CEQA Drainage/ Hydrology Study for

Jamacha Blvd Building

Spring Valley Ca 91977

APN # 579-300-32-00

And lot

APN # 579-300-33-00

Prepared By

Bajoua Engineering

Civil & Structural Engineering Services

3330 Greystone Drive Jamul CA, 91935

Tel (619) 244-9082

Prepared: Dec 01, 2018 (Submitted to County)

Revised: April 10, 2020 (Submitted to County)

Revised: Nov. 7 , 2020 (Submitted to County)

Revised: Sep 19, 2022

Prepared For

Mr. Mark Khouli

REGISTERED PROFESSIONAL ENGINEER
FAHMI MICHAEL BAJOUA
No. 45406
Exp. 9.30.24
CIVIL
STATE OF CALIFORNIA

F. Michael Bajoua

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1.0 Declaration of Responsible Charge:

I Hereby declare that I am the Engineer of Work for this project, that I am here exercised responsible charge over the design of the project as defined in section 6730 of the business and professions code, and that the design is consistent with the current standards.

Understand the check of project drawings and specifications by the County of San Diego is confined to review only and does not relieve me, as Engineer of work, of my responsibilities for project design.

REGISTERED PROFESSIONAL ENGINEER
FAHMI MICHAEL BAJOUA
No. 45406
Exp. 9.30.20
CIVIL
STATE OF CALIFORNIA

F. M. Bajoua 3-7-2020

2. Project / Site Description:

This project provides for design and construction of commercial building of 12,000 SF of retail space, 48 parking spaces to be paved with Permeable Pavement, and landscaping as required by the County of San Diego. The project will be built on two adjacent parcels owned by the same owner along Jamacha Blvd., Spring Valley, CA 91977 (parcel lot APN 579-300-32-00, and APN 579-300-33-00).

These are vacant lots and both lots together create almost rectangular shaped site of approximately 46,450 **SF (46,450/ 43560 = 1.06 Acre.**

3. Calculation Methodology:

Runoff calculations were performed in conformance with the requirements of the County of San Diego Drainage Design Manual dated July 2005 and the San Diego County Hydrology Manual dated June 2003. The Rational Method Procedure was used for drainage calculations with the following parameters:

<u>Storm Event:</u>

100-year, 24-hour event from Isopluvial Maps (Appendix 3)

P24= 5.8 inches

100-year, 6-hour event from Isopluvial Maps (Appendix 3)

P6= 2.8 inches

<u>Runoff Coefficient:</u>

Estimated actual imperviousness of each sub-basin.

The peak flow rate of runoff formula is:

Q = CIA

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 time of concentration for the area, inch per hour (in. /hr.)

I = 4.8 in/ hr. for Pre- Development Pre- Development.

I = 5.1 in/ hr. for Post- Development and Post development are tabulated below:

The storm water discharge for Pre- Development and Post development are tabulated in next pages.

4. Pre- Development:

The stormwater runoff for Pre- Development run as sheet flow through the vacant lot toward Jamacha Blvd.

There is no storm water flow coming from adjacent parcels to this lot. The Most southerly lot has a Braw Ditch where the storm water drain from West to East parallel to the Property line (about 5 feet from the project property line) and then turn about 90 degrees and drain from South to North through the Easterly adjacent parcel, and then drains to Jamacha Blvd. through existing curb outlet.

Pre- Development:

	C	T (min	I (inch/ hr	A (Arcs)	Q (cfs)	V (fps)
Pre- Development	0.35	10	4.8	1.06	1.78	0.80 Sheet flow

5. Post- Development:

Area A-1

The parking spaces will be paved with Permeable Pavement and some landscaping along the eastside of the property as shown on the plans.

The onsite Runoff from the roofs of the proposed building is designed to flow to via downspouts and discharge to permeable pavement and will be collected by the catch basin at the southeast corner of the parking. Then the discharge will drain via 8" PVC pipe to the 2nd catch basin which will be tied to curb outlet per San Diego Regional Standard Drawing (SDRSD) D-25.

Area A-2:

The area A-2 is fairly small, landscaped area. The storm water discharge (Q-2) of 0.09 cfs will drain to two sidewalk underdrain pipes per SDRSD D-27.

Post- Development:

	C	T (min	I (inch/ hr	A (Acres)	Q (cfs)	V (fps)
A-1 (building, parking lot and landscaped area	0.35	8.3	4.8	0.88	1.48	2.98
A-2; Landscaped area along Jamacha road	0.10	10	4.8	0.18	0.09	2.50

6. Pre- Development vs Post- Development:

The stormwater runoff for pre- development is 1.85 cfs, and for post-development is 1.57 fps. The difference in storm water discharge between **Pre-Development and Post development is -** 0.28 cfs which is a decrease in the storm water discharge on Jamacha Boulevard and won't impact the capacity of the existing stormwater drain system.

7. Summary/ Conclusion:

This project includes proposed 12,000 SF of commercial building to be built on vacant two lots of **43560 (1.06 Acre) total.** The vacant land is sloped toward Jamacha Blvd where the storm water drains form South of the property's boundary toward Jamacha Blvd in the North East Direction (see attached hydrology plans for pre- development)

For post development, also the storm water from area A-1 will drain in the North East Direction, and will be collected at the catch basin located at the southeast corner of the parking lot. Then the discharge will drain via 8" PVC pipe to the 2nd catch basin which will be tied to curb outlet D-25. Area A-2 is fairly small, and landscaped where the storm water discharge (Q-2) of 0.09 cfs will drain to two sidewalk underdrain pipes per SDRSD D-27.

The difference in storm water discharge between Pre- Development and Post development is negative value of -0.28 cfs due to **pervious concrete pavement and vegetated and landscaped areas.**

The proposed building is not placed within 100- year flood hazard area, and it is not mapped on flood hazard boundary, or flood insurance rate map, or County of San Diego Flood Maps in the flood zone (see Appendix A-5/ FEMA Flood Map).

References:

1. **San Diego County Hydrology Manual, 2003**
2. **County of San Hydraulic Design Manual, 2014**

Project:	**Bajoua Engineering**	Page: -of-
Designed by: **Mike Bajoua, P.E**	**Lic # C45046**	Project #:
Checked by:	*Civil & Structural Engineering Services*	Date: Revised:

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Hydrology and Drainage Calculations:

Pre- Development:

C = 0.35 (per table 3-1)

T = Time of Concentration or Travel Time for Natural watersheds.

T ≅ 0 (per Figure 3-4)

where $\Delta E = 20$ and L ≅ 230' (see next page).

T = 0 + 10 min (Add ten min per Section 3.1.4.1B (Natural or Rural Watersheds)

I = 4.8 inch/hr per (Fig 3-2)

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$$T_c = \left(\frac{11.9 L^3}{\Delta E}\right)^{0.385} \quad (\text{Fig } 3-4).$$

$\Delta = 18.5$ From (Figure 3-5)(Change in Elev. along Effective slope).

$L = 220'$

$$T = \left(\frac{11.9 \times (0.0416)^3}{18.5}\right)^{0.385} \simeq 0 \text{ min}$$

Where T_c = Time of concentration (hrs).

ΔE = Change in Elevation along Effective slop line (Fig 3-5) Ft.

L = Water Distance (Miles)

$Q_{\text{pre-Development}} = CIA$

$= 0.35 \times 4.8$ inch/hr $\times 1.06 = 1.78$ cfs

Sheet flow for pre-Development:

$n = 0.03$

$S = 8.3\%$

$Q = 1.85$ cfs

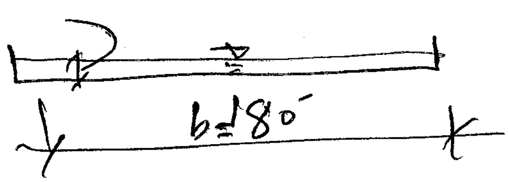
Results

Flow area	2.34	ft^2 ▼
Wetted perimeter	180.03	ft ▼
Hydraulic radius	0.01	ft ▼
Velocity, v	0.79	ft/sec ▼
Flow, Q	1.85	cfs ▼
Velocity head, h_v	0.01	ft ▼
Top width, T	180.00	ft ▼
Froude number, F	1.22	
Shear stress (tractive force), tau	0.07	psf ▼
Implied design ? riprap size based on n	0.21	ft ▼
Required bottom angular riprap size, D50, Maricopa County	0.02	ft ▼
Required side slope 1 angular riprap size, D50, Maricopa County	0.02	ft ▼
Required side slope 2 angular riprap size, D50, Maricopa County	0.02	ft ▼
Required angular riprap size, D50, per Maynord, Ruff, and Abt (1989)	NaN	ft ▼
Required angular riprap size, D50, per Searcy (1967)	0.00	ft ▼

Set units: m mm ft in

Bottom width	180 ft ▼
Side slope 1 (horiz./vert.)	0
Side slope 2 (horiz./vert.)	0
Manning roughness, n ? (http://www.engineeringtoolbox.com/mannings-roughness-d_799.html)	.030
Channel slope	8.3 % rise/run ▼
Flow depth	0.156 in ▼
Bend Angle? (/riprap-bend-angle.png) (for riprap sizing)	0
Stone specific gravity (2.65)	0

d = 0.156 inch.



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Post Development: For Area "A-1" (see Appendix ————).

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

$$T = \frac{1.8(1.1 - 0.53)\sqrt{271}}{\sqrt[3]{4.8}} = 10 \text{ min}$$

Area A-1: A-1 = 0.28 Acre Building + 0.6 Acre concrete pervious pavement for parking spaces & landscape Area

$$\boxed{C = 0.9 \times \%\,\text{Imp} + C_p \times (1 - \%\,\text{Imp})} \quad (\text{See } 3.1.2)$$

$$C_{building} = 0.9 \times 1 + 0.35(1-1) = 0.90$$

→ using Type "D" soil $C_p = 0.35$

$C_{\text{pervious concrete pavement}} = 0.10$ (per Table B.1-1 County of S.D Storm Water Manual)

$$C_{A-1} = \frac{\Sigma C_i A_i}{\Sigma A} = \frac{0.90 \times 0.28 + 0.10 \times 0.60}{0.28\ Ac + 0.60\ Ac} = \frac{0.252 + 0.06}{0.88} = 0.35$$

$I = 4.8$ (Fig 3-1)

$$Q_{A-1} = CIA = 0.35 \times 4.80 \times 0.88\ Ac = 1.48 \text{ cfs}$$

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④ Post Development Area "A-2":

$A_2 = 0.18$ Acre → Landscaped Area

$C_{A-2} = 0.9 \times \%\,Imp. + C_p^*(1 - \%\,Imp)$

$C_{A-2} = 0.9 \times 0 + 0.10\,(1 - 0)$

*(per Table B.1-1 County of S.D Stormwater Manual).

