125,160
9	REVETMENT CONSTRUCTION				$876,120
	REVETMENT MAINTENANCE				
12	MAINTENANCE @ YEAR 25 (PRESENT CONST. RATES)			20%	$175,224.00
13	FUTURE MAINTENANCE COST WITH INFLATION = Cost*(1+e*n)				$328,545
14	PRESENT WORTH OF MAINTENANCE = Future*$(1+i)^{-n}$				$52,332
	BEACH SAND PRE FILL				
15	SAND MOB & DEMOB	1	LS	$200,000	$200,000
16	BEACH SAND PRE FILL	11,500	CY	$50	$575,000
17	SUBTOTAL				$775,000
18	CONTINGENCY			20%	$155,000
19	SUBTOTAL				$930,000
20	ENGINEERING, DESIGN, PERMITTING			20%	$186,000
21	CONSTRUCTION ENG. & MGMT.			20%	$186,000
22	SAND PRE-FILL COST				$1,302,000
	NOURISHMENT				
23	SAND MOB & DEMOB	1	LS	$200,000	$200,000
24	REEF NOURISH (100% ORIGINAL QTY.)	11,500	CY	$50	$575,000
25	CONTINGENCY			20%	$155,000
26	SUBTOTAL				$930,000
27	ENGINEERING, DESIGN, PERMITTING			20%	$186,000
28	CONSTRUCTION ENG. & MGMT.			20%	$186,000
29	ONE BEACH NOURISHMENT				$1,302,000
	PROJECT YEAR			FUTURE INFLATED COST	PRESENT WORTH
	10			$1,757,700	$842,972
	20			$2,213,400	$509,092
	30			$2,669,100	$294,422
	40			$3,124,800	$165,309
30	TOTAL NOURISH PRESENT COST				$1,811,795

NOTES:

1. Annual Interest Rate: i = 7.625%
3. Annual Rate of inflation (From ENR): e = 3.50%
3. Project Life (years) = n: n = 50
4. Future Cost = Present Cost * (1+e*n). Amount paid for the same work n years in the future.
5. Present Worth = Future Cost/$(1+i)^n$. Amount placed in a bank account today.

TIE-BACK SEAWALL W/ TEXTURE (For 100-lf segment)

					40
ITEM NO.	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	AMOUNT
1	MOB., DEMOB. & PREP. WORK	1	LS	$250,000	$250,000
2	TIE-BACKS	45	EA	$70,000	$3,150,000
3	GROUT & MESH	3,000	SF	$90	$270,000
4	SUBTOTAL				$3,670,000
5	CONTINGENCY			20%	$734,000
6	SUBTOTAL				$4,404,000
7	ENGINEERING, DESIGN, PERMITTING			20%	$880,800
8	CONSTRUCTION ENG. & MGMT.			20%	$880,800
9	TIE-BACK CONSTRUCTION COST				$6,165,600
10	MAINTENANCE @ YEAR 25 (PRESENT CONST. RATES)			10%	$616,560
11	FUTURE MAINTENANCE COST WITH INFLATION = Cost*(1+e*n)				$1,156,050
12	PRESENT WORTH OF MAINTENANCE = Future*$(1+i)^{-n}$				$184,140
17	TOE PROTECTION ARMOR	400	TN	$75	$30,000
18	TOE PROTECTION QR	500	TN	$65	$32,500
19	SUBTOTAL				$62,500
20	CONTINGENCY			20%	$12,500
21	SUBTOTAL				$75,000
22	ENGINEERING, DESIGN, PERMITTING			20%	$15,000
23	CONSTRUCTION ENG. & MGMT.			20%	$15,000
24	TOE PROTECTION COST				$105,000
25	TOTAL PRESENT COST = TIE-BACK.+MAINT+TOE PROTECTION				$6,454,740
26	ROUNDED				$6,500,000

NOTES:
1. Armor & underlayer unit costs from local contractor
2. Annual Interest Rate: i = 7.625%
3. Annual Rate of inflation (From ENR): e = 3.50%
4. Project Life (years) = n: n = 50
5. Future Cost = Present Cost * (1+e*n). Amount paid for the same work n years in the future.
6. Present Worth = Future Cost/$(1+i)^n$. Amount placed in a bank account today.

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APPENDIX D
Sunset Cliffs Association Drainage Conditions

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SUNSET CLIFFS ASSOCIATION
DRAINAGE CONDITIONS AND RECOMMENDATIONS
SUNSET CLIFFS NATURAL PARK

INTRODUCTION

The Sunset Cliffs Association (SCA) is a nonprofit, unincorporated State of California Association (Secretary of State Registration Number 10039) that was established to provide a voice for people who want to see the Sunset Cliffs protected, degradation reversed, and resources for the area managed in a sustainable manner. The Mission Statement for SCA is: "Through education, help protect and ensure that the marine and terrestrial resources associated with Sunset Cliffs are used in a sustainable and unobtrusive manner. Education is meant in the broadest context and includes: newsletters, reports to governmental agencies (such as this one), serving on community boards and committees, maintaining a website, and making presentations to local schools and community organizations." Members of SCA have a long time (3 or more decades in some cases) association with Sunset Cliffs.

Based on the Scope of Work, between the City of San Diego and the DUDEK corporation for the Sunset Cliffs Natural Park (Park), a key aspect of Task 2 drainage studies is "to identify drainage sources and note areas of deterioration for possible erosion remediation." This information is needed because: "Task 3 will include establishing the criteria for design and identification of the opportunities and constraints for the drainage design alternatives."

SCA prepared this report to provide community input, on drainage sources and areas of deterioration due to surface waters in the Park that should be considered as sites of future remediation. Information summarized in this report is based on personal observations of SCA members, a 1999 Rick Engineering Report entitled Master Drainage Plan For the Sunset Cliffs Portion of the Ocean Beach Area In the City of San Diego Volume I of II [1] that will be referred to as the Rick Engineering Report, and a 2007 draft report by DUDEK entitled Sunset Cliffs Natural Park Hydrology and Hydraulic Analyses [2] that will be referred to as the DUDEK Report.

In order to analyze the need for drainage problem remediation in the Park, a necessary first step for each separate erosion problem is to identify the important aspects of the erosion problem including the locations of: (1) runoff sources, (2) areas of the Park that are degraded by runoff, and (3) the watercourses that connect the runoff sources with the degradation. SCA has focused discussion in this report on these three aspects of erosion due to surface waters, and has shown the location of each aspect on three annotated aerial photographs from Google Earth®.

Runoff moving rapidly in easily determined erosion gullies that make up the watercourses, causes most of the degradation due to erosion in the Hillside Park. During episodes of heavy rainfall, runoff can traverse the entire width of the Hillside Park in watercourses and flow over the cliffs. Significantly, the eight watercourses (B-I) noted on the Hillside Park map are down slope from the impervious surfaces of parking lots, roads, storm drain outfalls and compacted concentrated soil. Trails in the Park are *ad hoc* in the sense that they are not planned or engineered to prevent erosion as a trail would be if it were designed and constructed to meet commonly accepted guidelines for establishing trails [3]. An exception to the need for heavy rains to create major erosion degradation is watercourse G that exhibits erosion during most rainfall events because it receives the majority of drainage from the Point Loma Nazarene University (PLNU). Rainfall is concentrated by the culvert under the Western Loop Road and directed to the head of the major erosion gully commonly referred to as "Culvert Canyon".

Due to the easily eroded Baypoint Formation that makes up the majority of the Park soil surface layers, watercourses form easily along trails or other feature by runoff forming gullies along the fall line on the relatively steep slopes of the Hillside Park, or erosion rills throughout both the Linear and Hillside Park areas where the slope and runoff volume are lower. As can be seen on the Hillside Park map, not only do the watercourses tend to be down slope of impervious areas and structures that collect and concentrate runoff, they are associated with much of the degradation in the Hillside Park due to erosion. Once a watercourse is formed, water moves much more rapidly down slope than it would if spread over the area of an entire drainage basin, and it's velocity and concentrated volume becomes a significant erosive force.

The episodic nature of heavy rainfall in the coastal area exacerbates the tendency of runoff in watercourses to be an erosive force that is greater than would be expected if the runoff were spread over the floor of an entire drainage basin. It is common for intense rain to occur in San Diego County coastal areas for brief periods. For example, during 6 minute periods, rainfall can be the equivalent of an average rain of 7 inches per hour [4], and major erosion damage can be done in a relatively short period of time if erosion prevention measures are not taken to prevent it. This may have been the situation in October 2004 and the following winter months. There were a series of days of light to moderate rainfall that dropped 1.5 to 2 inches of rain over a 5 day period and probably saturated the soil. Then on October 27th, over 2.5 inches of rain fell in San Diego, and major erosion degradation was noted in the Park. Heavy rains continued after October and by the end of March 2005, 21.7 inches of rain had fallen in San Diego, making the 2005 Rain year that extends from October through September, one of the wettest on record. Therefore, to plan a comprehensive drainage system, it is necessary to identify the proper magnitude of runoff into watercourses during periods of maximal runoff flow, even if it is expected that this rainfall will only occur for a short period of time.

Watercourses were determined by several methods for this report. The most straight forward method was to identify a watercourse by it's eroded track in the soil, and then follow it while periodically obtaining GPS fixes. Another method we used was to note the position of a watercourse relative to a building or other easily identified landmark on an aerial photograph such as those provided by Google Earth® and are geo-registered so fixes can be obtained. These methods were employed to obtain the watercourse information for the Hillside Park map, and each GPS fix is indicated by a cross. Topology from a 1999 SanGIS 2' elevation contour topographic map of the Sunset Cliffs area prepared by DUDEK, that was geo-registered using the Fugawi 3® GPS mapping program, was also used to locate the watercourses shown on the Hillside Park map relative to gullies that were present when the 1999 map was prepared.

NORTHERN LINEAR PARK

Location 1 (see Northern Linear Park map)

Condition: Subaerial erosion of fill material is due to run off from the soil above it that has been compacted by pedestrian traffic. Lack of erosion control measures, such as revegetation to increase the permeability of the soil to water and not sloping the soil towards Sunset Cliffs Boulevard, is creating runoff degradation and contamination of adjacent nearshore waters. Marine erosion is also eroding the toe of this material in the figure below and contributes to nearshore water contamination.

Location 1 Continued:

Recommendation: Revegetation, sloping the soil towards Sunset Cliffs Boulevard so it doesn't erode seaward structures such as seawalls, and trail construction and maintenance using standard methods to prevent erosion are needed.

Location 2 (see Northern Linear Park map)

Condition: Marine, subterranean seepage, and subaerial erosion are all threatening both Sunset Cliffs Blvd., and the north wall of the cove that is just south of the Adair Street Seawall. If the north wall of the cove fails, the filled material behind the seawall will be exposed and expensive emergency repairs will be needed. Erosion problems at this site are discussed in an August 31, 2001 Moffett and Nichol report [5] that covers environmental, permitting, construction, and cost issues.

Recommendation: Review the seven alternatives described in the Moffatt & Nichol report as part of Task 3. The seven alternatives the report presented included: no project, beach nourishment, nearshore reef, perched reef, two tie-back wall alternatives, and a revetment. The Sunset Cliffs Natural Park Recreation Council (SCNPRC) on April 7, 2003 recommended that a tie-back retaining/seawall be constructed to prevent further surface water slumping and long-term marine erosion at Location 2 in a manner that conforms to the criteria established in the SCNPRC Erosion Committee April 7, 2003 recommendations.

Location 3 (see Northern Linear Park map)

Condition: Runoff from Parking Lot # 1 and areas of compacted soil and paving around it are responsible for erosion and piping that is destroying the cliffs, causing a safety hazard, and is contaminating the adjacent nearshore water with fine grained material that increases turbidity above natural levels.

Recommendation: Install curbs around the parking lot to insure that all water from it drains towards Sunset Cliffs Boulevard. Scarify the soil and plant native vegetation on it to increase it's permeability and minimize seaward runoff.

Location 4 (see Northern Linear Park map)

Condition: Runoff from Parking Lot # 2 and surrounding paved and compacted soil is undercutting the parking lot asphalt and causing erosion of the coastal bluffs.

Recommendation: Increase the slope of Parking Lot #2 and soil towards Sunset Cliffs Boulevard to maximize the runoff drains away from the ocean. Install curbs and remove impervious paved surfaces and compacted soil where possible, and replace them with vegetation or a trail surface that does not produce erosive runoff.

