SUNSET CLIFFS NATURAL PARK
HYDROLOGY AND HYDRAULIC ANALYSIS

FINAL REPORT

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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:

$$Q = C\ I\ A$$

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 the 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:

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

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:

$$T_c = T_i + T_t$$

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 calculate 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 minute
I1 and I2 = Intensity at tributary area 1 and 2, in inch/hour

Select the larger Q as peak flow at the junction

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

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

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 East toward Froude Street and discharges to a curb inlet East of Froude Street. The bypass discharges to Basins X3 and X6. Basin X8 is located South and East of Basin X9 and is generally bounded by Froude Street, Granger Street and Devonshire Drive. Storm water runoff flows to Tivoli Street and drains East 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 East 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 a point in 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	Elev I	End Elev	Soil Grp	C	Tt-1	Tt-5	Tt-10	Tt-50
	Acres	ft	ft			min	min	min	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 subbasins 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 Lomaland Drive. 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.

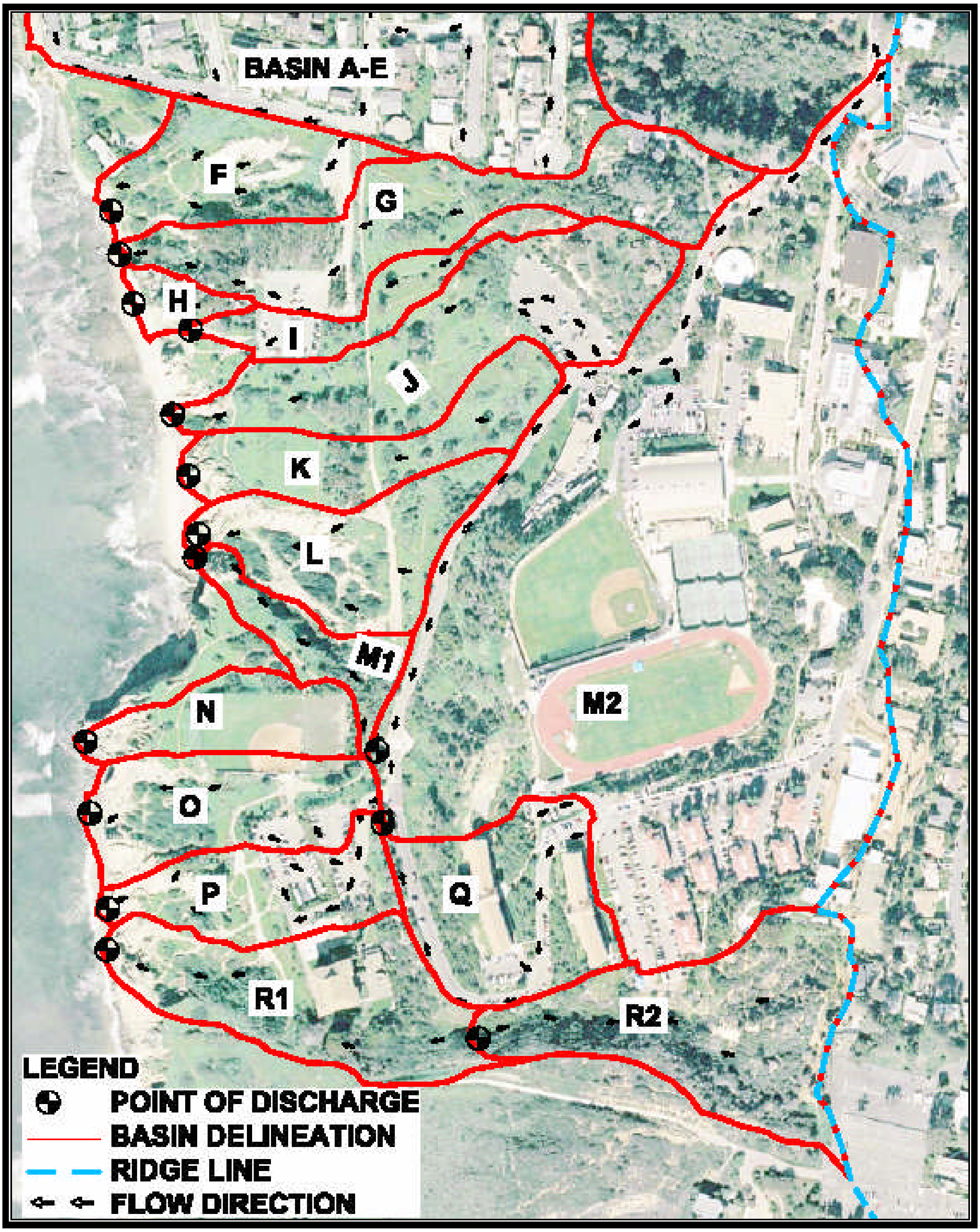

Figure 2. Drainage Basins along the Hillside Park

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

Again, Basin K is located South of, and adjacent to, Basin J. It is bounded at its East by Lomaland Drive. 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 Lomaland Drive 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 valley 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 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 Southeast 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 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. 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	High Elev	Low Elev	Soils	C	Tt