$C_{A-2} = 0.10$

$Q_{A-2} = C_{A2}\, I_{A2}\, A_{A2}$

$= 0.10 \times 4.8 \times 0.18$

$Q_{A-2} = 0.09$ cfs

$Q_{A-1} + Q_{A-2} = 1.48 + 0.09 = 1.57$ cfs

$\Delta_{\text{post Dev.} - \text{pre-Dev.}} = 1.57 - 1.85 = -0.28$ cfs

i.e Q Been Reduced

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Calculation for sidewalk underdrain pipe (D-27):

$Q = 0.09$ cfs ; $S = 2\%$; 3" P.V.C pipe

$$Q_{full} = A \times \frac{1.49}{n} (r_H)^{0.667} \sqrt{S}$$

$$A_{3"\,PVC} = 0.785\,D^2 = 0.785 \times \left(\frac{3}{12}\right)^2 = 0.0491 \text{ ft}^2$$

$$r_H = \frac{1}{4} D = \frac{1}{4} \times \frac{3"}{12} = 0.0625$$; $n = 0.012$ for P.V.C pipe

$S = 2\%$

$$Q = 0.0491 \times \frac{1.49}{0.012} \times (0.0625)^{0.667} \times \sqrt{0.02}$$

$$Q_{full} = 6.10 \times 0.157 \times 0.141$$

$$= 0.135 \text{ cfs}$$

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$$V_{full} = \frac{Q_{full}}{A}$$

$$V_{full} = \frac{0.135}{0.0491} = 2.75 \text{ cfs}$$

$$\frac{Q}{Q_{full}} = \frac{0.045}{0.135} = 0.34$$ → From Circular Channel Ratios (Appendix C) Civil Engineering Reference Manual 4th Ed.

$$\frac{d}{D} = 0.46$$

$$\frac{V}{V_{full}} = 0.76$$

$$V = 0.76 \times 2.75 = 2.10 \text{ cfs}$$

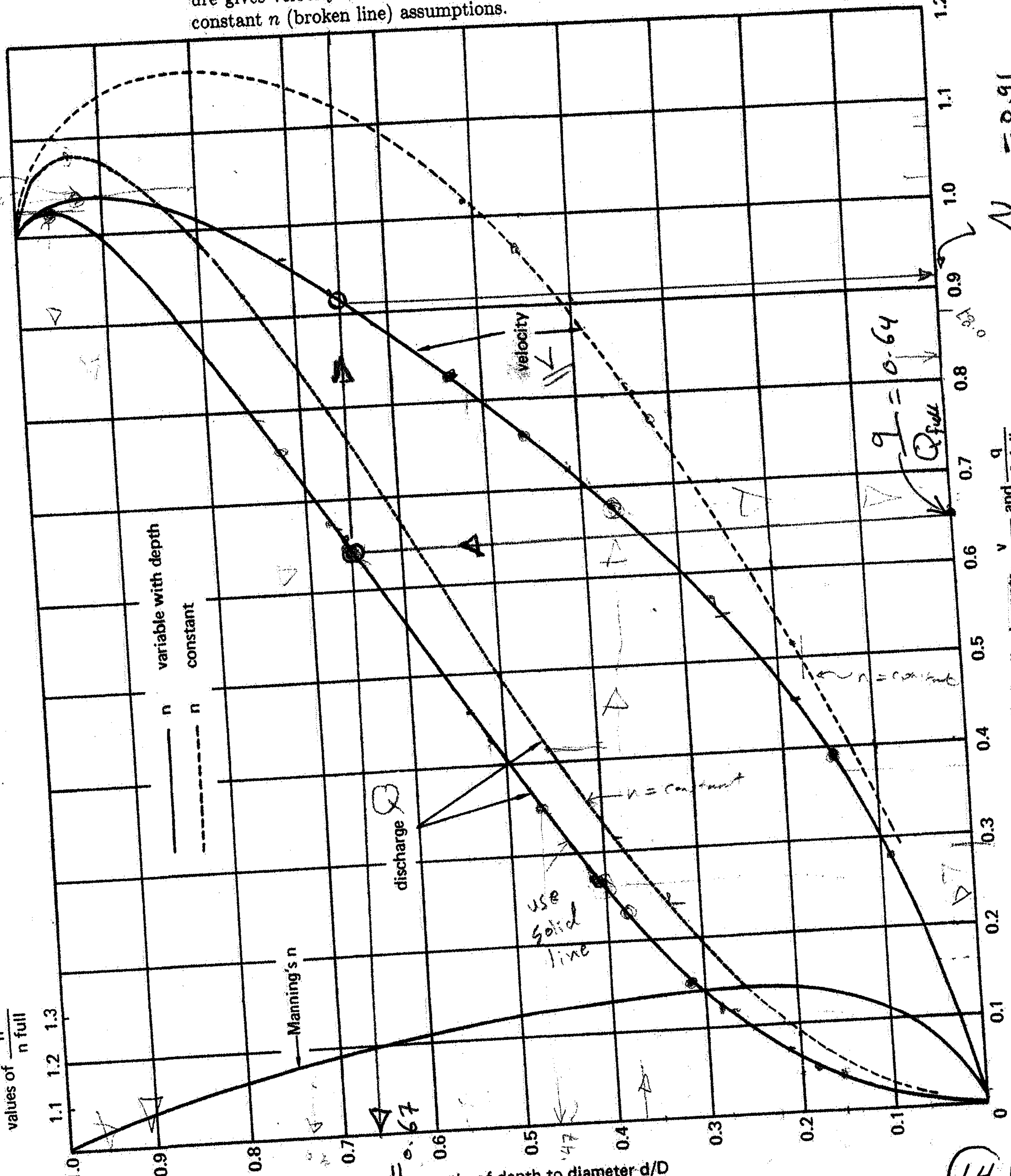
or (5-4) ← 3-43 w.b.

5

Appendix C: Circular Channel Ratios

D d only

Experiments have shown that n varies slightly with depth. This figure gives velocity and flow rate ratios for varying n (solid line) and constant n (broken line) assumptions.



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Manning Formula Uniform Trapezoidal Channel Flow at Given Slope and Depth

Trapezoidal Channel Flow (Curb Outlet (D-25) ,

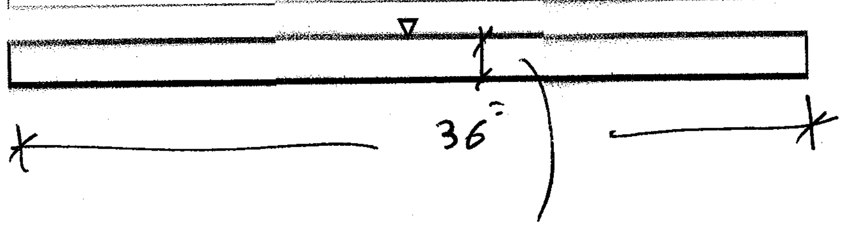
Jamacha Building

Inputs

Bottom width	3	ft
Side slope 1 (horiz./vert.)	0	
Side slope 2 (horiz./vert.)	0	
Manning roughness, n ?	0.014	
Channel slope	1.0	% rise/run
Flow depth	1.99	in
Bend Angle? (for riprap sizing)	0	
Rock specific gravity (2.65)		
Median rock size		in

Results

Flow area	0.4975	ft^2
Wetted perimeter	3.3317	ft
Hydraulic radius	0.1493	ft
Velocity, v	2.9873	ft/sec
Flow, Q (See notes)	1.4862	cfs
Velocity head, h_v	0.1387	ft
Top width, T	3.0000	ft
Froude number, F	1.29	
Shear stress (tractive force), tau	0.0932	psf
n for rock size per Strickler	0.000	
n for rock size per Blodgett	0.000	
n for rock size per Bathurst	NaN	
Blodgett vs. Bathurst	++++	
Required bottom angular rock size, D50 (Isbash & MC) ?	-2.0445	in
Required side slope 1 angular rock size, D50 (Isbash & MC) ?	-33389594268940668.0000	in
Required side slope 2 angular rock size, D50 (Isbash & MC) ?	-33389594268940668.0000	in
Required angular rock size, D50 (Maynord, Ruff, and Abt 1989)	NaN	in
Required angular rock size, D50 (Searcy 1967)	0.7181	in



d = 1.99"

Curb outlet (D-25).

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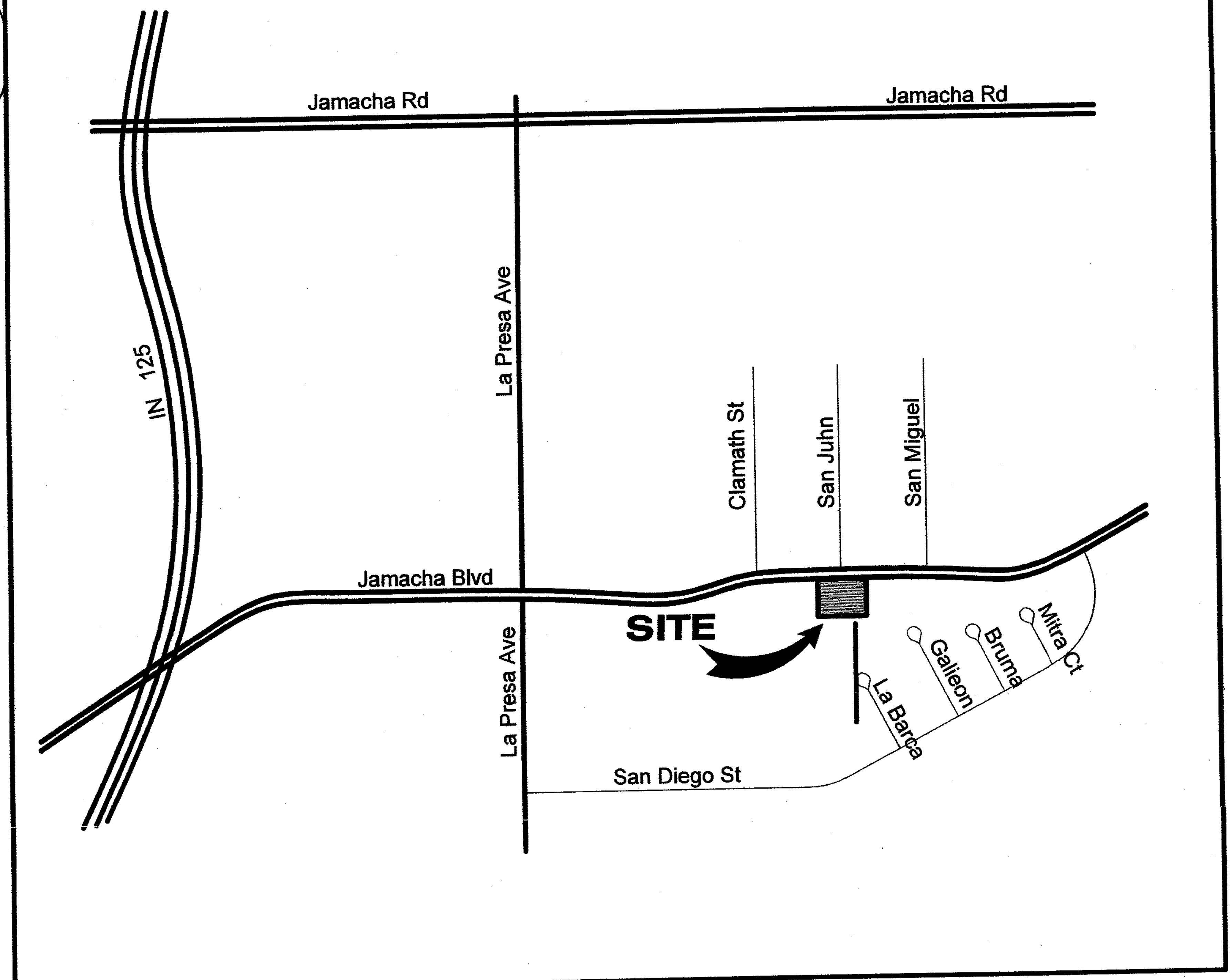
Appendix A-1