Location 5 (see Northern Linear Park map)

Condition: The condition of Location 5 (Rick Engineering Report Outlet 2) is described in the 1999 Rick Engineering report [1] as:

"*Outlet 2 consists of a 30-inch RCP that projects several feet off the face of the cliffs and discharges onto a riprap pad located at the base of the cliffs. Surface runoff from the tributary area is collected by a 17-foot curb inlet located near the upper end of the 30-inch RCP. The existing condition analyses indicate that the curb inlet does not have capacity to intercept the 100-year storm event. Therefore, runoff overtopping the curb may flow over the cliff face contributing to erosion. Runoff is conveyed from the inlet to a concrete inlet apron at the upper end of the 30-inch RCP. This inlet apron is cracked and may allow runoff to seep into the surrounding ground.*" The DUDEK report found for Location 5 that "the existing facilities are incapable of capturing the runoff from any of the storm events analyzed" and the excess runoff will "flow over the curb into the Linear Park."

Recommendations: The Rick Engineering Report recommendation for the condition at Location 5 quoted below, represents a good starting place for repair of the storm drain system at Locations 5.

"If the existing curb inlet is replaced with a single inlet of adequate capacity, the opening would need to be approximately 50-feet long. This option is not feasible due to the substantial opening requirement. Therefore, the recommended improvements are to remove and replace the existing curb inlet in Sunset Cliffs Boulevard with a 21-foot Type C-2 curb inlet (Standard Drawing D-3) and to construct a 23-foot Type C-1 curb inlet (Standard Drawing D-3) along both sides of Osprey Street at the intersection with Sunset Cliffs Boulevard. An 18-inch RCP would connect the curb inlets in Osprey Street. A 30-inch RCP would then convey the flow from the downstream curb inlet in Osprey Street to the curb inlet in Sunset Cliffs Boulevard, and finally to the outlet into the ocean. In addition, the existing concrete inlet apron should be removed."

Location 6 (see Northern Linear Park map)

Condition: Runoff from Parking Lot # 3 and surrounding paved and compacted soil walkways is undercutting the parking lot asphalt and causing erosion of the coastal bluffs.

Recommendation: Reslope and install curbs as needed for Parking Lot # 3 to facilitate runoff draining towards Sunset Cliffs Blvd., and minimize runoff from impervious surfaces around it through revetation and installation of trails that minimize runoff.

Location 7 (see Northern Linear Park map)

Condition: Runoff from Location 7, that includes Parking Lot # 4 and areas with compacted soil and pavement surrounding it, is a major source of cliff erosion and silt contamination of adjacent nearshore waters. Parking Lot # 4 has about a 4% slope to the west and an ocean storm drain outfall that is in total disrepair.

Recommendation: The Rick Engineering recommendation for Location 7 is:

""“Construct approximately 240 feet of AC berm along the westerly edge of the parking area to prevent runoff from discharging over the cliffs. This berm should be in accordance with San Diego Regional Standard Drawing G-5 (Type A asphalt concrete dike). A five-foot Type B curb inlet (Standard Drawing D-2) and an 18-inch RCP will have to be installed to convey the runoff out of the parking area and into the ocean."

It needs to be kept in mind that because Parking Lot # 4 has a steep slope to the west, adding fill material to raise the level of the parking lot so it would drain towards Sunset Cliffs Boulevard probably isn't economically feasible."

SOUTHERN LINEA PARK

Location 8 (see Southern Linear Park map)

Condition: Location 8, is Erosion Control Site No. 4 in the Rick Engineering report. A 100 curb that was added on the west side of Sunset Cliffs Boulevard as part of the "quick fixes" that were completed in 2002, appears to be keeping runoff Sunset Cliffs Boulevard water from the roadway from running over the cliffs at Location 8 and causing additional degradation. However, to the north several hundred yards, water is breaching the curb and running over the cliffs, possibly as a result of raising the curb at Location 8.

Recommendation: Continued monitoring of this area shown should be maintained to make sure that runoff doesn't run over the cliffs and further undercut the shoulder of the road and cause further cliff erosion. The erosion problems several hundred yards to the north needs to be investigated and action taken to prevent further cliff erosion.

Location 9 (Southern Linear Park map)

Condition: A 36" diameter storm drain discharges over the cliffs and into the ocean at the foot of Hill Street. Rick Engineering calculates this storm drain receives drainage from 88 acres, and that while it has a capacity of 63 cfs, flows of 113 and 121 cfs are expected from 50 and 100 year storm events that would overtop the curb and erode the cliffs. The findings in the DUDEK Report are similar to those reported by Rick Engineering in that they found the storm drain at location 9 drains Basin B that has an area of 103 acres, and during 50-year storm drain events "The excessive runoff will overtop the curb near the inlet and cause erosion within the linear park."

Recommendations: Rick Engineering recommends a major upgrade of the drainage system that discharges over the cliff at Location 9 that would include the replacement of four sections of pipe totaling 730 feet in length. This upgrade needs to be implemented in order to protect the cliffs from further erosion, and prevent Sunset Cliffs Boulevard from being undermined.

Location 11 (see Southern Linear Park map)

Condition: Rick Engineering lists Location 11 shown below as Outlet 5 which they describe: "This outlet consists of a concrete spillway that discharges over the cliff onto gabion slope protection. Approximately 2.2 acres are tributary to Outlet 5." Observations of the concrete spillway and gabion structures indicate there is bypass around the spillway due to settling in the area causing cracking of the concrete. Also, there is bypass around the gabion based on erosion of the cliffs adjacent to the gabion. The draft DUDEK Report calculates that the concrete spillway is capable of handling 5.3 cfs and that of the 22 cfs of flow generated within the basin tributary to it, only 4 cfs reaches it because most of the runoff that would otherwise reach it "- - - will flow over the berm and cliff" in Drainage Basin C2 unless the curbs along the west side of Sunset Cliffs for Location 12 are built up. Because building up the curbs along Sunset Cliffs as described below for Location 12 would direct flows to the spillway at Location 11 that it can't handle, fixes for the runoff conditions at Locations 11 and 12 need to be considered together.

Recommendation: Conduct detailed flow studies for Locations 11 and 12 to determine the size of a storm drain that would be needed to replace the concrete spillway and gabion protection device at Location 11. Alternatively, perhaps the road could be built up along the west side so water bypasses the spillway discharge and runs to the storm drain at the end of Monaco street. The discrepancy of the capacity of the Monaco Street storm drain being at or near capacity based on the Rick Engineering Report, and over capacity in the DUDEK Report, needs to resolved before any action is taken.

Location 10 (see Southern Linear Park map)

Condition: Heavy pedestrian traffic has compacted the Luscomb's Point (Location 10), soil, and the runoff from this compacted soil has caused extensive erosion rills to form. Silt from the eroded soil is increasing the turbidity in adjacent nearshore waters.

Recommendation: Trails designed to minimize runoff, and revegetion, are both needed to prevent the erosion and contamination of nearshore waters.

Location 12 (see Southern Linear Park map)

Condition: The Rick Engineering report, with concurrence in the DUDEK report, reported for Location 12:
"The westerly edge of Sunset Cliffs Boulevard between Carmelo Street and Outlet 5 contains an AC berm that is not to standard height. Therefore, runoff can flow over the edge of the road and contribute to cliff erosion." In addition, if this curb is brought up to standard height the concrete spillway and gabion protection device will not be able to handle the flows, so projects at Locations 11 and 12 needed to be coordinated as previously pointed out. Also, while Rick Engineering identifies one drainage basin (OB-08) between Carmelo Street and mid-way between Monaco and Hill Streets along Sunset Cliffs Blvd., DUDEK breaks the same area into two drainage basins, C1 and C2, that are divided at the Location 11. This discrepancy needs to be resolved in order to correctly solve drainage problems for Location 12.

Recommendation: Follow the recommendation of Rick Engineering to:
"Remove and replace approximately 500 feet of AC berm along the westerly edge of the Sunset Cliffs Boulevard to prevent runoff from discharging over the cliffs. This berm should be in accordance with San Diego Regional Standard Drawing G-5 (Type A asphalt concrete dike)." In addition to extending the berm 500 feet, assuming there isn't an insurmountable divide between DUDEK drainage Basins C1 and C2, consideration should be given to building up the pavement along the westerly edge of the road in the vicinity of Location 11 so that runoff will continue past it and discharge through the Monaco Street storm drain subject to considerations previously mentioned. Perhaps coupled with curb replacement to prevent cliff erosion, there should be the installation of properly designed storm drain at Location 11.

Location 13 (see Southern Linear Park map)

Condition: As described in the Rick Engineering report for their Erosion Control Area No. 7 (Location 13), "The westerly edge of Sunset Cliffs Boulevard between Carmelo Street and Ladera Street does not have an existing AC berm nor curb. Therefore, runoff can flow over the edge of the road and contribute to cliff erosion." Observations of this area during periods of heavy rainfall indicate that both runoff from Sunset Cliffs Blvd. and compacted soil are contributing to cliff erosion in this area.

Recommendation: Follow the recommendation of Rick Engineering to: "Construct approximately 600 feet of AC berm along the westerly edge of the Sunset Cliffs Boulevard to prevent runoff from discharging over the cliffs. This berm should be in accordance with San Diego Regional Standard Drawing G-5 (Type A asphalt concrete dike). Currently, vehicular parking is provided along the westerly edge of Sunset Cliffs Boulevard. In order to continue providing parking, the westerly edge of pavement should be extended approximately four feet and the AC berm should be constructed along the extended edge of pavement." The AC berm in this area should be placed under the guard rail protection to prevent people from walking on it and breaking it down.

HILLSIDE PARK

Hillside Park Watercourse A (see Hillside Park map)

Condition: Watercourse A (Location 29) is a 24 inch diameter pipe that receives runoff from the most northerly portion of the Hillside Park, and then coveys it to a driveway off Cornish Drive. The runoff from Location 29 is generated by the surfaces of the land in Drainage Basin D2 [2], and point source discharges from PLNU storm drain outfalls such as those shown in the pictures below, and PLNU parking lots.

Recommendation: Monitor runoff discharges into Basin D2 from the PLNU storm drain system for erosion, and where possible direct the runoff away from Watercourse A by either storing it on the PLNU campus for use in irrigation, or piping it to the nearby brow drainage system ditch that runs along the PLNU west property line in the vicinity of Goodwin Dormitories, and discharges down Monaco Street.

Hillside Park Watercourse B (see Hillside Park map)

Condition: Watercourse B is in DUDEK report Drainage Basin G, and runs from the 6 inch drain on the shoulder of Lomaland Drive near the "zebra crossing' adjacent to the PLNU Security Building, through the last residential property on the eastern side of Stafford Place, and through the Hillside Park to the sludge line trail as shown on the Hillside Park map. During low rainfall periods the runoff from Watercourse B travels down the sludge line trail and meet with runoff from to Watercourse C. During high flow periods it probably flows across the sludge line trail and into Watercourse I.

Recommendations: The runoff entering the 6 inch pipe near the PLNU Security Building could diverted to the same infrastructure in the DUDEK Report Drainage Basin M2 that is built to handle flows from Lomaland Drive and the Western Loop Road, to reduce the flow of runoff that traverses Watercourse B. Surge dams or tanks could be used to reduce flows in Watercourse B that can't be captured before they reach the Park so they can either be spread over a large enough vegetated area that they do not cause erosion, or they could be diverted to a street that drains into the storm drain at the intersection of Sunset Cliffs Blvd. and Ladera Street pursuant to the overflow concerns discussed for Location 14.

Hillside Park Watercourse C (see Hillside Park map)

Condition: Watercourse C originates on the unvegetated slope just west of the Lomaland Properties, and then runs downslope through the Lower Parking Lot and then the *Eucalyuptus* grove at the northern end of the Lower Parking Lot, before going over the cliffs at the north end of Garbage Beach at a site that is very prone to cliff failure.

Recommendation: The majority of runoff that now flows in this watercourse, probably can be diverted with techniques that spread the water over a large vegetated area to reduce flows to a velocity and volume that will cause minimal erosion degradation. How these techniques can be implemented will need to be studied. Runoff reaching the Lower Parking Lot should be directed to the storm drain at Location 17 and not allowed to run over the cliff as it does now.

Hillside Park Watercourse D (see Hillside Park map)

Condition: Watercourse D in Drainage Basin J originates at the spillway shown below in the northwest corner of the Upper Parking Lot. A clearly eroded watercourse can be traced from the vicinity of the parking lot spillway shown in the picture below, where dense vegetation makes determining exact watercourses very difficult, to the sludge line trail where the water spreads out before it travels down the slope without any major erosion until it reaches the badly eroded area where rocks have been displaced at Location 18. The clearly visible sheen on the runoff before it discharges from the Upper Parking Lot and into Watercourse D indicates the Park is receiving contamination from this runoff when it rains.