	Acres	ft	ft	Group		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 were 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 topo, (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 has less resistance and therefore transport runoff at much higher velocities, The grater velocities reduce travel time and leads to quicker peak times and higher flows. **Table 4** summarizes the developed data.

Table 4

	1 Year Storm		5 Year Storm		10 Year Storm		50 Year Storm	
Basin	I	Q	I	Q	I	Q	I	Q
	in/hr	cfs	in/hr	cfs	in/hr	cfs	in/hr	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 flows; 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 in to 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.

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

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

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-feet 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 : 1-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	30.14	57.50	1.45	8.62	19.27	0.00	0.22
X2	X2 thru X9+A	142.83	29.00	57.54	0.91	16.87	9.71	7.16	0.45
X3	X3+X6+X9	17.27	19.13	9.10	1.37	1.37	3.98	0.00	0.19
X4	X4+A	60.29	26.17	25.72	2.44	2.44	6.33	0.00	0.38
X5	X5+X7+X8	63.02	18.08	34.79	4.29	18.21	2.24	15.96	0.41
X6	X6+X9/2	9.64	18.20	5.30	2.53	2.53	4.68	0.00	0.25
X7	X7	18.10	11.36	13.48	13.48	13.48	9.76	3.72	0.30
X8	X8	37.15	16.79	21.51	21.51	21.51	11.32	10.20	0.36
X9	X9	10.07	14.69	6.35	6.35	6.35	6.81	0.00	0.29
A	A	54.56	14.22	34.42	34.42	34.42	31.10	3.32	0.67
B1	B1-4+B6	99.95	25.38	43.82	2.84	5.95	18.30	0.00	0.26
B2	B2-4+B6	93.47	23.31	43.44	5.59	5.92	2.81	3.11	0.25
B3	B3+B4+B6	81.45	22.25	39.33	22.96	25.63	25.30	0.33	0.38
B4	B4+B6	33.90	22.00	16.49	6.55	8.67	6.00	2.67	0.24
B5	B5	3.09	12.15	2.07	2.07	2.07	2.85	0.00	0.22
B6	B6	20.44	16.67	11.88	11.88	11.88	9.76	2.12	0.30
C1	C1+C3+C4	42.36	23.38	19.46	2.86	14.20	14.23	0.00	0.29
C2	C2+D1+D2	31.58	17.74	17.41	1.84	2.78	5.30	0.00	0.25
C3	C3	17.52	15.32	10.54	10.54	10.54	5.90	4.64	0.38
C4	C4	18.62	15.72	11.25	11.25	11.25	4.54	6.71	0.27
D1	D1+D2	28.25	15.90	16.81	6.22	6.22	5.28	0.94	0.40
D2	D2	17.80	12.43	12.51	12.51	12.51	12.51	0.00	0.98
E	E	12.91	15.55	7.69	7.69	7.69	14.40	0.00	0.24