Vicinity Map

P: A 1.1

VICINITY MAP

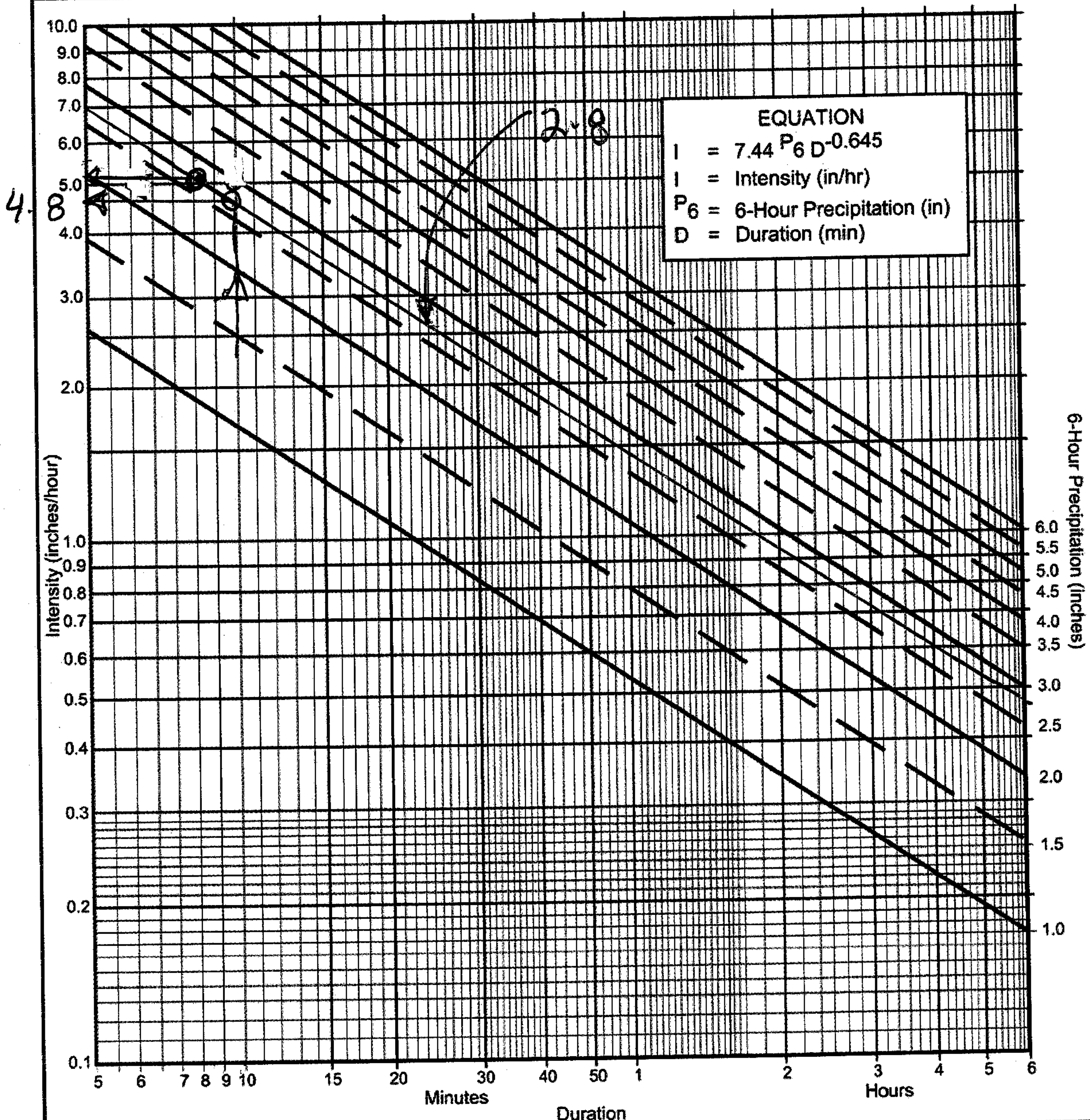


Appendix A-2

San Diego Hydrology Manual Figures 1 through 5

(as we applied for calculations)

P: A2.1



Directions for Application:

(1) From precipitation maps determine 6 hr and 24 hr amounts for the selected frequency. These maps are included in the County Hydrology Manual (10, 50, and 100 yr maps included in the Design and Procedure Manual).

(2) Adjust 6 hr precipitation (if necessary) so that it is within the range of 45% to 65% of the 24 hr precipitation (not applicaple to Desert).

(3) Plot 6 hr precipitation on the right side of the chart.

(4) Draw a line through the point parallel to the plotted lines.

(5) This line is the intensity-duration curve for the location being analyzed.

Application Form:

(a) Selected frequency 100 year

(b) P_6 = 2.8 in., P_{24} = 5.8, $\frac{P_6}{P_{24}}$ = 48 %(2)

(c) Adjusted $P_6^{(2)}$ = 2.8 in.

(d) t_x = 10 min.

(e) I = 4.81 in./hr.

Note: This chart replaces the Intensity-Duration-Frequency curves used since 1965.

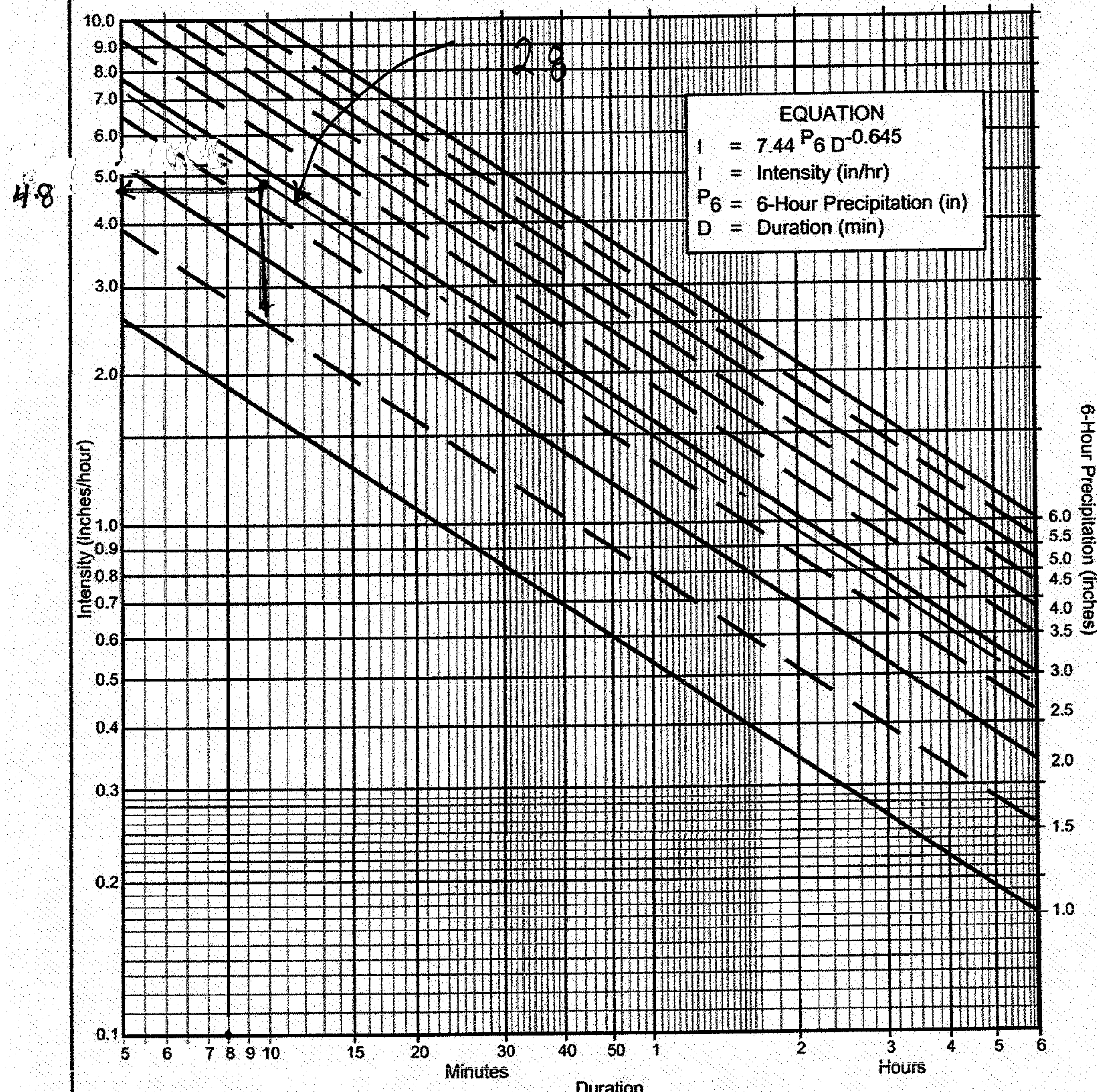
P6 / Duration	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
5	2.63	3.95	5.27	6.59	7.90	9.22	10.54	11.86	13.17	14.49	15.81
7	2.12	3.18	4.24	5.30	6.36	7.42	8.48	9.54	10.60	11.66	12.72
10	1.68	2.53	3.37	4.21	5.05	5.90	6.74	7.58	8.42	9.27	10.11
15	1.30	1.95	2.59	3.24	3.89	4.54	5.19	5.84	6.49	7.13	7.78
20	1.08	1.62	2.15	2.69	3.23	3.77	4.31	4.85	5.39	5.93	6.46
25	0.93	1.40	1.87	2.33	2.80	3.27	3.73	4.20	4.67	5.13	5.60
30	0.83	1.24	1.66	2.07	2.49	2.90	3.32	3.73	4.15	4.56	4.98
40	0.69	1.03	1.38	1.72	2.07	2.41	2.76	3.10	3.45	3.79	4.13
50	0.60	0.90	1.19	1.49	1.79	2.09	2.39	2.69	2.98	3.28	3.58
60	0.53	0.80	1.06	1.33	1.59	1.86	2.12	2.39	2.65	2.92	3.18
90	0.41	0.61	0.82	1.02	1.23	1.43	1.63	1.84	2.04	2.25	2.45
120	0.34	0.51	0.68	0.85	1.02	1.19	1.36	1.53	1.70	1.87	2.04
150	0.29	0.44	0.59	0.73	0.88	1.03	1.18	1.32	1.47	1.62	1.76
180	0.26	0.39	0.52	0.65	0.78	0.91	1.04	1.18	1.31	1.44	1.57
240	0.22	0.33	0.43	0.54	0.65	0.76	0.87	0.98	1.08	1.19	1.30
300	0.19	0.28	0.38	0.47	0.56	0.66	0.75	0.85	0.94	1.03	1.13
360	0.17	0.25	0.33	0.42	0.50	0.58	0.67	0.75	0.84	0.92	1.00

FIGURE

3-1

Intensity-Duration Design Chart - Template

For Pre-Development



Directions for Application:

(1) From precipitation maps determine 6 hr and 24 hr amounts for the selected frequency. These maps are included in the County Hydrology Manual (10, 50, and 100 yr maps included in the Design and Procedure Manual).

(2) Adjust 6 hr precipitation (if necessary) so that it is within the range of 45% to 65% of the 24 hr precipitation (not applicaple to Desert).

(3) Plot 6 hr precipitation on the right side of the chart.

(4) Draw a line through the point parallel to the plotted lines.

(5) This line is the intensity-duration curve for the location being analyzed.

Application Form:

(a) Selected frequency 100 year

(b) P_6 = 2.8 in., P_{24} = 5.8, $\frac{P_6}{P_{24}}$ = 48 %(2)

(c) Adjusted $P_6^{(2)}$ = 2.8 in.

(d) t_x = 10 min.

(e) I = 4.8 in./hr.

Note: This chart replaces the Intensity-Duration-Frequency curves used since 1965.