Recommendation: The runoff from the Upper Parking Lot should be captured by a surge dam or tank where it leaves the parking lot spillway, treated to remove contaminants, and then either discharged down slope at a rate that will not cause erosion or channeled to infrastructure that discharges to a storm drain. With the constraints noted for Location 14 to prevent overflows of the storm drain structure at the corner of Ladera Street and Sunset Cliffs Blvd., runoff could be disposed of there along with flows from other watercourses to storm drain infrastructure that terminates at Location 14. Alternatively, flows from Watercours D could be diverted to the treatment and ocean disposal outfall that will be needed to handle runoff from the PLNU drainage system that drains Drainage Basin M2.

Hillside Park Watercourse E (see Hillside Park map)

Condition: Watercourse E extends from the western edge of Western Loop Road downslope of the 18 inch PLNU drainage outfall at Location 28 on Park property that probably created it, to a deep erosion gully at Location 20. During extremely heavy rainfall, runoff from the 18 inch drainage outfall and runoff coming down the road collect on the Western Loop Road and then flow over the edge of the road and down Watercourse E. Vehicle traffic on the Western Loop Road exacerbate the runoff flows near the road by their tires splashing water over the edge of the road.

Recommendation: Repair and increase the height of the curb along the western edge of the Western Loop Road at the origin of Water Course E on a temporary basis. As part of the drainage system for the Hillside Park, runoff flows from the PLNU 18 inch drainage outfall that discharges directly onto Park property should be channeled into a pipe or concrete lined brow ditch that then conveys it to a drainage system that in turn disposes of it through an ocean outfall at sea level so it doesn't cause erosion degradation in the Park. A pipeline in the Western Loop Road roadbed should be considered for conveying the runoff from the 18 inch drainage outfall and other flows that are now directed down the road from Basin M2, to avoid destroying Park vegetation. To minimize Watercourse E collecting and concentrating rain water when the runoff from the loop road no longer traverses it, grading to flatten it and allow water to spread over a larger area and thereby reduce it's velocity, or natural looking check dams should be installed along with replanting it with native vegetation.

Hillside Park Watercourse F (see Hillside Park map)

Condition: Watercourse F during periods of extremely heavy rainfall receives runoff that goes over the edge of the Western Loop Road and flows into the large erosion area called "The Badlands" in the Master Plan. This runoff is from the 24 inch PLNU drainage outfall on Park property at Location 27 above it, and other Drainage Basin M2 runoff that is directed down the Western Loop Road. It is unclear at this juncture exactly where in the Badlands the runoff in Watercourse F flows.

Recommendation: The recommended course of action for stopping the deleterious effects of runoff traversing Watercourse F is the same as described for Watercourse E.

Hillside Park Watercourse G (see Hillside Park map)

Condition: Watercourse G is the largest erosion degradation feature in the Park, and besides causing major problems for traversing the Hillside Park in a north-south direction, the eroded material is a major source of turbidity pollution in the adjacent nearshore water as shown in the photograph below. Watercourse G receives most of the runoff from drainage Basin M2 that is discharged either through 18, 24 and 12 inch outfalls at Locations 28, 27 and 26 respectively, or directly down the Western Loop road. Watercourse G starts at Location 26, where there is a catch basin and an Arizona crossing of the Western Loop Road, that are designed to handle 76 cfs of runoff. The runoff then traverses the Hillside Park terrace before discharging over the cliff above the south end of Garbage Beach. Watercourse G was mostly formed by erosion and mass wasting after 1960 based on aerial photos of the Hillside Park, and in some place it is 30 feet or more deep.

Recommendation: Once the runoff that created Watercourse G and is still enlarging it has been diverted into a drainage system that conveys it to the ocean for disposal, the Master Plan recommendation for "filling to more natural finish grade (following contours of the natural canyon) and revegetate with native shrubs and groundcover plants." should be implemented. Perhaps fill material from the adjacent ball field when it is removed under provisions of the Park Master Plan could be used since it is native soil.

Hillside Park Watercourse H (see Hillside Park map)

Condition: Watercourse H is about 300 feet long, but because it terminates at the cliff edge, it has played a significant role in the cliff erosion at Location 15. Runoff is generated by rainfall falling on the gravel driveway and roof of the northern most Ladera Property.

Recommendation: Divert and spread out water running down the driveway so it is absorbed by the soil, thereby minimizing the volume and velocity of water going over the cliff. Cobble check dams, such as those used by Native American Indians in the San Diego area to build soil for planting crops, should be investigated for use in the landscaping to slow down and spread water from the driveway that can't otherwise be diverted or spread. When the northern most Ladera Property is removed as called for in the Master Plan, the gravel driveway should also be removed, and the land scarified to enhance water absorption, and contoured and revegetated to minimize erosion and cliff degradation.

Hillside Park Watercourse I (see Hillside Park map)

Condition: Runoff from the access road to the Park's Lower Parking Lot and the compacted soil parking area shown in the picture below have resulted in the formation of Watercourse I. I runs along the northern edge of the *Eucalyptus* before it runs over the edge of the canyon that is just south of Ladera Street, and then joins the anatomizing minor watercourses where the canyon terminates at the edge of the cliff.

Recommendation: Watercourse I runoff should be channeled to the storm drain at Location 17 along with the runoff from Watercourse C.

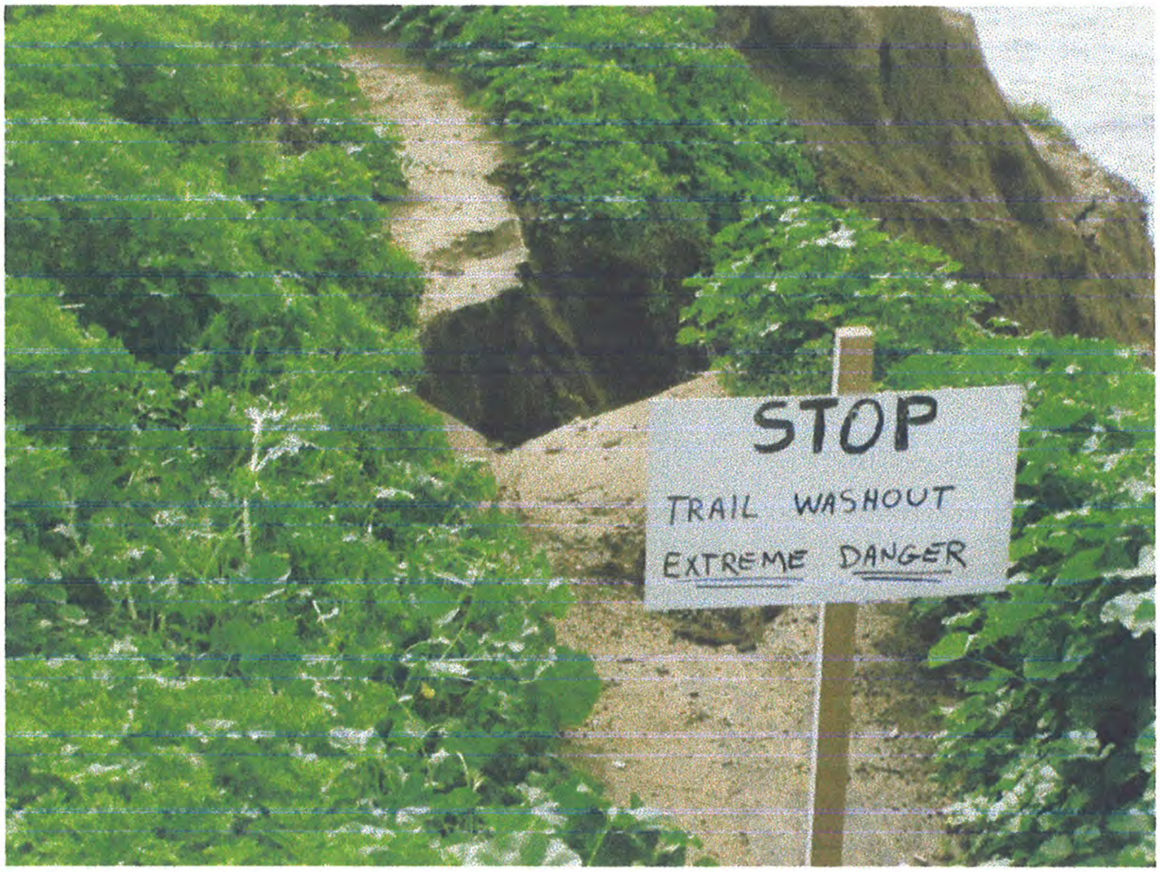
Location 14 (see Hillside Park map)

Condition: Location 14 is the 24" diameter storm drain that discharges over the cliffs adjacent to the corner of Sunset Cliffs Blvd. and Ladera Street. Both the Rick Engineering Report and the DUDEK Report calculations indicate that flows through this storm drain would be slightly under its design capacity during major rainfall events. Observations by SCA members during periods of maximal rainfall support the calculations of Rick Engineering and DUDEK. Since possible drainage control measures in the Hillside Park could conceivably discharge to the Ladera Street storm drain at Location 14, before any such measures are implemented, the consequences of adding additional flows to this storm drain should be considered and mitigation measures (such as delaying additional discharges with a surge tank or check dam until the peak flows from runoff now being discharged through during a rainfall event have passed) taken to keep storm drain overflow from causing cliff erosion.

Recommendation: If discharges from the Hillside Park drainage system are routed so they enter the storm drain at Location 14, mitigation measures, such as surge tanks, should be used to minimize the chances of the overflow and cliff erosion.

Location 15 (see Hillside Park map)

Condition: Watercourse H, that appears to be mostly generated by the driveway for the northernmost Ladera Property is causing a major erosion cut in the cliffs at Location 15. This cut has cut through an unofficial pathway along the top of the cliffs between Ladera Street and the north end of Garbage Beach that is used for viewing Garbage Beach and reefs to the south, and to reach the Hillside Park Lower Parking Lot.



Recommendation: Investigate methods for diverting water traversing the driveway to the canyon to the south in a way that does not increase erosion in this canyon, and minimizes the amount of water going over the cliffs and causing the erosion shown above.

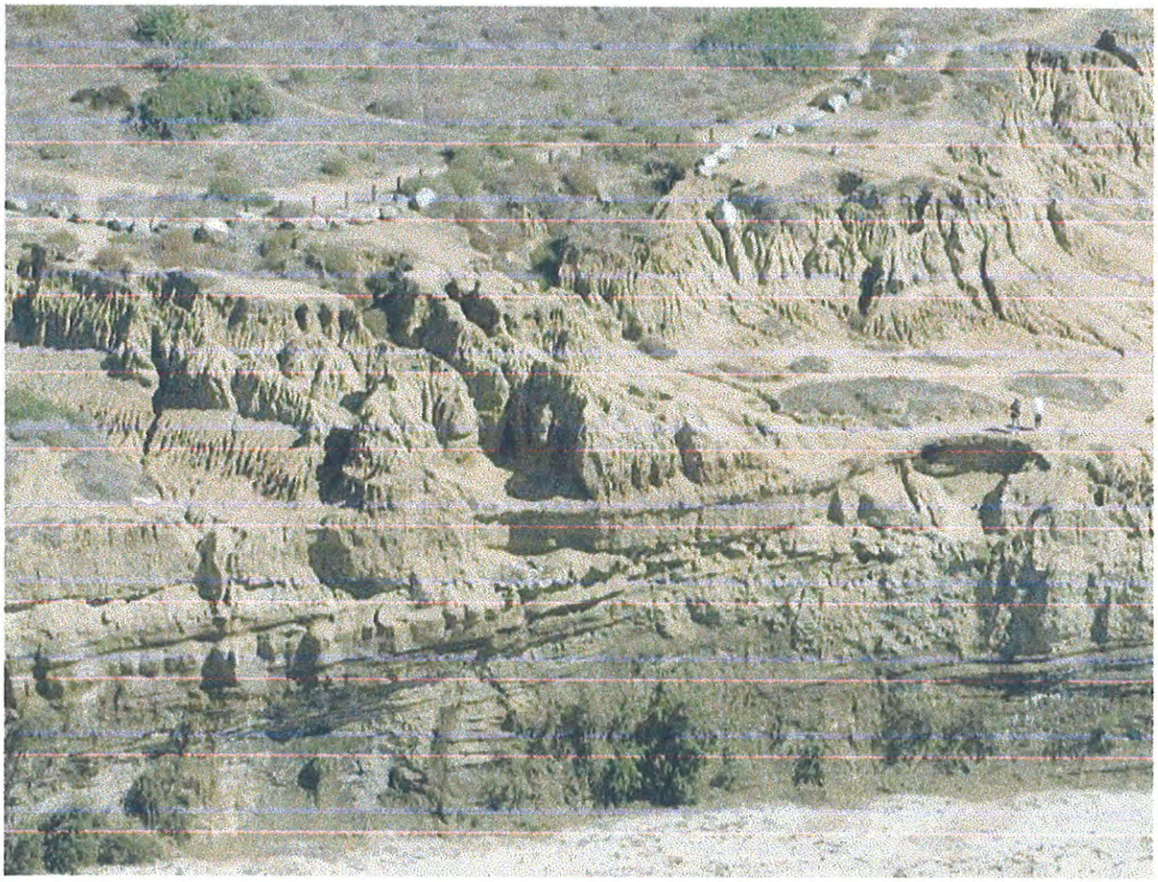
Locations 16 and 17 (see Hillside Park map)

Condition: Locations 16 and 17 are part of an un-permitted storm drain system, that appears to be designed to collect Lower Park Lot runoff with a grated inlet (location 17) in the southwest corner of the parking lot, and then channel this runoff through an underground pipe and concrete brow ditch to a discharge point at the approximate center of Garbage Beach (Location 16). Topographic maps and field observations indicate this storm drain system can only collect, at most, about 1/3 of the Lower Parking Lot runoff that is generated by the impervious surface of the parking lot and Watercourses B and C shown on the Hillside Park map. As noted in the draft DUDEK Report, "The surface erosion in the area clearly shows the drainage system in not functioning as it was designed to function."