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	Drainage	Area	Tc	Qt	Qb	Qi	Icap	Qby	D
Inlet	Zone	ACRE	MIN	CFS	CFS	CFS	MIN	CFS	FT
X1	X1 thru X9 + A	146.53	25.56	95.94	2.42	36.16	36.21	0.51	0.00
X2	X2 thru X9+A	142.83	24.42	96.44	1.52	46.98	13.24	0.63	33.74
X3	X3+X6+X9	17.27	18.35	14.03	2.11	3.27	4.32	0.22	0.00
X4	X4+A	60.29	22.34	42.72	4.06	20.58	9.32	0.59	11.27
X5	X5+X7+X8	63.02	17.42	53.44	6.60	36.85	2.66	0.50	34.19
X6	X6+X9/2	9.64	17.34	8.20	3.92	5.03	4.98	0.28	0.05
X7	X7	18.10	10.81	20.88	20.88	20.88	11.07	0.35	9.81
X8	X8	37.15	16.20	33.01	33.01	33.01	12.56	0.41	20.45
X9	X9	10.07	14.09	9.79	9.79	9.79	7.56	0.33	2.23
A	A	54.56	13.60	53.14	53.14	53.14	31.10	0.67	22.04
B1	B1-4+B6	99.95	19.15	67.14	4.35	28.24	28.25	0.46	0.00
B2	B2-4+B6	93.47	15.23	66.47	8.55	26.88	2.99	0.28	23.89
B3	B3+B4+B6	81.45	21.62	60.09	35.08	46.07	27.74	0.43	18.33
B4	B4+B6	33.90	21.39	25.18	10.00	17.55	6.56	0.27	10.99
B5	B5	3.09	11.90	3.14	3.14	3.14	3.55	0.26	0.00
B6	B6	20.44	16.07	18.26	18.26	18.26	10.71	0.34	7.55
C1	C1+C3+C4	42.36	22.16	30.21	4.43	26.05	26.06	0.65	0.00
C2	C2+D1+D2	31.58	17.10	26.74	2.82	6.43	6.53	0.33	0.00
C3	C3	17.52	14.99	16.03	16.03	16.03	6.54	0.43	9.49
C4	C4	18.62	15.44	17.07	17.07	17.07	4.94	0.30	12.13
D1	D1+D2	28.25	15.35	25.79	9.54	9.54	5.94	0.46	3.60
D2	D2	17.80	12.06	19.14	19.14	19.14	19.14	2.30	0.00
E	E	12.91	15.38	11.62	11.62	11.62	15.51	0.27	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 8 : 50-year Storm									
Basin Inlet	**Drainage Zone**	**Area ACRE**	**Tc MIN**	**Qt CFS**	**Qb CFS**	**Qi 0**	**Icap 0**	**Qby 0**	**D FT**
X1	X1 thru X9 + A	146.53	24.91	123.55	3.12	66.68	67.06	1.16	0.00
X2	X2 thru X9+A	142.83	23.83	124.10	1.96	77.63	14.07	0.67	63.55
X3	X3+X6+X9	17.27	18.07	17.95	2.70	6.94	4.50	0.24	2.44
X4	X4+A	60.29	21.80	54.98	5.22	33.19	10.87	0.69	22.32
X5	X5+X7+X8	63.02	16.64	69.74	8.61	53.57	2.66	0.50	50.91
X6	X6+X9/2	9.64	17.00	10.52	5.02	7.25	5.23	0.30	2.02
X7	X7	18.10	10.57	26.83	26.83	26.83	11.81	0.38	15.03
X8	X8	37.15	15.51	43.01	43.01	43.01	13.07	0.43	29.94
X9	X9	10.07	13.86	12.53	12.53	12.53	8.08	0.36	4.45
A	A	54.56	13.27	68.40	68.40	68.40	31.10	0.67	37.29
B1	B1-4+B6	99.95	19.02	86.07	5.58	46.12	37.77	0.67	8.35
B2	B2-4+B6	93.47	15.15	85.20	10.96	43.72	3.18	0.30	40.55
B3	B3+B4+B6	81.45	21.24	77.01	44.96	62.72	29.96	0.47	32.76
B4	B4+B6	33.90	21.02	32.26	12.81	24.49	6.72	0.28	17.76
B5	B5	3.09	11.68	4.03	4.03	4.03	4.54	0.30	0.00
B6	B6	20.44	15.69	23.48	23.48	23.48	11.81	0.38	11.68
C1	C1+C3+C4	42.36	22.25	38.17	5.60	35.39	26.66	0.67	8.73
C2	C2+D1+D2	31.58	16.93	34.09	3.60	9.35	9.36	0.53	0.00
C3	C3	17.52	14.71	20.55	20.55	20.55	7.19	0.48	13.36
C4	C4	18.62	15.33	21.72	21.72	21.72	5.29	0.33	16.43
D1	D1+D2	28.25	15.23	32.83	12.14	12.14	6.39	0.50	5.75
D2	D2	17.80	12.05	24.26	24.26	24.26	24.26	3.69	0.00
E	E	12.91	15.27	14.78	14.78	14.78	16.60	0.29	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.

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 Lomaland 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 Lomaland 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

			1 Year		5 Year		10 Year		50 Year	
Basin	Drainage	Area	I	Q	I	Q	I	Q	I	Q
Outlet	Zones	ACRE	IN/HR	CFS	IN/HR	CFS	IN/HR	CFS	IN/HR	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

APPENDIX