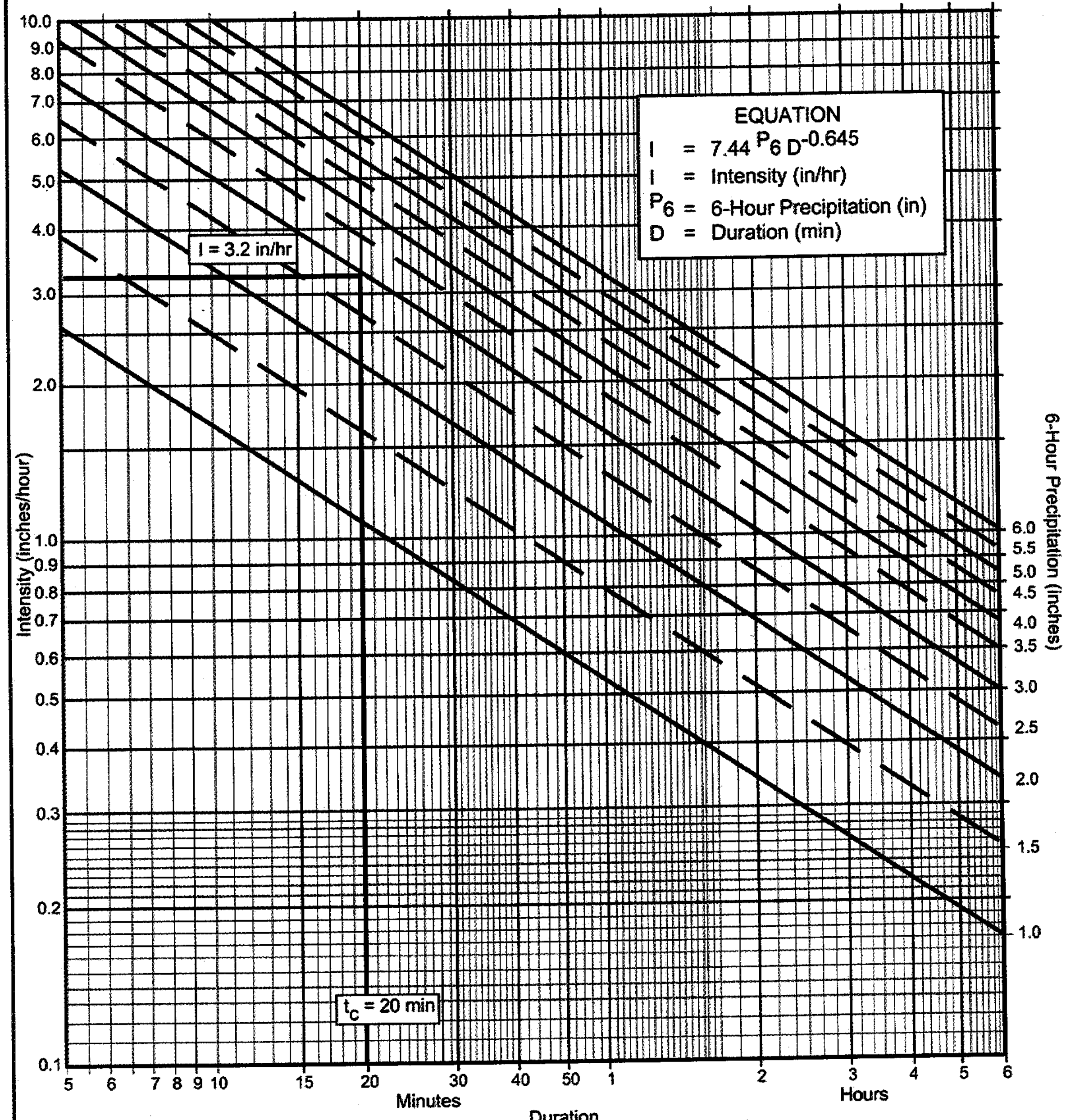
P6	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
Duration	I	I	I	I	I	I	I	I	I	I	I
5	2.63	3.95	5.27	6.59	7.90	9.22	10.54	11.86	13.17	14.49	15.81
7	2.12	3.18	4.24	5.30	6.36	7.42	8.48	9.54	10.60	11.66	12.72
10	1.68	2.53	3.37	4.21	5.05	5.90	6.74	7.58	8.42	9.27	10.11
15	1.30	1.95	2.59	3.24	3.89	4.54	5.19	5.84	6.49	7.13	7.78
20	1.08	1.62	2.15	2.69	3.23	3.77	4.31	4.85	5.39	5.93	6.46
25	0.93	1.40	1.87	2.33	2.80	3.27	3.73	4.20	4.67	5.13	5.60
30	0.83	1.24	1.66	2.07	2.49	2.90	3.32	3.73	4.15	4.56	4.98
40	0.69	1.03	1.38	1.72	2.07	2.41	2.76	3.10	3.45	3.79	4.13
50	0.60	0.90	1.19	1.49	1.79	2.09	2.39	2.69	2.98	3.28	3.58
60	0.53	0.80	1.06	1.33	1.59	1.86	2.12	2.39	2.65	2.92	3.18
90	0.41	0.61	0.82	1.02	1.23	1.43	1.63	1.84	2.04	2.25	2.45
120	0.34	0.51	0.68	0.85	1.02	1.19	1.36	1.53	1.70	1.87	2.04
150	0.29	0.44	0.59	0.73	0.88	1.03	1.18	1.32	1.47	1.62	1.76
180	0.26	0.39	0.52	0.65	0.78	0.91	1.04	1.18	1.31	1.44	1.57
240	0.22	0.33	0.43	0.54	0.65	0.76	0.87	0.98	1.08	1.19	1.30
300	0.19	0.28	0.38	0.47	0.56	0.66	0.75	0.85	0.94	1.03	1.13
360	0.17	0.25	0.33	0.42	0.50	0.58	0.67	0.75	0.84	0.92	1.00

FIGURE 3-1

Intensity-Duration Design Chart - Template

For Post-Development

p A2.3



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(1) From precipitation maps determine 6 hr and 24 hr amounts for the selected frequency. These maps are included in the County Hydrology Manual (10, 50, and 100 yr maps included in the Design and Procedure Manual).

(2) Adjust 6 hr precipitation (if necessary) so that it is within the range of 45% to 65% of the 24 hr precipitation (not applicaple to Desert).

(3) Plot 6 hr precipitation on the right side of the chart.

(4) Draw a line through the point parallel to the plotted lines.

(5) This line is the intensity-duration curve for the location being analyzed.

Application Form:

(a) Selected frequency 50 year

(b) P_6 = 3 in., P_{24} = 5.5, $\frac{P_6}{P_{24}}$ = 54.5 %(2)

(c) Adjusted $P_6^{(2)}$ = 3 in.

(d) t_x = 20 min.

(e) I = 3.2 in./hr.

Note: This chart replaces the Intensity-Duration-Frequency curves used since 1965.

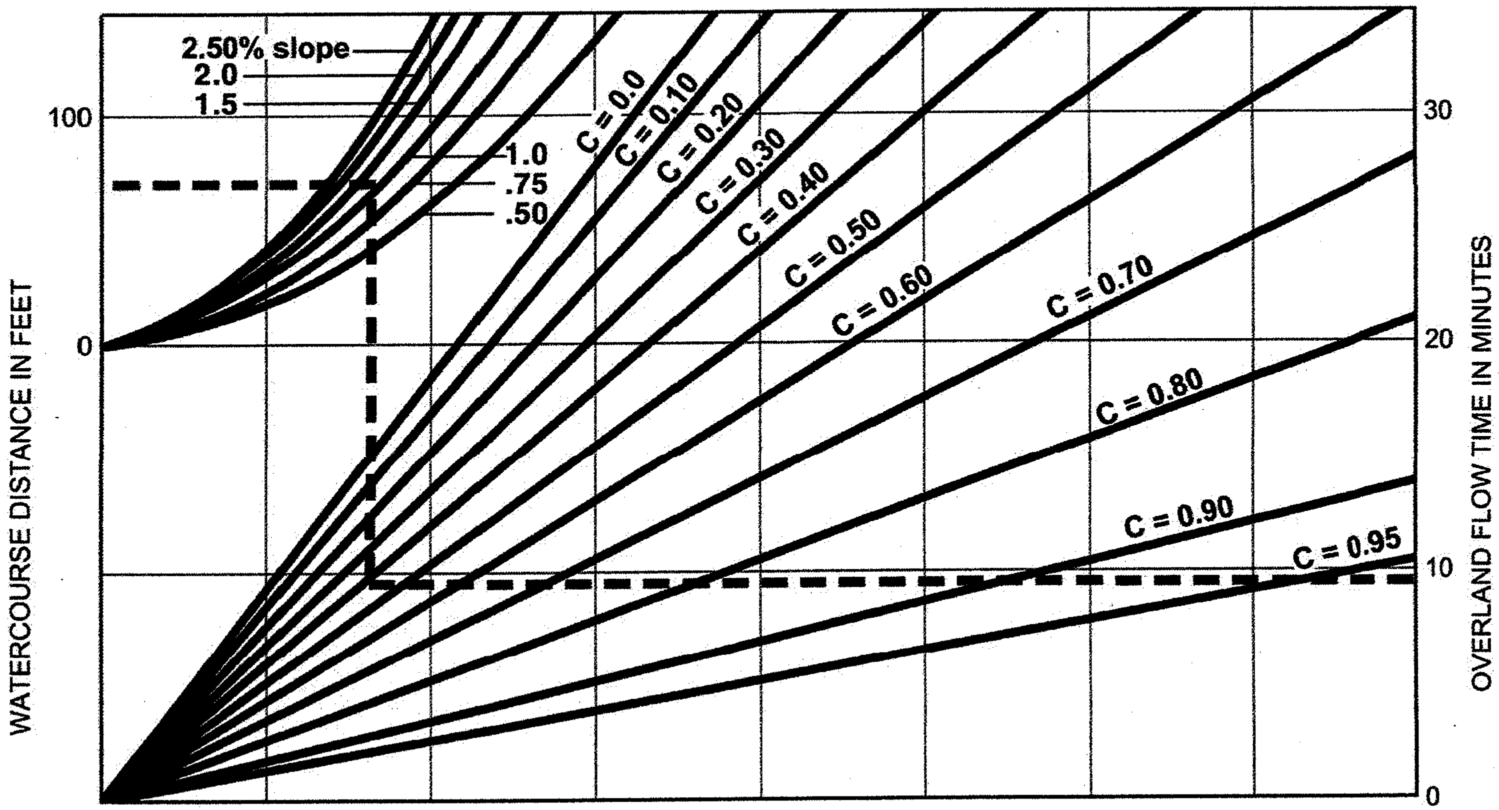
P6 / Duration	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
	I	I	I	I	I	I	I	I	I	I	I
5	2.63	3.95	5.27	6.59	7.90	9.22	10.54	11.86	13.17	14.49	15.81
7	2.12	3.18	4.24	5.30	6.36	7.42	8.48	9.54	10.60	11.66	12.72
10	1.68	2.53	3.37	4.21	5.05	5.90	6.74	7.58	8.42	9.27	10.11
15	1.30	1.95	2.59	3.24	3.89	4.54	5.19	5.84	6.49	7.13	7.78
20	1.08	1.62	2.15	2.69	3.23	3.77	4.31	4.85	5.39	5.93	6.46
25	0.93	1.40	1.87	2.33	2.80	3.27	3.73	4.20	4.67	5.13	5.60
30	0.83	1.24	1.66	2.07	2.49	2.90	3.32	3.73	4.15	4.56	4.98
40	0.69	1.03	1.38	1.72	2.07	2.41	2.76	3.10	3.45	3.79	4.13
50	0.60	0.90	1.19	1.49	1.79	2.09	2.39	2.69	2.98	3.28	3.58
60	0.53	0.80	1.06	1.33	1.59	1.86	2.12	2.39	2.65	2.92	3.18
90	0.41	0.61	0.82	1.02	1.23	1.43	1.63	1.84	2.04	2.25	2.45
120	0.34	0.51	0.68	0.85	1.02	1.19	1.36	1.53	1.70	1.87	2.04
150	0.29	0.44	0.59	0.73	0.88	1.03	1.18	1.32	1.47	1.62	1.76
180	0.26	0.39	0.52	0.65	0.78	0.91	1.04	1.18	1.31	1.44	1.57
240	0.22	0.33	0.43	0.54	0.65	0.76	0.87	0.98	1.08	1.19	1.30
300	0.19	0.28	0.38	0.47	0.56	0.66	0.75	0.85	0.94	1.03	1.13
360	0.17	0.25	0.33	0.42	0.50	0.58	0.67	0.75	0.84	0.92	1.00