Recommendation: Through the use of surge tanks and catch basins, reduce, spread and divert the peak flows in Watercourses B and C so they do not traverse the slope of the Hillside Park to enter the Lower Parking Lot. Through changing the north-south slope of the Lower Parking Lot, or installing a southward sloping drain system at the west end, divert all runoff on the Lower Parking Lot and the road leading to it from Ladera Street (see Watercourse I discussion) to the existing drainage system if calculations indicate it will handle the flows. If the existing Lower Parking Lot drainage system will not accommodate expected runoff from the parking lot and road, redesign it and build a drainage system that will accommodate the runoff flows.

Location 18 (see Hillside Park map)

Condition: The major erosion shown in the photograph is the result of Location 18 receiving runoff from Watercourses D and E. An appreciation for the dynamics of the erosion degradation near Location L can be gained by viewing how much the trail boulders in this area, that were put in place during the early 1990s, have been undercut and moved down slope.



Recommendation: Minimize the runoff from Watercourses D and E following SCA recommendations provided for them. Remaining runoff flowing to Location 18 within Drainage Basin J, and maybe the north part of Basin K, should be controlled with spreading and revegetation.

Location 20 (see Hillside Park map)

Condition: Location 19 is a large sinkhole at head of a 10 foot, or more, deep erosion gully that is part of major erosion feature that is called "The Badlands" in the Park Master Plan. Boulders have been placed into the sinkhole to keep it from eroding further to the east, but the erosion gully shown in the photograph below is still getting deeper and more extensive.

Recommendation: After repairing the curbs on the Western Loop Road and properly channeling the runoff from the road itself and the runoff from the 18 inch diameter storm drain at location 28 is properly directed so it doesn't flow down Water Course E, use water spreading and revegetation to minimize runoff that reaches location 20.

Location 21 (see Hillside Park map)

Condition: In the south wall of Watercourse G, that is generally referred to as "Culvert Canyon", after the culvert under the Western Loop Road where Watercourse G originates, trash in an abandoned landfill is visible at Location 21. The contents of this landfill need to be considered during any re-contouring work that is undertaken on Culvert Canyon or other erosion degradation in the area. The old landfill appears to have been constructed in a canyon that once extended from the amphitheatre on the PLNU campus.

Recommendation: Test the landfill for possible hazardous material that could pollute the ground water, and remove and properly dispose of its contents if they could potentially compromise the use of the Park or contaminate the nearshore waters.

Location 22 (see Hillside Park map)

Condition: The erosion and mass wasting associated with Location 22, is massive and it posses a safety hazard to Park users who can easily fall through the surface crust piping has created. Much of Location 22 is the remnant of a road that allegedly was bulldozed in to provide a way of reaching Garbage Beach for rescues. Besides the rills from surface water and piping that are apparent in the picture below of the north wall of Culvert Canyon, the terminus of an eastward running large crack, that may be a predecessor of a major slumping event that will destroy a major amount of the Hillside Park, is also visible. The degradation at Location 22 is probably the result of runoff from a variety of locations including: the Western Loop Road overflow, the compacted soil of the pedestrian trail that follows the sludge line and lacks vegetation cover, and the barren soil at Location 22 that was created when the road was constructed.

Recommendation: Repair the erosion and mass wasting damage at Location 22 after the runoff that is causing it has been identified and controlled. This repair will be a major undertaking that should be carried out in conjunction with remediation work done on Culvert Canyon caused by Watercourse G.

Location 23 (see Hillside Park map)

Condition: Location 23 is at the northwest corner of the lower parking lot for theYoung Hall dormitory, where there was drainage spillway that is now in total disrepair and undermining the parking lot pavement. Runoff from both the upper and lower Young Hall parking lots is channeled to Location 23. Through temporary measures by PLNU, Location 23 runoff it is kept from destroying the major north-south trail (shown in the picture below) that provides access to the Ab Reef access location (Location 25). These temporary measures also protect the 23 mile long sludge line that connects the Point Loma wastewater treatment plant with sludge processing facilities near the Miramar Marine Corp Air Station. Location 23 runoff is a major cause of erosion degradation below the Young Hall parking lots.

Recommendation: Direct the Location 23 runoff into the drainage system that is used to dispose of runoff from Basin M2 for the PLNU campus.

Location 24 (see Hillside Park map)

Condition: There seem to be several sources of runoff at Location 24 that are contributing to the erosion feature shown in the picture below that almost cut through the major north-south trail that is shown in the photograph below just below the toe of the embankment. It is not clear which source contributes the most. These sources include: the compacted soil of trails in the area, runoff coming down areas without vegetation on the Lower Parking Lot embankment, the trail that many use to traverse the embankment from the lower parking lot to the Ab Reef access trail, and parking lot runoff along the western curb that may jump the curb during periods of heavy rainfall.

Recommendation: Improve and maintain the trail from the parking lot so it doesn't generate runoff, and revegetate the area around Location 24 so it maximizes rainfall absorption.

Location 25 (see Hillside Park map)

Condition: Location 25 is the South Canyon watercourse that originates adjacent to private and US Navy property near Garden Lane to the east, and then runs to the west along the approximate property line between PLNU and the US Navy property before it crosses onto Park property. It terminates at the Ab Reef shoreline access point at the western most Location 25 arrow on the Hillside Park map. There has been considerable degradation of the area around Location 25, some of which is part of a registered archeological site, as a result of pedestrian traffic and runoff flowing down the South Canyon watercourse.

Recommendation: This South Canyon should be surveyed to determine if there are any drainage system outfalls discharging into it that could be modified so the velocity of the runoff is dissipated and spread in a way that allows the runoff to sink into the soil. The trail along the Navy fence needs trail erosion maintenance to minimize the runoff that it produces. In the western portion of Location 25 on Park land, trails leading to the Ab Reef shoreline access locations should be designed, constructed and maintained in a way that minimizes further degradation due to erosion.

Location 26 (see Hillside Park map)

Condition: Location 26 is a 10 inch diameter outfall pipe that drains the southern area of Drainage Basin M2 in the vicinity of the PLNU track and soccer field, and discharges into the catch basin that is just upstream of the Arizona crossing with the culvert that is at the head of Watercourse G that includes Culvert Canyon.

Recommendation: Flows from this outfall should be directed into the Hillside Park drainage system for ocean disposal of runoff along with runoff from: Drainage Basin M2, Young Hall Parking lots, and other drainage basins and locations in the Park.

Location 27 (see Hillside Park map)

Condition: A 24 inch diameter outfall pipe that drains Drainage Basin M2, discharges at Location 27 into a pool on SCNP property shown in the picture below that has been constructed to catch "dry weather" flows. This pool poses a hazard to wildlife, because any animals that drink from it will be exposed to any contaminants that might be washed into it from irrigation runoff.

During periods of heavy rain, the 24 inch diameter outfall pipe, that is located where the white piece of pipe can be seen in the picture below, discharges over the pond and into the non-native shrubbery below that should be removed per the Master Plan.

Recommendation: Flows from the outfall at Location 27 should be directed into the Park drainage system for ocean disposal along with runoff from drainage Basin M2, Young Hall Parking lots, and other drainage basins and locations in the Park. Dry weather flows from this outfall should be monitored for contamination and perhaps treated to remove any agricultural chemicals that are used to maintain the PLNU athletic fields and happen to enter the storm drain system.

Location 28 (see Hillside Park map)

Condition: Location 28, that is pointed out in the photograph below, is the location of an 18 inch diameter outfall from the PLNU drainage system that terminates and discharges onto Park property. "Dry weather" flow can be heard trickling down inside the broken cement structure at the end of the outfall pipe, but a pool or stream leading away from it has not been found due to the dense grove of extremely invasive non-native weed *Arundo donax* that is growing around the terminus of the outfall pipe. Since *Arundo donax* burns explosively, it should be removed so it doesn't pose a fire hazard to the nearby PLNU Physical Plant buildings. Location 27 shown in the photograph is the terminus of a 24 inch diameter outfall from the PLNU drainage system.

Recommendation: The effluent from the drainage outfall at Location 28 should be plumbed into the same ocean outfall storm drain system that is used for the disposal of runoff from Basin M2 and other runoff sources that impinge on the Hillside Park. The dry weather flows from this outfall should be monitored and perhaps treated before ocean disposal.

Location 29 (see Hillside Park map)

Condition: Location 29 is a pipeline that connects the northern part of the Hillside Park (DUDEK Report Drainage Basin D2, with an outfall that discharges down an alley way just east of Cornish Drive and into Drainage Basin D1. A cleanout in a depression near the western end of Park Property in the northern part of the Hillside Park that is the input structure to the pipeline that is Location 29, is shown in the photograph below. During the heavy rainfall period from October 2004 through February 2005, this depression was filled with a considerable amount of soil due to erosion in drainage basin D2. Muddy water was also observed in the streets below where the Location 29 pipeline discharged.

Recommendation: Monitor Drainage Basin D2 and the drainage system outfall for evidence of erosion, and correct conditions when evidence of erosion is found. New drainage system outfalls from the PLNU that discharge onto Park property and were installed in 2005 or at a later date (see location 30 and Water should

Location 30 (see Hillside Park map)

Condition: Location 30 is a 6 inch drainage outfall and spreading structure in the vicinity of the westernmost PLNU Nease Hall dormitory complex as shown in the picture below, and a nearby discharge into rip-rap shown in the second photograph. These outfall were constructed in 2006 and 2005 respectively, and they discharges into DUDEK Report Drainage Basin D2.

Recommendation: Monitor areas around PLNU outfall structure to see if runoff from them is responsible for erosion in DUDEK Report Drainage Basin D2. A possible action that could be taken to either prevent erosion from occurring or eliminate it if it is found, would be for PLNU to collect the runoff and use it for irrigation during the dry season.