Intensity-Duration Design Chart - Example

FIGURE

3-2

p: A2.4



EXAMPLE:

Given: Watercourse Distance (D) = 70 Feet
Slope (s) = 1.3%
Runoff Coefficient (C) = 0.41
Overland Flow Time (T) = 9.5 Minutes

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

SOURCE: Airport Drainage, Federal Aviation Administration, 1965

"FAA"

3-10

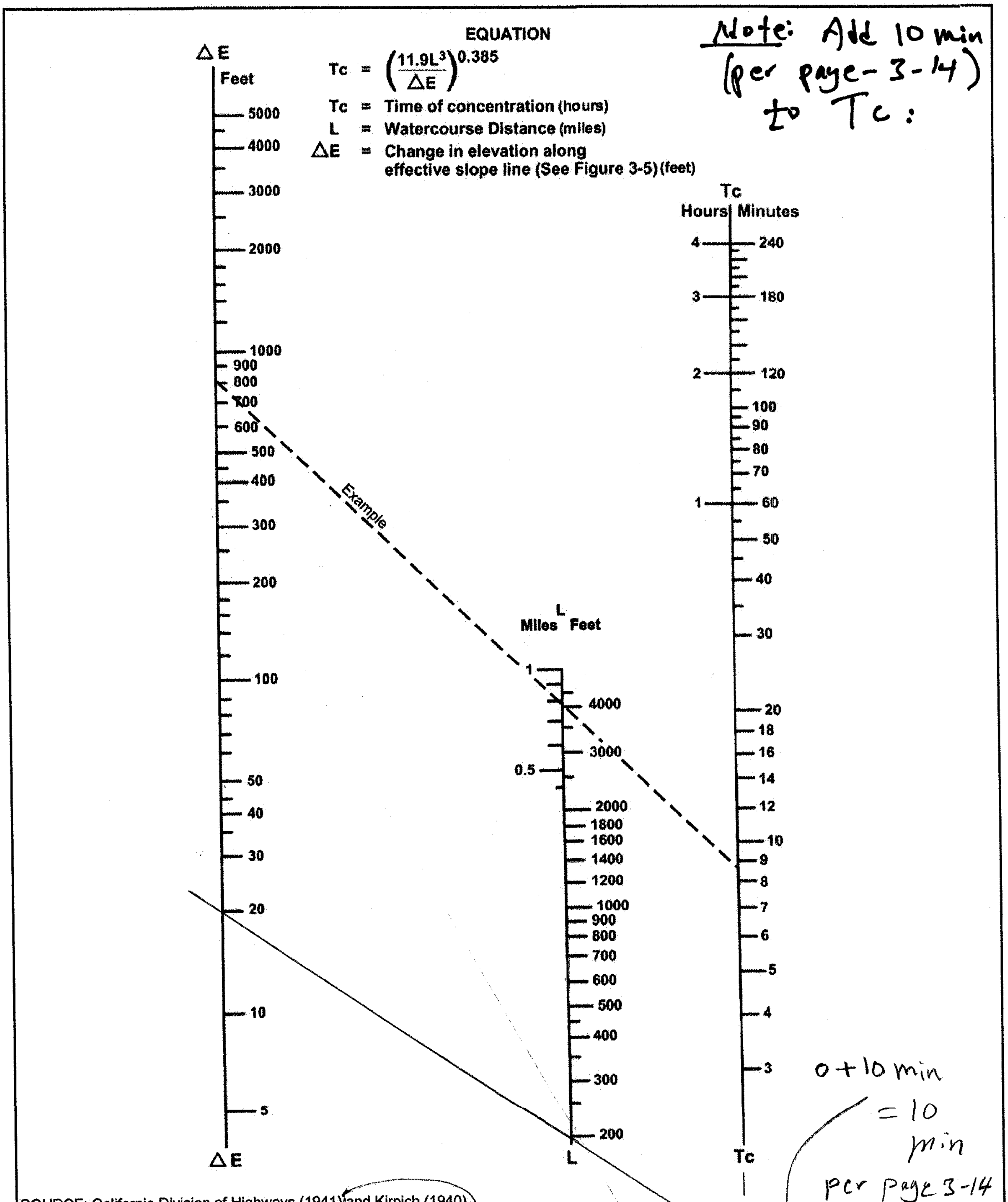
FIGURE

3-3

Rational Formula - Overland Time of Flow Nomograph

p. (A2.5)

Note: Add 10 min (per page-3-14) to Tc:

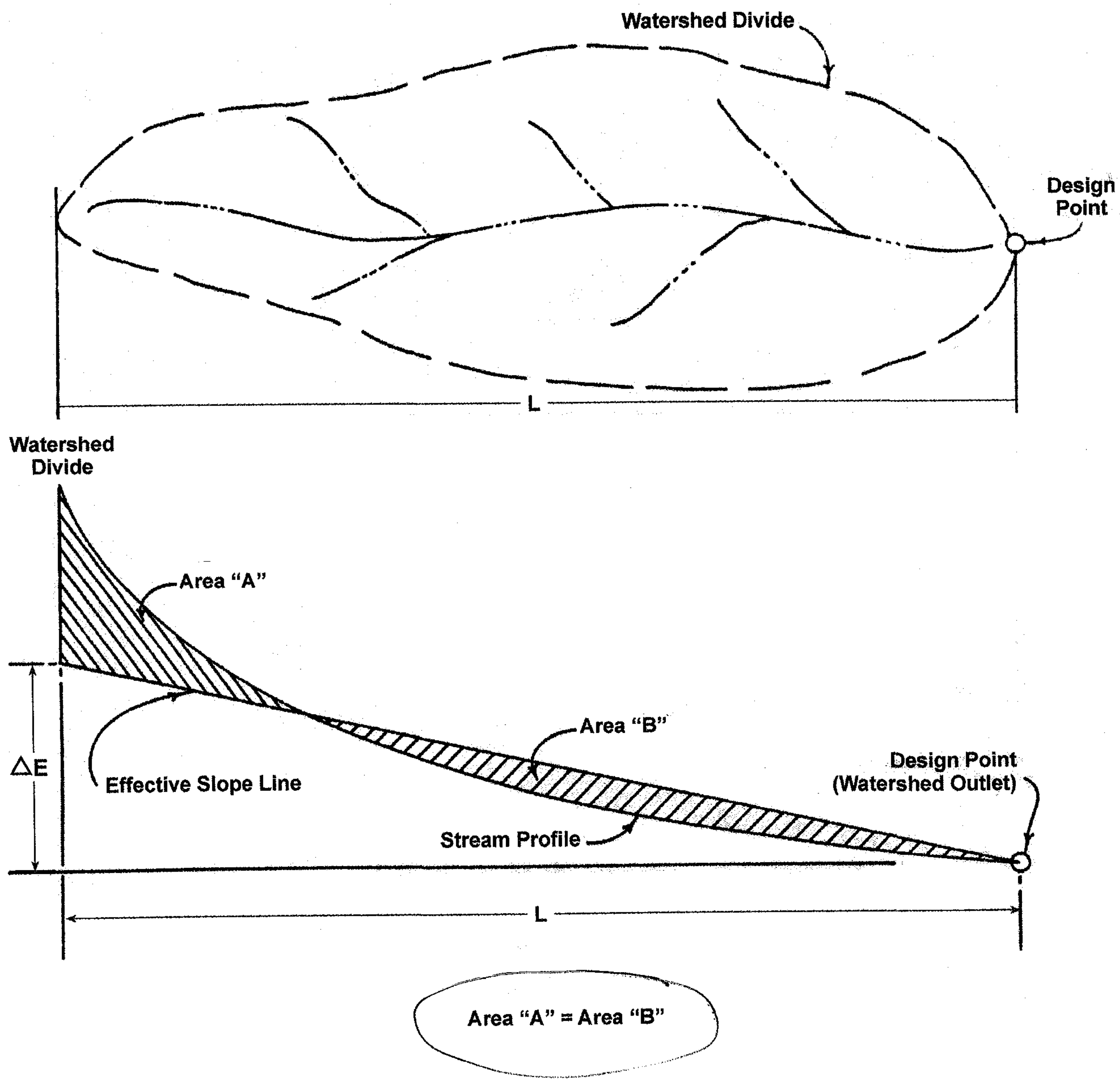


SOURCE: California Division of Highways (1941) and Kirpich (1940)

Nomograph for Determination of Time of Concentration (Tc) or Travel Time (Tt) for Natural Watersheds

FIGURE **3-4**

p: A2.6

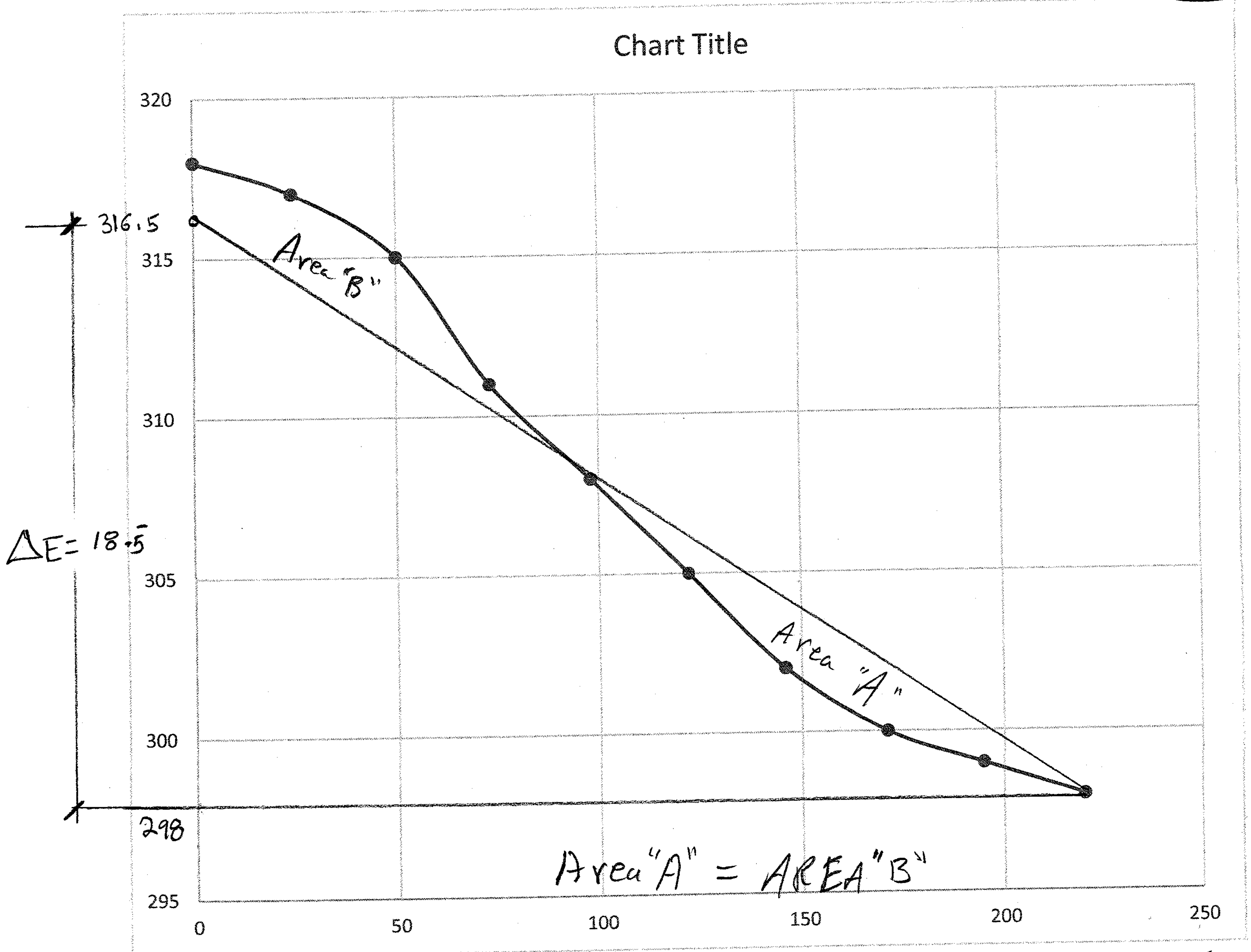


SOURCE: California Division of Highways (1941) and Kirpich (1940)

Computation of Effective Slope for Natural Watersheds

FIGURE **3-5**

A2.7

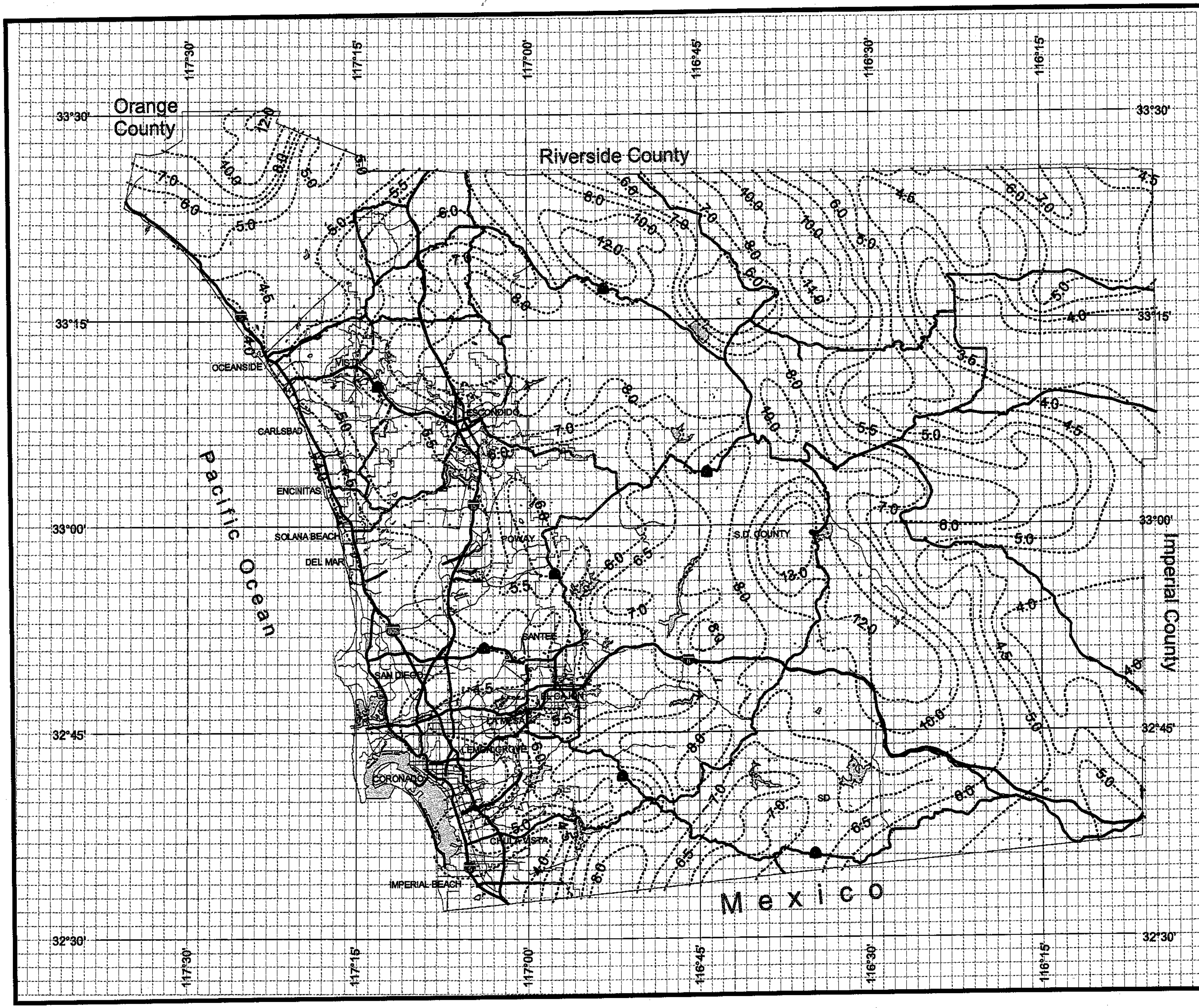


per; Fig 3-5

Appendix A-3

Rainfall Isophluvials

p: A3.1



County of San Diego Hydrology Manual

Rainfall Isopluvials

100 Year Rainfall Event - 24 Hours

-------- Isopluvial (inches)

DPW GIS
Department of Public Works Geographic Information Services

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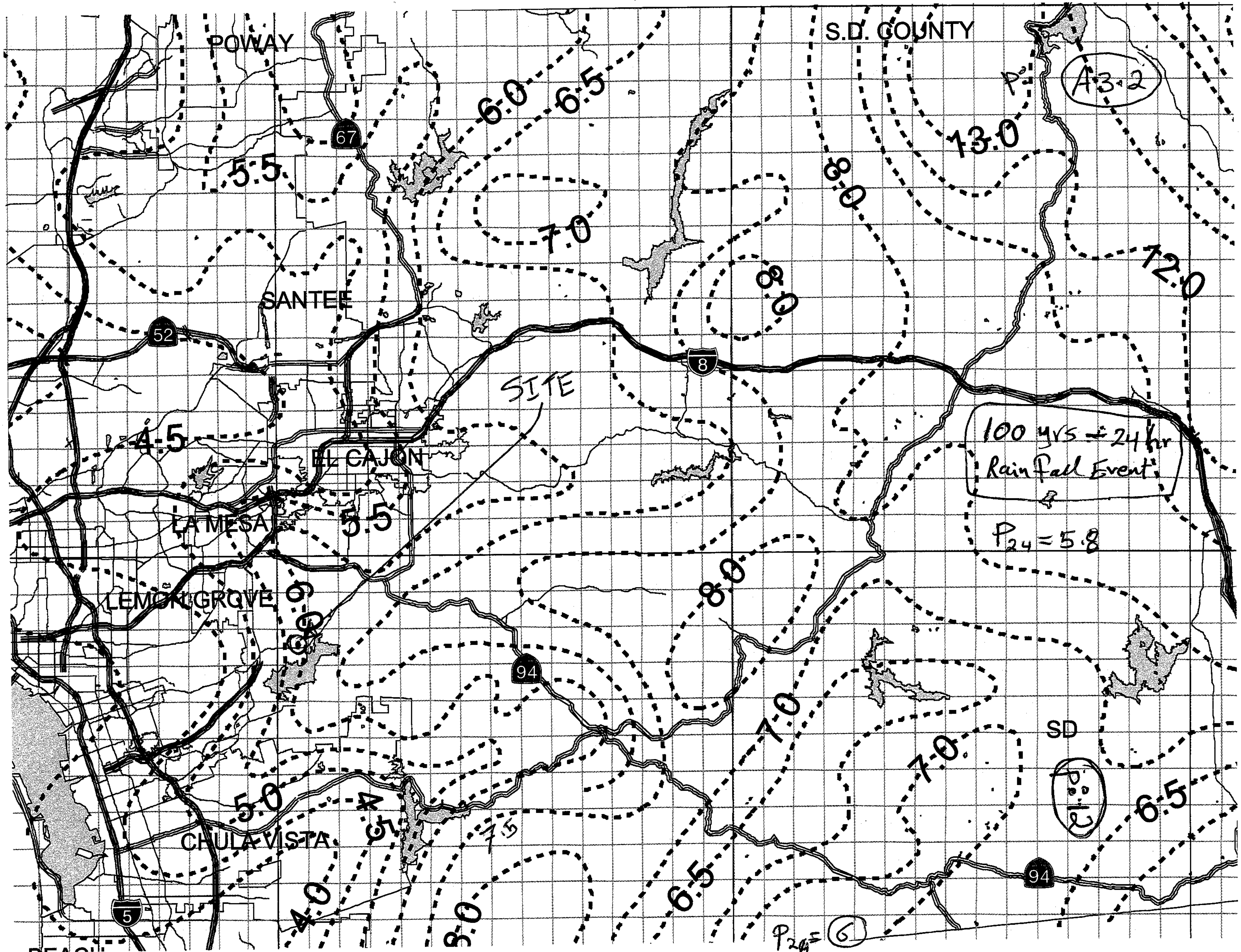
We Have San Diego Covered!

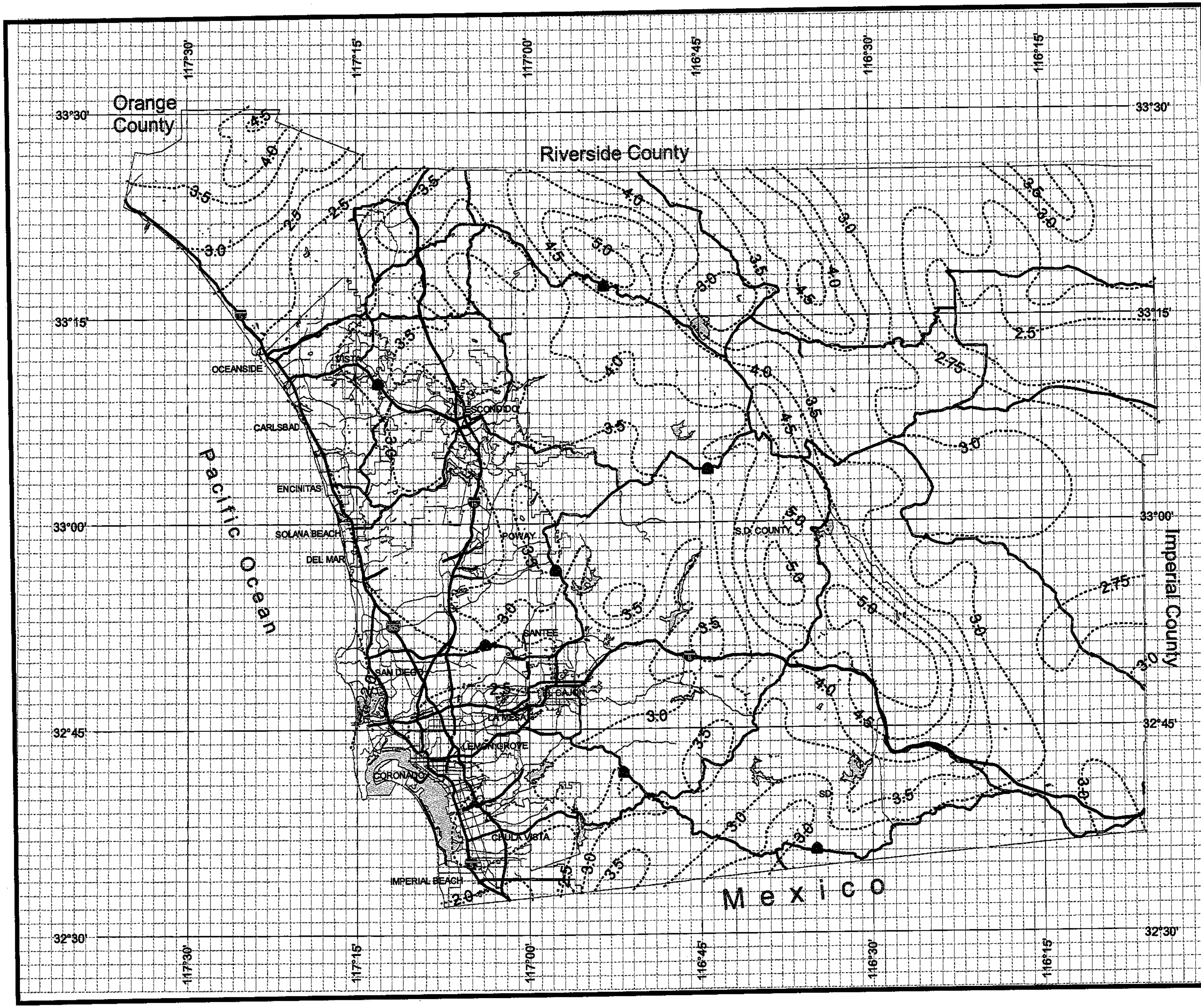
N S E W

3 0 3 Miles



POWAY
S.D. COUNTY
A3.2
6.0
6.5
13.0
5.5
67
7.0
8.0
8.0
12.0
SANTEE
52
8
SITE
4.5
EL CAJON
100 yrs - 24 hr
Rainfall Event
LA MESA
5.5
P24 = 5.8
LEMON GROVE
8.0
6.0
94
7.0
SD
7.0
6.5
5.0
CHULA VISTA
7.5
6.5
4.0
8.0
5
94
P24 = 6

p: A3-3



County of San Diego Hydrology Manual

Rainfall Isopluvials

100 Year Rainfall Event - 6 Hours

------- Isopluvial (inches)