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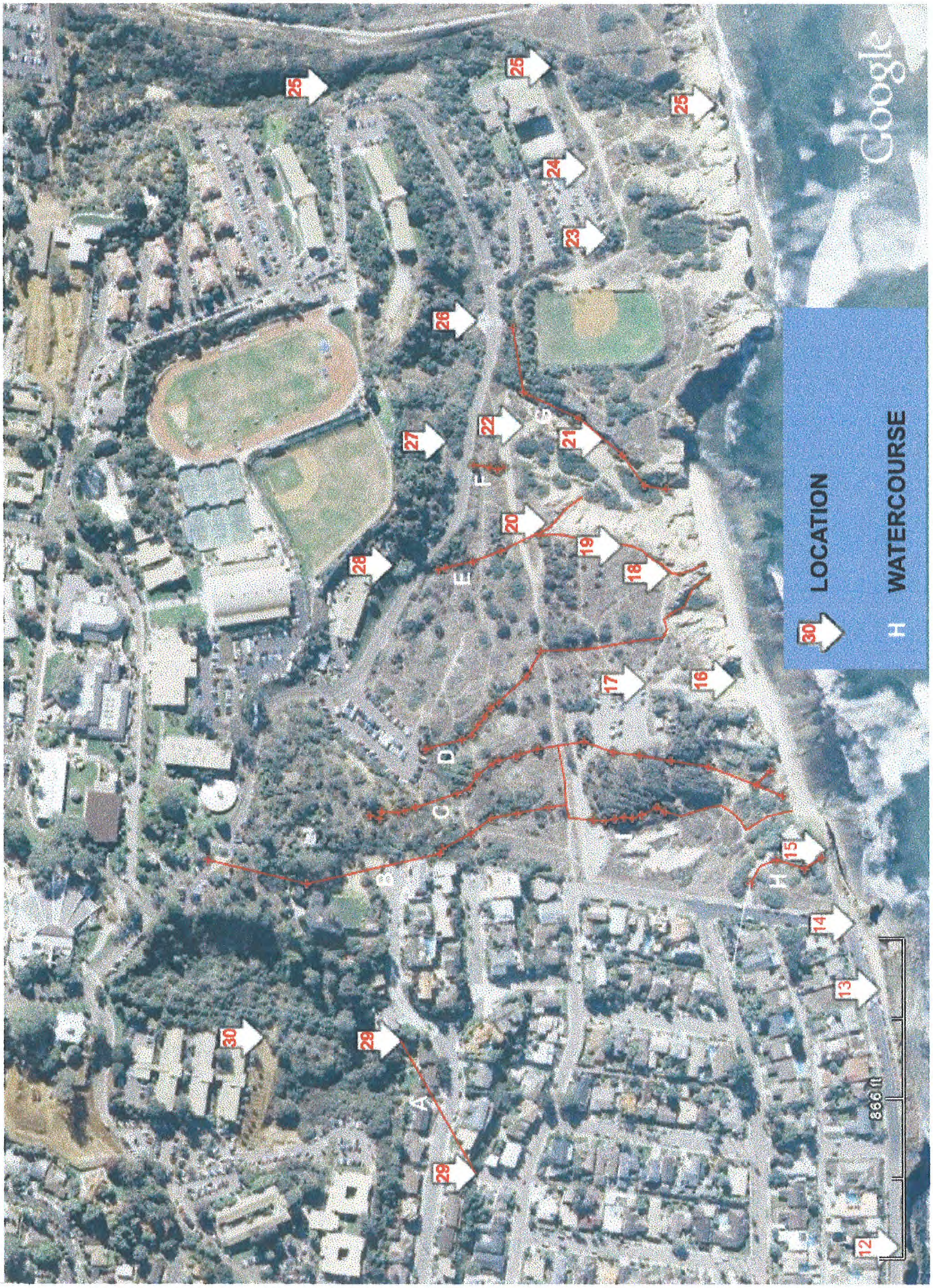
SOUTHERN LINEAR PARK

NORTHERN LINEAR PARK

HILLSIDE PARK



APPENDIX E

Recent Erosion and Mass Wasting Observed in Sunset Cliffs Natural Park

Recent Erosion and Mass Wasting Observed in Sunset Cliffs Natural Park

Craig Barilotti and Camilla Ingram
Sunset Cliffs Association

15 February 2011

Sunset Cliffs Natural Park Erosion and Mass Wasting, 15 February 2011

Executive Summary

A major source of nearshore pollution in San Diego is the erosion occurring in Sunset Cliffs Natural Park (Park) due to uncontrolled urban runoff causing major loss of parkland and creating nearshore pollution from the huge amounts of sediment and contaminants in the runoff and the dirt. The highest priority for the SCNP Master Plan is to control this excessive runoff and stop the extensive environmental damage to the Park and the large amount of nearshore pollution, likely a violation of the Federal Clean Water Act. So much sediment is flowing into the ocean, that many surfers believe the sediments are filling in and covering reefs and changing the way that waves break. The key to controlling the urban runoff is identifying the constructed features such as storm drains, roads, parking lots, and unplanned trails causing most of the erosion. This report discusses two major sources of urban runoff whose major environmental effects can be controlled: the Western Loop Road (Loop Road) and the Lower Parking Lot.

A comprehensive drainage system, now in the planning stages, will fully control, treat, and properly discharge the runoff to Ocean Plan standards, thus eliminating the environmental damage discussed in this report. However, due to the lengthy time it takes to design and build a major capital project, it is important to look for interim runoff control projects that can be quickly and inexpensively built, to control a significant amount of the damage and pollution.

The large December 2010, 5-day storm, a 5-10 year rainfall event, provided clear physical evidence of the source and flow courses of the runoff and the damage they cause, identified previously by the Sunset Cliffs Association in the Hillside Park[i] and with photographs and diagrams in this report. As a result, it is possible to propose methods to control the most damaging aspects of the runoff until the comprehensive drainage system is designed and built.

The Loop Road and the Lower Parking Lot, both concentrate runoff from drainage basins up slope of them. Two major storm drains (an 18-inch pipe and a 24-inch pipe) empty onto the Loop Road and contribute the majority of the runoff. During moderate to large rainfall events, much of the runoff "ponds" and flows over the western edge of the road causing extensive damage to parkland and cliffs above the southern areas of Garbage Beach. The Lower Parking Lot concentrates upslope and impervious surface runoff that causes erosion and cliff loss above northern areas of Garbage Beach. Runoff from both these sources is cutting across the Park's proposed trail system, including ones designated ADA, primary, and secondary trails.

Runoff that doesn't run off the western edge of the Loop Road flows into a culvert and then into a large, continuing erosion feature known as Culvert Canyon; the *de facto* storm drain for the Park and major source of sediment. Engineering studies for the comprehensive drainage system estimate a 10-year storm event can generate urban runoff flows of 50-cu ft/sec or over 1 million gallons for a 1-hour storm from the 39 acres drained. Due to the magnitude of Culvert Canyon flows, it is not possible to control them with inexpensive interim measures.

Almost all of the erosion from these sources can be easily controlled NOW by installing simple, inexpensive measures to direct the runoff to existing storm drains in the Park:

- Build or increase asphalt curb height on the Loop Road to a minimum of 6" to keep the water on the road. Keeping flows on the road will incrementally increase the damage to Culvert Canyon, but will protect parkland above Garbage Beach from further damage.
- Install asphalt curbs around the Lower Parking Lot and add a low berm to direct runoff to the storm drain in the southwest corner of the lot. These measures will eliminate most of the erosion and cliff failure occurring down slope of the parking lot.

Sunset Cliffs Natural Park Erosion and Mass Wasting, 15 February 2011

Report on Erosion and Mass Wasting Observed in Sunset Cliffs Natural Park

Rainfall during the current rainfall season that began in July 2010 is causing considerable erosion and environmental damage to the Sunset Cliffs Natural Park (Park). Recent major rainfall events have helped give a new and better understanding of urban runoff problems identified previously by the Sunset Cliffs Association (SCA) in the Hillside Park[1]. This report identifies the sources of the runoff, explains what is happening including clearly showing the runoff sources and courses on maps, and documents both with photographs. The runoff with its contaminants and massive amounts of sediments from this erosion have increased the turbidity and sediment deposition in the nearshore waters adjacent to the Park (Figure 1), both of which probably are in violation of the Federal Clean Water Act. Some say the deposition of these sediments has filled in subtidal reefs and changed the way surf breaks off the Park.

Specifically, the large December 2010, 5-day storm, a 5-10 year rainfall event, provided clear physical evidence of the source and flow courses of the runoff and the damage it causes. As a result, it is possible to propose methods to control the most damaging aspects of the runoff until the comprehensive drainage system is designed and built. The construction of curbs or berms at select locations can control much of the environmental damage discussed in this report. These simple measures should be implemented immediately.

Figure 1. Turbid nearshore waters adjacent to the Sunset Cliffs Natural Park December 22, 2010 visually alerts the public to the contaminants and massive erosion from the Park. Turbid waters, due to erosion from the Park, can be seen off the Park after every storm.

1. December 2010 Storm

What prompted this study and report, was new erosion damage in the Park that was observed after a major storm in December. A Wednesday, December 22nd San Diego Union Tribune article by Robert Krier and Gary Robbins, characterized this storm as being a "massive subtropical storm system that began bringing rain to California almost a week ago", and it delivered "its final and hardest blow to San Diego Wednesday, dropping more than 2 inches of rain before passing into Nevada, Arizona and Utah." (Figure 2 shows National Weather Service Doppler Radar records for December 21, 2010 that show the size of this storm). With regards to the frequency and duration of this type of storm, Krier and Robbins write: "The region hasn't received this much rain, over this long a period, since early 2005, say forecasters. 'This was pretty rare for Southern California, which usually gets storm fronts that move through in 3 to 6 hours,' said Brandt Maxwell, a forecaster at the National Weather Service in Rancho Bernardo. 'We get these kind of events only once every 5 to 10 years.' " Of particular note is that this type of storm does occur every 5 to 10 years.

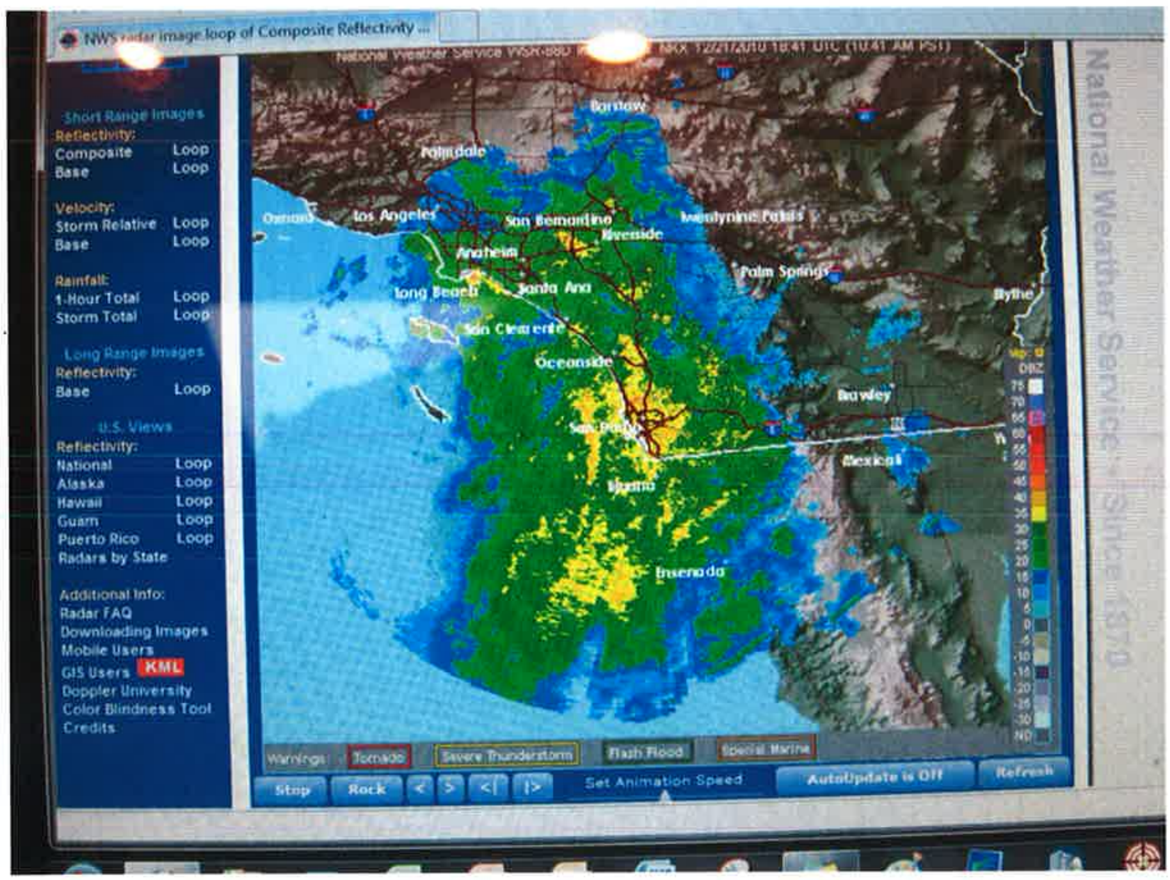


Figure 2. Doppler Radar record of the late December 2010 storm showing the storm's size.

Summary statistics by the National Weather Service for the period July 1, 2010 through January 3, 2011, indicate that above average rainfall occurred throughout southern California during December.

2. Erosion and Mass Wasting Sources and Examples

Recent erosion (damage to the land resulting from rapidly moving water that dislodges and moves soil particles) and mass wasting (where the weight of soil is increased by water saturation and the soil slumps or falls as blocks due to gravity) will be discussed and illustrated with photographs for the Hillside Park (Figure 3). The runoff from the major December storm caused new rill, gully and stream erosion features (see Figure 4 for a diagram illustrating these terms), thereby enabling us to identify sources of erosion that were previously uncertain.

Significant examples of erosion are documented in this study, much of it from runoff from the Western Loop Road (Loop Road) and drainage basins identified in the Hydrology and Hydraulic Analysis Report prepared by Dudek for the Park Comprehensive Drainage System (Figure 5). These runoff sources, the flow path, and resultant erosion will be identified below and discussed in photos taken after the December storm.

The environmental damage noted below, can be controlled by constructing curbs along the Loop Road. Many of these simple measures will only need to be taken until the comprehensive drainage system, called for in the Park Master Plan and presently in planning and design, is finished. By taking these measures now, much of the massive erosion within the Park and pollution of the nearshore waters will be avoided.