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N S E W

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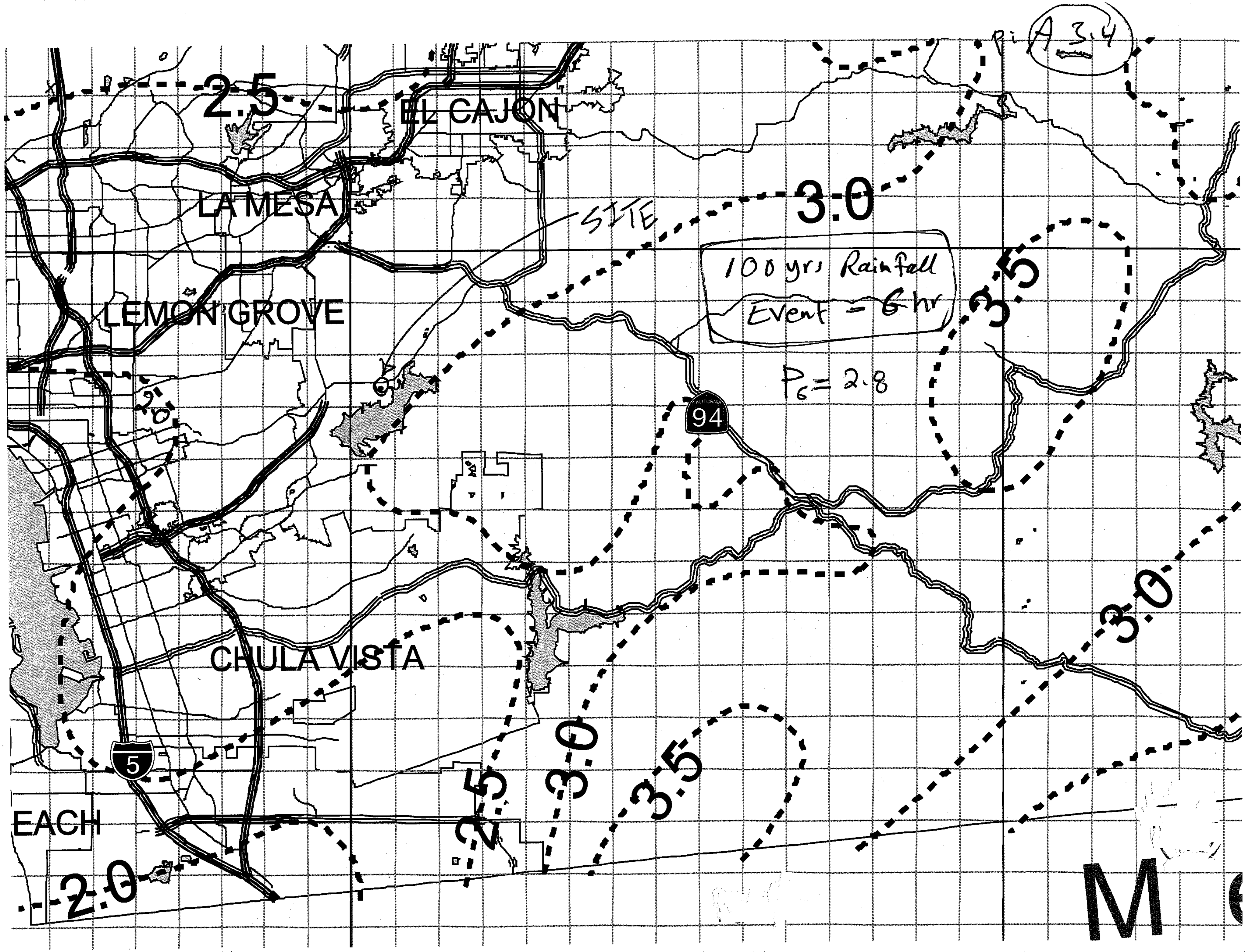
3 0 3 Miles

p.13



A 3.4
2.5
EL CAJON
LA MESA
3.0
SITE
100 yrs Rainfall
Event - 6 hr
LEMON GROVE
3.5
$P_6 = 2.8$
94
2.0
3.0
CHULA VISTA
5
3.0
2.5
3.5
EACH
2.0
M

Appendix A-4

San Diego Hydrology Manual sections

as related to Calculations.

p: A4.1

SECTION 3
RATIONAL METHOD AND MODIFIED RATIONAL METHOD

3.1 THE RATIONAL METHOD

The Rational Method (RM) is a mathematical formula used to determine the maximum runoff rate from a given rainfall. It has particular application in urban storm drainage, where it is used to estimate peak runoff rates from small urban and rural watersheds for the design of storm drains and small drainage structures. The RM is recommended for analyzing the runoff response from drainage areas up to approximately 1 square mile in size. It should not be used in instances where there is a junction of independent drainage systems or for drainage areas greater than approximately 1 square mile in size. In these instances, the Modified Rational Method (MRM) should be used for junctions of independent drainage systems in watersheds up to approximately 1 square mile in size (see Section 3.4); or the NRCS Hydrologic Method should be used for watersheds greater than approximately 1 square mile in size (see Section 4).

The RM can be applied using any design storm frequency (e.g., 100-year, 50-year, 10-year, etc.). The local agency determines the design storm frequency that must be used based on the type of project and specific local requirements. A discussion of design storm frequency is provided in Section 2.3 of this manual. A procedure has been developed that converts the 6-hour and 24-hour precipitation isopluvial map data to an Intensity-Duration curve that can be used for the rainfall intensity in the RM formula as shown in Figure 3-1. The RM is applicable to a 6-hour storm duration because the procedure uses Intensity-Duration Design Charts that are based on a 6-hour storm duration.

3.1.1 Rational Method Formula

The 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

P: A4.2

flow from the most remote point of the basin to the location being analyzed. The RM formula is expressed as follows:

$$\boxed{Q = C\,I\,A}$$

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 (Note: If the computed T_c is less than 5 minutes, use 5 minutes for computing the peak discharge, Q)
- A = drainage area contributing to the design location, in acres

Combining the units for the expression CIA yields:

$$\left(\frac{1\text{ acre} \times \text{inch}}{\text{hour}}\right)\left(\frac{43{,}560\text{ ft}^2}{\text{acre}}\right)\left(\frac{1\text{ foot}}{12\text{ inches}}\right)\left(\frac{1\text{ hour}}{3{,}600\text{ seconds}}\right) \Rightarrow 1.008\text{ cfs}$$

For practical purposes the unit conversion coefficient difference of 0.8% can be ignored.

The RM formula is based on the assumption that for constant rainfall intensity, the peak discharge rate at a point will occur when the raindrop that falls at the most upstream point in the tributary drainage basin arrives at the point of interest.

Unlike the MRM (discussed in Section 3.4) or the NRCS hydrologic method (discussed in Section 4), the RM does not create hydrographs and therefore does not add separate subarea hydrographs at collection points. Instead, the RM develops peak discharges in the main line by increasing the T_c as flow travels downstream.

Characteristics of, or assumptions inherent to, the RM are listed below:

- The discharge flow rate resulting from any I is maximum when the I lasts as long as or longer than the T_c.

P: A4-3

Table 3-1
RUNOFF COEFFICIENTS FOR URBAN AREAS

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

*The values associated with 0% impervious may be used for direct calculation of the runoff coefficient as described in Section 3.1.2 (representing the pervious runoff coefficient, Cp, for the soil type), or for areas that will remain undisturbed in perpetuity. Justification must be given that the area will remain natural forever (e.g., the area is located in Cleveland National Forest).
DU/A = dwelling units per acre
NRCS = National Resources Conservation Service

p: A4.4.A

3.1.4 Time of Concentration

The Time of Concentration (T_c) is the time required for runoff to flow from the most remote part of the drainage area to the point of interest. The T_c is composed of two components: initial time of concentration (T_i) and travel time (T_t). Methods of computation for T_i and T_t are discussed below. 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." Guidelines for designating the initial subarea are provided within the discussion of computation of T_i. 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. For the RM, the T_c at any point within the drainage area is given by:

$$T_c = T_i + T_t$$

Methods of calculation differ for natural watersheds (nonurbanized) and for urban drainage systems. When analyzing storm drain systems, the designer must consider the possibility that an existing natural watershed may become urbanized during the useful life of the storm drain system. Future land uses must be used for T_c and runoff calculations, and can be determined from the local Community General Plan.

3.1.4.1 Initial Time of Concentration

The initial time of concentration is typically based on sheet flow at the upstream end of a drainage basin. The Overland Time of Flow (Figure 3-3) is approximated by an equation developed by the Federal Aviation Agency (FAA) for analyzing flow on runaways (FAA, 1970). The usual runway configuration consists of a crown, like most freeways, with sloping pavement that directs flow to either side of the runway. This type of flow is uniform in the direction perpendicular to the velocity and is very shallow. Since these depths are ¼ of an inch (more or less) in magnitude, the relative roughness is high. Some higher relative roughness values for overland flow are presented in Table 3.5 of the *HEC-1 Flood Hydrograph Package User's Manual* (USACE, 1990).

P: A4.4B

B.1.2 Step 1B - Tributary Area

Determine the total tributary area through evaluation of the drainage area delineations performed as outlined in Section 3. These areas will be analyzed in additional detail in Step 1C below.

B.1.3 Step 1C - Runoff Factor

Runoff factors (C) represent the ratio of storm water runoff over rainfall that is anticipated for a particular surface type. Impervious surfaces typically have high runoff factors (0.90) as nearly all rainfall is converted into runoff. Pervious surfaces typically have low runoff factors (0.10) as much of the rainfall is retained in natural surface features. Applicants should evaluate all of the surface coverages within a drainage area and assign runoff factors consistent with the values in Table B.1-1.