Figure 3. Areas in the Hillside Park where erosion and mass wasting problems can be corrected, by constructing curbs along the Western Loop Road. Features, both constructed and due to erosion, associated with discharge from the 18" drainage pipe are shown with red dots and those with the 24" drainage pipe with blue dots. The Lower Parking Lot's main features are shown in white.

Geographic features discussed and referred to throughout the report, include from north to south: Upper and Lower Parking Lots; the "Badlands" (as it is called in the Park Master Plan) the large badly eroded area near the center of the Hillside Park that is mostly denuded of vegetation; and "Culvert Canyon" where part of the northern wall recently collapsed. Culvert Canyon is the major erosion feature and contributor to nearshore sediment pollution in the Park. The Dudek Hydrology report estimates that it can be expected to receive up to 50 cubic feet per second of runoff during a 10-year storm, like we had in December.

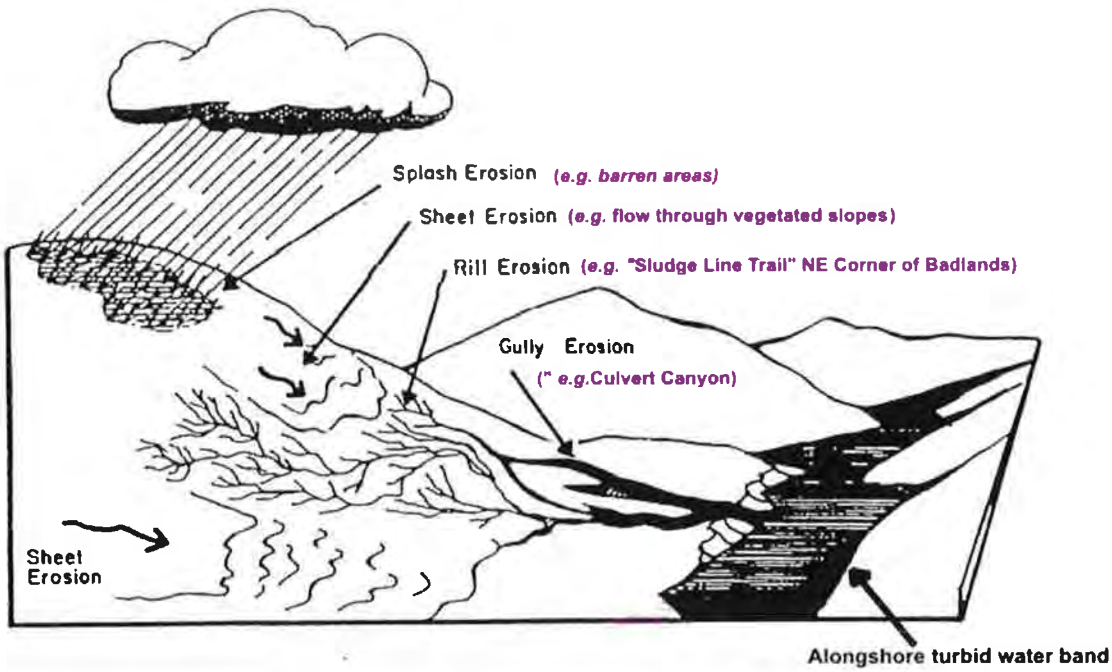


Figure 4. Standard erosion terminology used throughout this report and examples of where these features can be found. Mass wasting, where canyon or cliff failure occurs due to undermining by runoff or soil saturation is another process causing environmental damage in the Park in addition to the various scales of erosion shown in the diagram.

Distinct drainage basins defined by the topography can be identified in the Park. If the Park had native vegetation in these drainage basins and no constructed features with impermeable surfaces such as parking lots directing runoff toward the Park, most of the rainfall, even during large events like the December storm, would soak into the ground where it fell and be released slowly without causing significant erosion. Small steams and minor erosion features might develop, but they would not be of the size presently found in the Park. The large erosion features in the Park are the result of constructed features concentrating runoff from impermeable surfaces into pipes or a road that discharges onto easily erodible parkland. As a result, Sunset Cliffs Natural Park may be the largest source of sediment pollution due to erosion in the nearshore waters of San Diego.

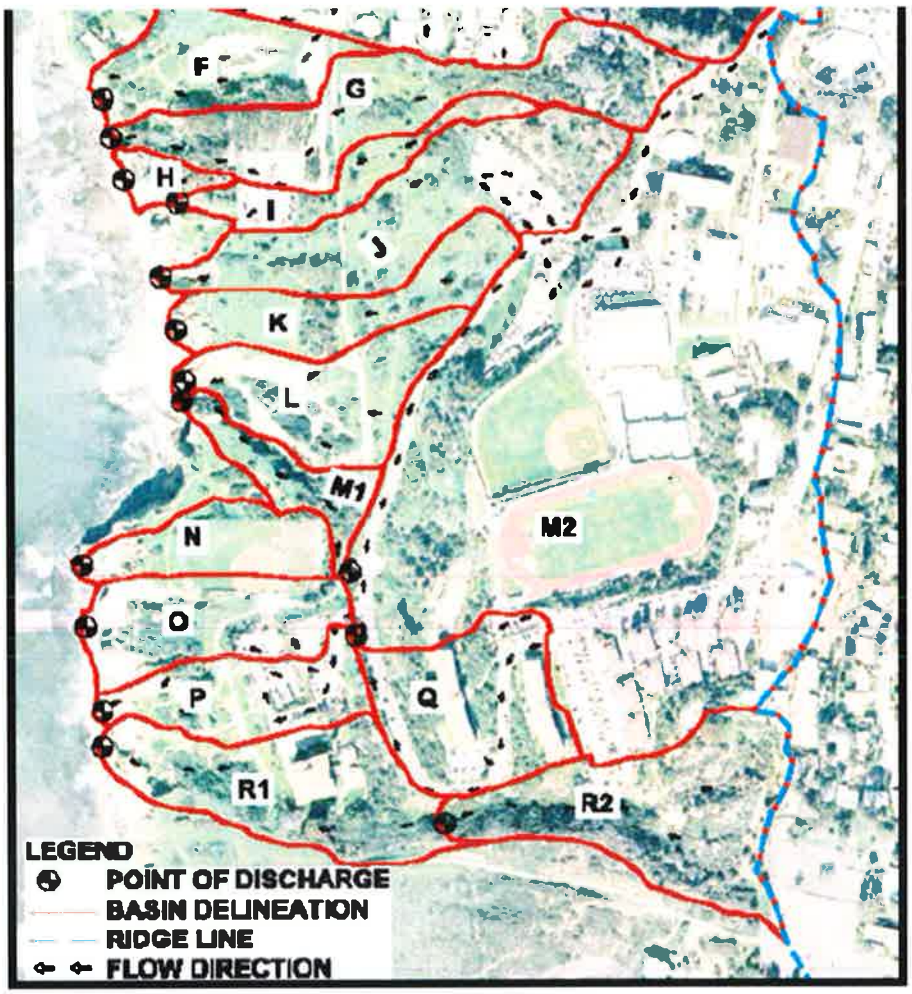


Figure 5. Hillside Park Drainage Basins identified in the Dudek Hydrology and hydraulics Report. The red lines on this map delineate distinct runoff basins based on the topography. The circles with an hourglass symbol inside note where the runoff discharges over the cliffs or into an adjacent drainage basin. The discharge from drainage pipes flows into some of these drainage basins, significantly increasing the runoff and the resultant erosion damage that occurs.

2A. Sources of Erosion Along the Western Loop Road

The Loop Road receives runoff discharges from the drainage system of Point Loma Nazarene University (PLNU), as well as runoff generated by the Loop Road itself, and Lomaland Drive that runs through the PLNU campus. In particular, erosion damage can be associated with the following sources of runoff, going from north to south:

- **18" pipe** – Is a drainage pipe from PLNU that discharges on the slope above the Loop Road, below the PLNU maintenance building. During moderate to heavy rains, runoff from the 18" pipe, along with runoff already on the loop road, "ponds" to the west of where the 18" line discharges to the Loop Road, before flowing over the road edge where curbing is too low or is missing. This runoff can be identified by vegetation being flattened and the presence of new erosion rills and gullies. This runoff is responsible for erosion and mass wasting in the form of slumping just below the Loop Road, cutting through existing trails, and causing erosion rill and gully formation, and mass wasting in the Badlands. (See Figures 3 and 6.)
- **24" pipe** – Is a PLNU drainage pipe that discharges onto the hillside about 500 feet south of the maintenance building and then onto the Loop Road. This discharge combines with runoff from other sources on the Loop Road and flows over the road edge where the curbing is low. After the major December storm, the water "courses" this runoff follows after it jumps the curbs were obvious as shown by vegetation being flattened and the presence of newly formed erosion rills and gullies. These features made it possible to link runoff to erosion rills along the sludge line trail and gullies that carried runoff to the failure of the northern wall of Culvert Canyon due to mass wasting. (See Figures 3, 18 and 20.)

The runoff from both the 18" and 24" drain pipes, and the Loop Road ends up flowing through Culvert Canyon carrying sediments and other contaminants to the ocean. Culvert Canyon is the narrow, deep (estimated to vary between 10 and 30 feet deep) erosion feature west of the Loop Road that has been visible in aerial photographs for over three decades. The Dudek Hydrology report estimates that it can be expected to receive up to 50 cubic feet per second of runoff during a 10-year storm. Culvert Canyon is the major erosion feature in the Park and the source for the vast majority of pollution and sediments (amounting to hundreds to thousands of pounds each rain season) in the nearshore waters. People who have surfed the wave breaks off the Park believe the vast amounts of sediments flowing into the nearshore waters have filled in reefs and changed the surf breaks. Eliminating environmental damage due to runoff in Culvert Canyon will require the measures planned in the Park comprehensive drainage system.

Erosion Caused by Discharges from the 18-Inch Pipe to the Western Loop Road
Runoff from the 18" pipe mostly stays on the road until the asphalt curb on the western side of the road decreases from approximately 6-inches to a height of 3". The few sand bags seen in Figure 8, demarcate the location where there is no longer a curb on the western edge of the road. As a result, the water flows off the Loop Road at various locations eroding the Park. A 6" asphalt curb on the western edge should be put in place along the entire length of the Loop Road. Potholes and cracks are also developing along the Loop Road. These should be resealed to minimize seepage under the roadbed. When the Comprehensive Drainage System Study has been completed and the comprehensive drainage system has been designed and built, erosion damage from runoff discharged through the 18" and the 24" pipes to the Loop Road should be eliminated, if the option for putting all discharges into pipes is implemented.

Figure 6. Runoff and erosion features associated with the 18" PLNU drainage pipe that discharges onto the Loop Road at the marked location. The large red dots show the location of features, and the small dots and streamlines show where evidence of watercourses transporting runoff that ponds on the Loop Road due to discharges from the 18" pipe can be seen after major rainfall events.

Figure 7. The 18" pipe discharges on this slope among the concrete rubble and dead Arundo plants. Runoff from the 18" pipe mostly stays on the road until the asphalt curb on the western side of the road decreases from approximately 6" in height to 3" and then is nonexistent.

In the 2004/2005 rain season, the length of the road where runoff flowed off the road was much longer. The asphalt curb was subsequently built up to 6" in some places, protecting parkland that used to get eroded by the runoff. If this were done down the length of the Loop Road, it would protect much of the parkland to the west of the Loop Road from erosion. This would not solve the problem of erosion within Culvert Canyon, but at least the damage would be for the most part limited to Culvert Canyon. Only implementation of the Comprehensive Drainage Plan can solve the erosion problems within the Park.

Figure 8. The curb down slope from the 18" storm water discharge pipe is only about 3" high in places, and ends at the sand bags. Runoff flows over the 3" curb and the area without a curb. It should be built up to 6". The runoff to this point does not include runoff from the PLNU 24" storm drain. The few sand bags seen, demarcate the location where there is no longer a curb on the western edge of the road, but the water flows off the road starting where the curb height is reduced to 3". A consistent 6" high curb placed along the western edge of the Loop Road would minimize water going off the road and onto parkland. Potholes and cracks are developing along the loop road, and these should be resealed to minimize seepage under the roadbed.

Figure 9. The curb height changes from 6" to 3" allowing storm water to run off between the two Acacia trees by the unpainted wood post, as well as, where the curb is missing, down slope from the sand bags.

Figure 10. Runoff from the 18" drain goes over the edge of the Loop Road where the curb is 3" or less in height, and it then flows under the bushes and follows the "trail" down the hill, shown by laid down weeds.

Figure 11A. This runoff course, due to the 18" pipe, is slightly further down the Loop Road than is the one shown in Figure 10. The runoff caused the "trail" and the slump and erosion to the right.

Figure 11B. Close-up of the erosion/mass wasting/mud hole due to runoff from the Loop Road. Those who use this trail to access surfing areas on an almost daily basis did not observe this damage to the Park prior to the December 2010 storm.

The runoff from the 18" pipe as it goes on down the hill is causing a significant collapse of a trail that is planned to become part of the Park formal trail system.

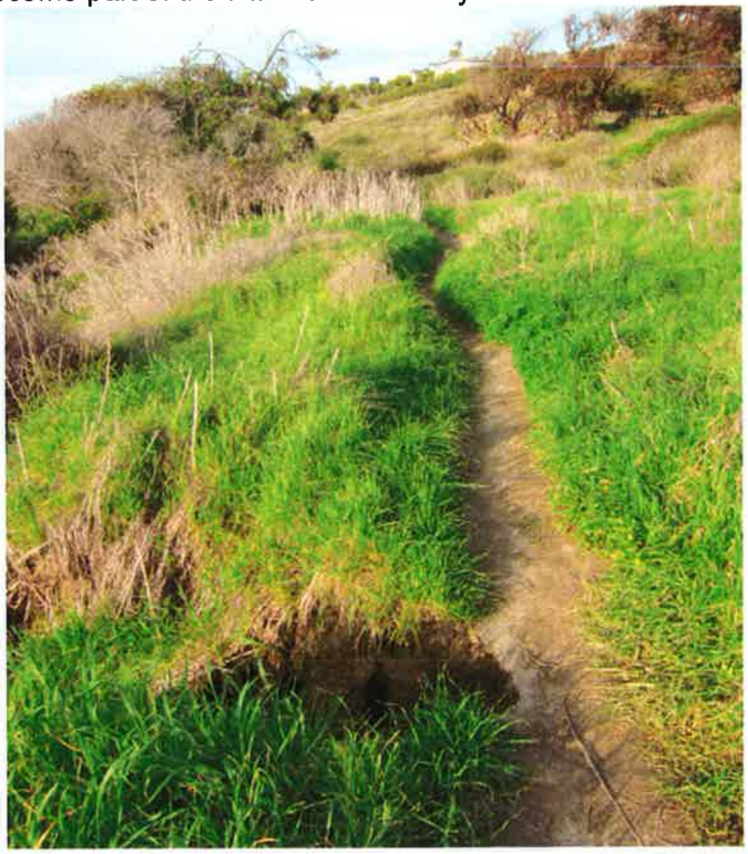


Figure 12A. Runoff from the 18" pipe is undercutting this trail that is being considered as a secondary trail for the Park, and has caused a significant collapse of the trail and the slope adjacent to it.

Figure 12B. Close up of erosion washing out the path. 6" curbs along the Loop Road could easily eliminate the runoff causing this washout and allow this trail to be part of the permanent trail system.

The runoff from the 18" pipe (Figures 13 & 14) continues down the slope, crosses the sludge line where it formed a 4-6 inch deep erosion rill, and then continues westward where it has started a new erosion gully on the northeast side of the badlands (Figure 15). This runoff is also exacerbating existing erosion gullies down slope along the northern edge of the Badlands and westward including extensive erosion between the Lower Parking Lot and the Badlands.

Figure 13. Runoff, mostly from the 18" drain pipe, crosses the sludge line trail, where it has cut this major erosion rill that is 4-6 inches in depth as well as the minor ones, before running into the Badlands along portions of the northern edge. From the Badlands, the runoff runs into Culvert Canyon where it is mixed with flows from Drainage Basins M1 and M2 that can total up to 50 cubic feet per second for a 10-year storm, like the December 2010 storm, before discharging over the cliffs to the ocean. A new erosion gully was formed where the person is standing in the photograph. 6" curbs along the Loop Road could eliminate the runoff causing erosion of the sludge line trail, based on preliminary designs for the Park comprehensive drainage system and observations of their effectiveness when used along parts of the Loop Road after the 2004-2005 rains. This should be done so the trail and sludge line will not require costly repairs.

Figure 14. This new erosion gully from runoff from the 18" drain coming off the Western Loop Road was first noticed after the major December 2010 storm. The upper, southward running portions of this recently formed gully should be filled in and stabilized to prevent it from enlarging.

Figure 15. Southern part of the new erosion gully formed by runoff from the 18" drain where it runs into the Badlands. This runoff flows into Culvert Canyon and into the nearshore waters.

Figure 16. Additional erosion caused by discharges from the 18" pipe along the northern edge of the Badlands. 6" curbs on the Loop Road could easily control this erosion.

Figure 17. One of many sinkholes next to the trail in the Badlands that pose a clear danger to people and pets, because they are over 10" in depth. These are caused by runoff from the 18" pipe.

Figure 18. Further erosion on the western portion of the Badlands due to runoff from the 18" pipe. The runoff joins the erosion gully in the bottom which joins the runoff from Culvert Canyon before it exits over the cliffs. This damage to the park would not occur if curbs were added all along the Loop Road.

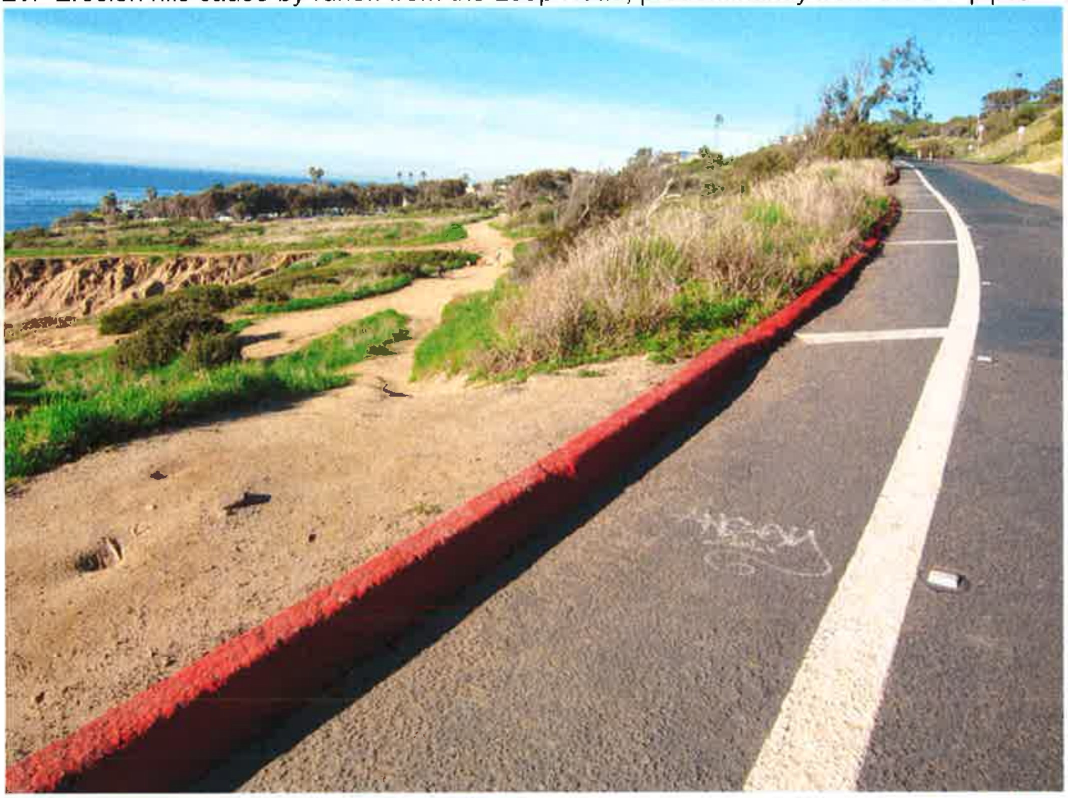
Figure 19. Runoff from both the 18" pipe and the Upper Parking Lot are likely responsible for this massive loss of cliff.

Erosion Caused by Discharges from the 24-Inch Pipe to the Western Loop Road

The 24" drainage pipe from PLNU discharges onto Park hillside above the Loop Road, about 500 feet south of the PLNU maintenance building. Even without rain, there is dry-weather flow from this pipe, evidenced by a persistent pond on the slope. This discharge combines with runoff from other sources on the Loop Road, including the 18" pipe, to create ponding on the road. After the December 2010 storm, the water courses this runoff follows were obvious (Figure 18), making it possible to link runoff to erosion rills along the sludge line trail to the resulting failure of the northern wall of Culvert Canyon from mass wasting. Culvert Canyon is a narrow and deep erosion feature west of the Loop Road (estimated 10 to 30 feet deep).

Figure 20. Erosion Caused by Discharges from the 24" Pipe to the Western Loop Road. For reference, the Badlands are on the left, Culvert Canyon is the deep feature to the left of center, and the old ball field is to the right. Runoff from the 24" pipe flows over the Loop Road curb and into the southeast corner of the Badlands and Culvert Canyon at locations indicated by blue dots. Runoff across the Park roughly follows the blue streamlines showing the linkage between erosion on the sludge line trail and the wall collapse area in Culvert Canyon. During periods of high flow the runoff also flows over vegetated areas.

Figure 21. Erosion rills cause by runoff from the Loop Road, predominantly from the 24" pipe.



22A

22B

Figure 22A & 22B. Low curbs transition to 6" high curbs where the ramp from the sludge line trail joins the Loop Road (Figure 22A). The low curbs enable road runoff, made up in large part by discharges from the 24" pipe, to flow down the transition trail (Figure 22B) and into Culvert Canyon where it contributes to mass wasting on the northeast wall of Culvert Canyon as shown in the next figure. This major section of the proposed Park trail system is being compromised by runoff from the 24" pipe.

Figure 23. Culvert Canyon wall collapse where runoff from 24" pipe and other sources crosses the Loop Road and continues down erosion gullies in Drainage Basin M1 until it reaches the north wall of Culvert Canyon causing the wall to fail by the erosion process called mass wasting.

Just north of the Arizona Crossing (the lighter colored concrete in Figure 22) runoff goes over the curb and is undermining the Arizona Crossing (Figures 23A and 23B) and is exacerbating erosion in the area of the concrete swale. This will cause the failure of the Arizona Crossing if it is not attended to. The runoff from the Loop Road to the Arizona Crossing goes into a concrete swale, which then empties onto riprap, and then onto unprotected parkland. Culvert Canyon is an unnatural feature created by runoff from the Loop Road and is the largest drainage/erosion feature in the Park. Most of the contaminants and sediments seen in Figure 1 are carried to the nearshore waters through Culvert Canyon, making this a highly significant source of marine pollution.

Figure 24. Due to low curb height at the northern edge of the Loop Road Arizona Crossing, runoff during moderate to high flow periods tends to flow around the northern edge of the Arizona Crossing apron and undermine it. The black Acacia branches cover the point where the runoff goes off the road.

Figure 25A. Undermining of the Loop Road Arizona Crossing by runoff that flows over the western curb (primarily from the 24" pipe). An adequate height curb could eliminate this maintenance problem.

Figure 25B. Close up of the undermining of the Loop Road Arizona Crossing. Most of the water causing this damage is from the 24" drain.

2B. Erosion and Cliff Failure Associated With Runoff From the Lower Parking Lot

Due to improper grading of the Lower Parking lot, little, if any of the rain and runoff that falls onto or goes into the parking lot from the hillside above it, leaves it through the existing storm drain in the southwest corner of the lot. As a consequence of improper grading, major erosion problems can be found down slope of outlets at both the southwest and northwest corners of the lot.

Besides the runoff the impervious surface of the Lower Parking Lot generates, it also receives runoff from Drainage Basins G and I, and perhaps from the Upper Parking Lot during peak rainfall periods (see Figure 5). The sum total of all runoff sources needs to be taken into account when designing short and long term fixes for erosion and cliff failure problems associated with the Lower Parking Lot. Runoff reaching the Lower Parking Lot should be directed to the storm drain at the southwest corner of the lot, and not allowed to run over the cliff as it now does.

Figure 26. Pathways runoff follows (shown in yellow) before runoff enters and after it leaves the Lower Parking Lot. Very little parking lot runoff passes through the existing Lower Parking Lot drainage system (in the southwest corner). This drain should carry the runoff to the discharge and the Gunite swale, seen as a light colored feature running across the bowl shaped area in the center of the picture.

Erosion and Cliff Failure Due to Runoff from the South 1/3 of the Lower Parking Lot

The un-vegetated slope below the Lomaland rental property, part of Drainage Basin I, appears to be a source of runoff, due to the lack of vegetation that would otherwise help absorb rainwater. It appears that pedestrian traffic has been responsible for denuding the loosely consolidated sandstone (Cabrillo Formation) that occurs in this area. Efforts to reduce runoff coming from this area will need to consider reducing pedestrian traffic and revegetation with native plants.

Figure 27. Area of Park where pedestrian traffic appears to have denuded the hillside (see A, B and C). Without a good cover of native plants, rainfall doesn't sink into the ground resulting in runoff generation that is causing erosion down slope, just before the runoff enters the Lower Parking Lot (see D).

Runoff from Drainage Basin I flows into the Lower Parking Lot along the south side. About mid-way down the south side of the lot, the runoff leaves the lot creating an erosion rill. The runoff does not enter the parking lot storm drain inlet in the southwest corner of the Lower Parking Lot, because the southern side of the parking lot lacks curbing that would keep runoff on the lot until it entered the storm drain inlet. The Master Plan calls for reconstruction of the Lower Parking Lot so it drains properly into whatever drainage system is finally built.

Figure 28. The south side of the Lower Parking Lot, showing both the storm drain inlet and an erosion gully to the south of the storm drain inlet. Clearly, runoff from Drainage Basin I flows along the south side of the lot, causing rill and gully erosion before it discharges over the cliff (Figure 24).

The storm drain in the southwest corner of the lower parking lot is part of an old storm drain system. It was supposed to collect Lower Park Lot runoff through the grated inlet and then channel this runoff through an underground pipe that discharges into a Gunite swale that carries the runoff approximately to the center of Garbage Beach. Lack of berms or curbs on the edge of the parking lot, has resulted in very little water going into this storm drain. As a result, runoff flows onto native soil, causing erosion gullies that bypass the drain inlet. Installation of these simple fixes on the south and west side of the Lower Parking Lot would allow the existing storm drain system to function as designed. This simple repair would prevent runoff from causing the massive erosion to the terrace and cliff below (see Figures 31, 32 & 33).

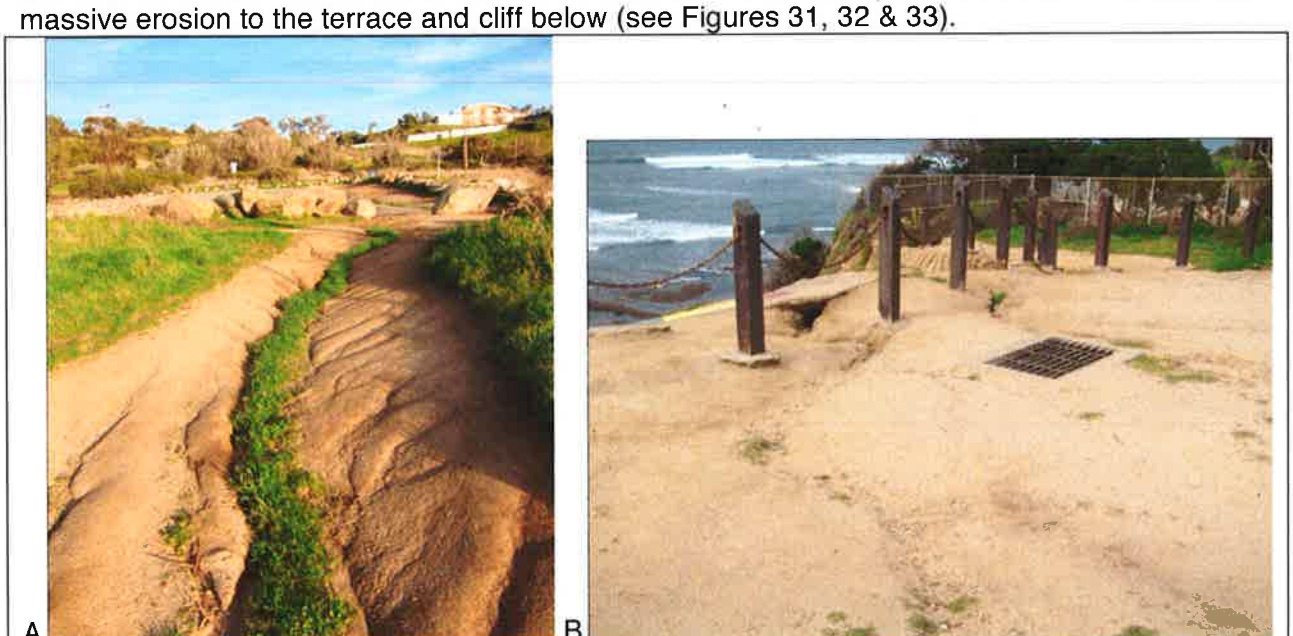


Figure 29A & B. Runoff gully from the southwest corner of the Lower Parking Lot leading an inlet on the bluff above the Gunite swale. The inlet and swale are higher than the erosion gullies cut around them, therefore the runoff continues to bypass the drainage system and is undermining the Gunite drainage swale. These maintenance problems should be addressed before the system is irreparably damaged.

Figure 30. Runoff from the southwest corner of the Lower Parking Lot, that has bypassed the storm drain, mostly follows this trail that runs southwest. This trail is planned as ADA. If the runoff is not abated, a raised walkway will need to be installed to provide an adequate surface for an accessible path.

Figure 31. Bluff erosion caused mainly by runoff from the southwest corner of the Lower Parking Lot.

Figure 32. Bluff erosion caused mainly by runoff from the southwest corner of the Lower Parking Lot.

Figure 33. Bluff erosion caused mainly by runoff from the southwest corner of the Lower Parking Lot. This erosion and that shown in the previous figures could be eliminated by putting curbs and berms in the southwest portion of the Lower Parking Lot.

Erosion and Cliff Failure Problems Associated with Runoff from the Northern 2/3 of the Lower Parking Lot

Runoff from Drainage Basins G and H drains onto the northern two thirds of the Lower Parking Lot, and along with runoff generated by the impervious surface of the Lower Parking Lot, exits at the northwest corner of the lot. None of this water runs to the drain in the southwest corner due to improper grading of the parking lot. Instead the runoff drains through the *Eucalyptus* grove that is northwest of the Lower Parking Lot and down an erosion gully before going over the cliffs at the north end of Garbage Beach to a site that is very prone to cliff failure.

Figure 34. Runoff exiting the northwest corner of the Lower Parking Lot.

Figure 35. Erosion gully that receives runoff from the northwest corner of the Lower Parking Lot after it has run through the Eucalyptus grove (Figure 32).

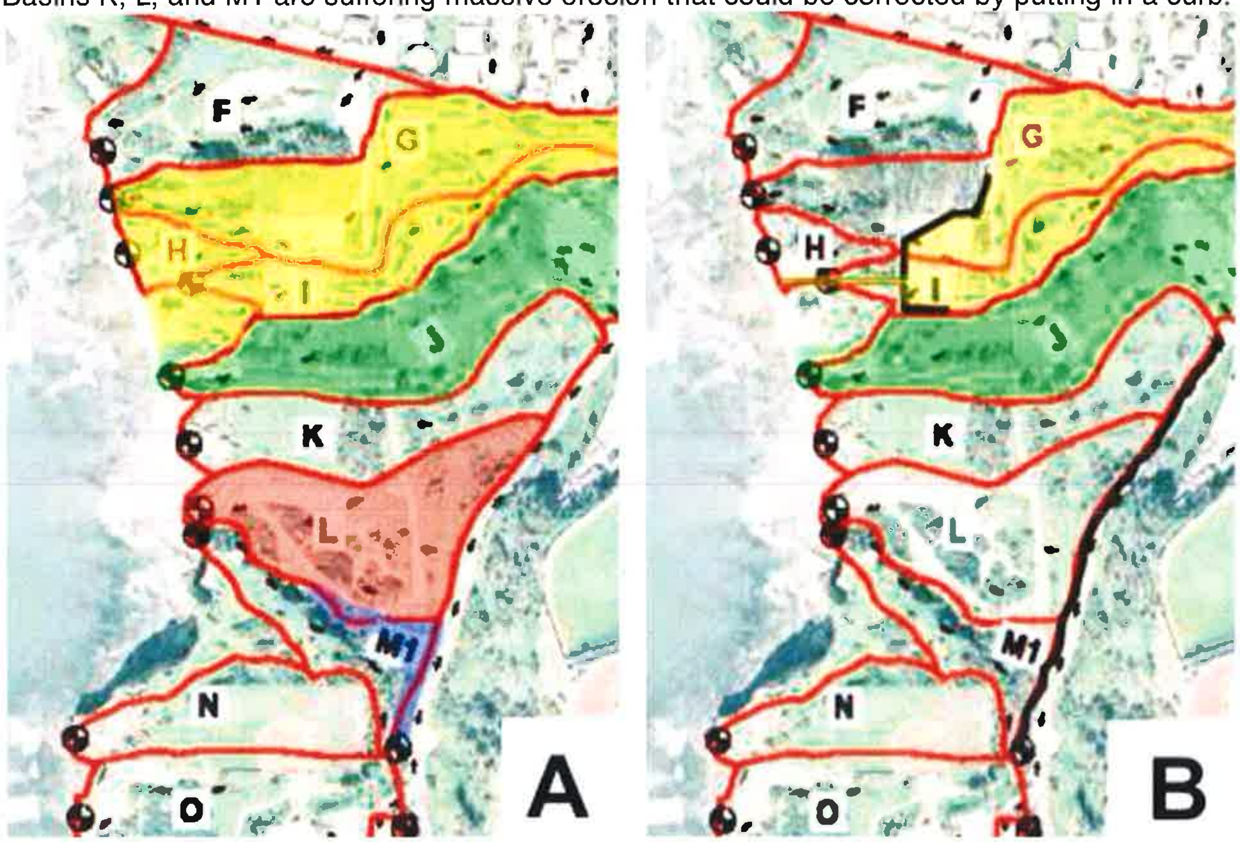
Figure 36. Runoff discharge from the NW corner of the Lower Parking Lot occurs all along this cliff above Garbage Beach. This dangerous cliff loss could be eliminated by diverting as much runoff as possible in the northern portion of the parking lot to the drain inlet in the southwest corner. Wattles and trail bars in the *Eucalyptus* grove should be used to build soil, and slow and spread the runoff so it isn't concentrated in one watercourse as it is now.

Figure 37. Northern end of Garbage Beach where runoff from the northwest corner of the Lower Parking Lot discharges over the cliff. Cliff block falls in this area pose a clear danger to beach users below as illustrated by these two photos taken 2 weeks apart.

Summary and Recommendations

This report has provided numerous examples, documented by photos, of the on-going erosion damage occurring in SCNP. Contaminants and hundreds to thousands of pounds of sediment washed into the nearshore waters are a major source of pollution in San Diego. Much of this ongoing, significant pollution and damage to the Park could be eliminated by placing simple, cost effective measures, such as asphalt curbs and berms in the locations shown in Figure 36B. The comprehensive drainage system, now in planning, will eliminate the runoff these measures will control, but due to budget constraints it will not be possible to implement this plan for years.

The Western Loop Road presently acts as an above ground storm drain channeling runoff from the road, and the 18" and 24" drains from Point Loma Nazarene University. These pipes carry runoff from approximately half the campus. By raising the curb along the west side of the Loop Road to a minimum of 6" in height, the runoff will stay on the road and go into Culvert Canyon. This will not eliminate the erosion occurring within Culvert Canyon and the bluff, but at least the damage to the park will be limited to this feature. Presently multiple areas within Drainage Basins K, L, and M1 are suffering massive erosion that could be corrected by putting in a curb.



Figures 38 A and B. Figure A shows the drainage basins and areas of the Park where erosion damage due to runoff from the Lower Parking Lot and the Loop Road is occurring. Figure B shows where simple measures, such as building asphalt curbs and berms, can be placed to control runoff that will eliminate or reduced substantially the erosion caused by this runoff.

Runoff from the Lower Parking Lot is creating massive erosion features on the cliffs in Drainage Basins G, H, I, and parts of J. There is a storm drain in the southwest corner of the Lower Parking Lot, but due to lack of curbs and a low berm, almost no runoff enters the storm drain, flowing around it and taking various paths to flow over the cliffs causing massive erosion and dangerous conditions for people on Garbage Beach (bluff collapse).

Figure 39. Shows preliminary Park Trails and the locations of erosion that will affect the Park Trails. The red numbers refer to Figures that discuss and show the erosion problems. The effects of erosion need to be considered in trail design and engineering.

If the simple measures discussed above are not implemented, the Park Trail system will continue to be negatively impacted. In order to spend the Coastal Conservancy and private grant funds for trails in the Hillside Park wisely, a relatively small investment needs to be made now to prevent further damage due to uncontrolled runoff.

[1] SCA Drainage Conditions and Recommendations Report for the SCNP. March 22, 2007. 45 pages

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