Table B.1-1: Runoff factors for surfaces draining to BMPs – Pollutant Control BMPs

Category	Surface Type	Runoff Factor (C)
Impervious Surfaces	Roofs, Concrete, Asphalt, Unit Pavers (grouted)	0.90
Semi-Pervious Surfaces	Decomposed Granite, Cobbles, Crushed Aggregate, Compacted soil (unpaved parking)	0.30
Engineered Pervious Surfaces	Green Roofs per SD-C Permeable Pavement per SD-D, Amended Soils per SD-F, Landscaped/Mulched Soils, Permeable Pavement per INF-3	0.10
Natural Pervious Surfaces	Type A Soil	0.10
	Type B Soil	0.14
	Type C Soil	0.23
	Type D Soil	0.30
Impoundments	Swimming pools, fountains, ponds, etc.	0.00
Dispersion Areas	Areas routed to or serving as a dispersion area per SD-B	See Dispersion Area Text Below

If a drainage area is comprised of more than one surface type, an area-weighted runoff factor must be calculated per the following equation where C represents the runoff coefficient and A represents the area of each surface.

$$C_{area-weighted} = \frac{\sum C_{surface\ 1} A_{surface\ 1} + C_{surface\ 2} A_{surface\ 2} + C_{surface\ x} A_{surface\ x}}{\sum A_{all\ surfaces}}$$

P: A4-5

The sheet flow that is predicted by the FAA equation is limited to conditions that are similar to runway topography. Some considerations that limit the extent to which the FAA equation applies are identified below:

- Urban Areas – This "runway type" runoff includes:
  1) Flat roofs, sloping at 1% ±
  2) Parking lots at the extreme upstream drainage basin boundary (at the "ridge" of a catchment area).
     Even a parking lot is limited in the amounts of sheet flow. Parked or moving vehicles would "break-up" the sheet flow, concentrating runoff into streams that are not characteristic of sheet flow.
  3) Driveways are constructed at the upstream end of catchment areas in some developments. However, if flow from a roof is directed to a driveway through a downspout or other conveyance mechanism, flow would be concentrated.
  4) Flat slopes are prone to meandering flow that tends to be disrupted by minor irregularities and obstructions. Maximum Overland Flow lengths are shorter for the flatter slopes (see Table 3-2).

- Rural or Natural Areas - The FAA equation is applicable to these conditions since (.5% to 10%) slopes that are uniform in width of flow have slow velocities consistent with the equation. Irregularities in terrain limit the length of application.
  1) Most hills and ridge lines have a relatively flat area near the drainage divide. However, with flat slopes of .5% ±, minor irregularities would cause flow to concentrate into streams.
  2) Parks, lawns and other vegetated areas would have slow velocities that are consistent with the FAA Equation.

The concepts related to the initial time of concentration were evaluated in a report entitled *Initial Time of Concentration, Analysis of Parameters* (Hill, 2002) that was reviewed by the Hydrology Manual Committee. The Report is available at San Diego County Department of Public Works, Flood Control Section and on the San Diego County Department of Public Works web page.

A- 4.6

Note that the Initial Time of Concentration should be reflective of the general land-use at the upstream end of a drainage basin. A single lot with an area of two or less acres does not have a significant effect where the drainage basin area is 20 to 600 acres.

Table 3-2 provides limits of the length (Maximum Length (L_M)) of sheet flow to be used in hydrology studies. Initial T_i values based on average C values for the Land Use Element are also included. These values can be used in planning and design applications as described below. Exceptions may be approved by the "Regulating Agency" when submitted with a detailed study.

Table 3-2

MAXIMUM OVERLAND FLOW LENGTH (L_M) & INITIAL TIME OF CONCENTRATION (T_i)

Element*	DU/	.5%		1%		2%		3%		5%		10%	
	Acre	L_M	T_i	L_M	T_i	L_M	T_i	L_M	T_i	L_M	T_i	L_M	T_i
Natural		50	13.2	70	12.5	85	10.9	100	10.3	100	8.7	100	6.9
LDR	1	50	12.2	70	11.5	85	10.0	100	9.5	100	8.0	100	6.4
LDR	2	50	11.3	70	10.5	85	9.2	100	8.8	100	7.4	100	5.8
LDR	2.9	50	10.7	70	10.0	85	8.8	95	8.1	100	7.0	100	5.6
MDR	4.3	50	10.2	70	9.6	80	8.1	95	7.8	100	6.7	100	5.3
MDR	7.3	50	9.2	65	8.4	80	7.4	95	7.0	100	6.0	100	4.8
MDR	10.9	50	8.7	65	7.9	80	6.9	90	6.4	100	5.7	100	4.5
MDR	14.5	50	8.2	65	7.4	80	6.5	90	6.0	100	5.4	100	4.3
HDR	24	50	6.7	65	6.1	75	5.1	90	4.9	95	4.3	100	3.5
HDR	43	50	5.3	65	4.7	75	4.0	85	3.8	95	3.4	100	2.7
N. Com		50	5.3	60	4.5	75	4.0	85	3.8	95	3.4	100	2.7
G. Com		50	4.7	60	4.1	75	3.6	85	3.4	90	2.9	100	2.4
O.P./Com		50	4.2	60	3.7	70	3.1	80	2.9	90	2.6	100	2.2
Limited I.		50	4.2	60	3.7	70	3.1	80	2.9	90	2.6	100	2.2
General I.		50	3.7	60	3.2	70	2.7	80	2.6	90	2.3	100	1.9

*See Table 3-1 for more detailed description

A4.7

3.1.4.1A Planning Considerations

The purpose of most hydrology studies is to develop flood flow values for areas that are not at the upstream end of the basin. Another example is the Master Plan, which is usually completed before the actual detailed design of lots, streets, etc. are accomplished. In these situations it is necessary that the initial time of concentration be determined without detailed information about flow patterns.

To provide guidance for the initial time of concentration design parameters, Table 3-2 includes the Land Use Elements and other variables related to the Time of Concentration. The table development included a review of the typical "layout" of the different Land Use Elements and related flow patterns and consideration of the extent of the sheet flow regimen, the effect of ponding, the significance to the drainage basin, downstream effects, etc.

3.1.4.1B Computation Criteria

(a) Developed Drainage Areas With Overland Flow - T_i may be obtained directly from the chart, "Rational Formula – Overland Time of Flow Nomograph," shown in Figure 3-3 or from Table 3-2. This chart is based on the Federal Aviation Agency (FAA) equation (FAA, 1970). For the short rain durations (<15 minutes) involved, intensities are high but the depth of flooding is limited and much of the runoff is stored temporarily in the overland flow and in shallow ponded areas. In developed areas, overland flow is limited to lengths given in Table 3-2. Beyond these distances, flow tends to become concentrated into streets, gutters, swales, ditches, etc.

(b) Natural Or Rural Watersheds – These areas usually have an initial subarea at the upstream end with sheet flow. The sheet flow length is limited to 50 to 100 feet as specified in Table 3-2. The Overland Time of Flow Nomograph, Figure 3-3, can be used to obtain T_i. The initial time of concentration can excessively affect the magnitude of flow further downstream in the drainage basin. For instance, variations in the initial time of concentration for an initial subarea of one acre can change the flow further downstream where the area is 400 acres by 100%. Therefore, the initial time of concentration is limited (see Table 3-2).

The Rational Method procedure included in the original Hydrology Manual (1971) and Design and Procedure Manual (1968) included a 10 minute value to be added to the initial time of concentration developed through the Kirpich Formula (see Figure 3-4) for a natural watershed. That procedure is superceded by the procedure above to use Table 3-2 or Figure 3-3 to determine T_i for the appropriate sheet flow length of the initial subarea. The values for natural watersheds given in Table 3-2 vary from 13 to 7 minutes, depending on slope. If the total length of the initial subarea is greater than the maximum length allowable based on Table 3-2, add the travel time based on the Kirpich formula for the remaining length of the initial subarea. Fig. 3-4

3.1.4.2 Travel Time

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. The T_t is computed by dividing the length of the flow path by the computed flow velocity. Since the velocity normally changes as a result of each change in flow rate or slope, such as at an inlet or grade break, the total T_t must be computed as the sum of the T_t's for each section of the flow path. Use Figure 3-6 to estimate time of travel for street gutter flow. Velocity in a channel can be estimated by using the nomograph shown in Figure 3-7 (Manning's Equation Nomograph).

A 4.9

(a) Natural Watersheds – This includes rural, ranch, and agricultural areas with natural channels. Obtain T_t directly from the Kirpich nomograph in Figure 3-4 or from the equation. This nomograph requires values for length and change in elevation along the effective slope line for the subarea. See Figure 3-5 for a representation of the effective slope line.

This nomograph is based on the Kirpich formula, which was developed with data from agricultural watersheds ranging from 1.25 to 112 acres in area, 350 to 4,000 feet in length, and 2.7 to 8.8% slope (Kirpich, 1940). A maximum length of 4,000 feet should be used for the subarea length. Typically, as the flow length increases, the depth of flow will increase, and therefore it is considered a concentration of flow at points beyond lengths listed in Figure 3-2. However, because the Kirpich formula has been shown to be applicable for watersheds up to 4,000 feet in length (Kirpich, 1940), a subarea may be designated with a length up to 4,000 feet provided the topography and slope of the natural channel are generally uniform.

Justification needs to be included with this calculation showing that the watershed will remain natural forever. Examples include areas located in the Multiple Species Conservation Plan (MSCP), areas designated as open space or rural in a community's General Plan, and Cleveland National Forest.

pipes

(b) Urban Watersheds - Flow through a closed conduit where no additional flow can enter the system during the travel, length, velocity and T_t are determined using the peak flow in the conduit. In cases where the conduit is not closed and additional flow from a contributing subarea is added to the total flow during travel (e.g., street flow in a gutter), calculation of velocity and T_t is performed using an assumed average flow based on the total area (including upstream subareas) contributing to the point of interest. The Manning equation is usually used to determine velocity. Discharges for small watersheds typically range from 2 to 3 cfs per acre, depending on land use, drainage area, and slope and rainfall intensity.

Note: The MRM should be used to calculate the peak discharge when there is a junction from independent subareas into the drainage system.

A-4.10

4.1.2.1 Hydrologic Soil Group

Soil properties influence the relationship between rainfall and runoff since soils have differing rates of infiltration. Based on infiltration rates, the NRCS has divided soils into four hydrologic soil groups.

<u>Group A</u>
Soils have high infiltration rate when thoroughly wetted; chiefly deep, well-drained to excessively drained sand, gravel, or both. Rate of water transmission is high; thus runoff potential is low.

<u>Group B</u>
Soils have moderate infiltration rate when thoroughly wetted; chiefly soils that are moderately deep to deep, moderately well drained to well drained, and moderately coarse textured. Rate of water transmission is moderate.

<u>Group C</u>
Soils have slow infiltration rate when thoroughly wetted; chiefly soils that have a layer impeding downward movement of water, or moderately fine to fine textured soils that have a slow infiltration rate. Rate of water transmission is slow.

<u>Group D</u>
Soils have very slow infiltration rate when thoroughly wetted; chiefly clays that have a high shrink-swell potential, soils that have a high permanent water table, soils that have a claypan or clay layer at or near the surface, or soils that are shallow over nearly impervious material. Rate of water transmission is very slow.

A list of soils throughout San Diego County and their hydrologic classification is located on the map in Appendix A. Soil Survey maps can be obtained from local NRCS offices for use in estimating soil type. The NRCS maps are also available at the County of San Diego DPWFCS. Consideration should be given to the effects of urbanization on the

 Rip Rap Sizing: P: A4-11

gradations. Where topography is steeper than desirable, drop structures may be used to maintain design velocities (see Section 7.11).

Table 7-3. Channel Bottom Riprap Protection

Design Velocity (ft/s)	Rock Gradation
6-10	No. 2 Backing
10-12	¼ ton
12-14	½ ton
14-16	1 ton
16-18	2 ton
> 18	Special Design

use this

7.6.2 Roughness Coefficients

The Manning roughness coefficient (n) for hydraulic computations shall be estimated for loose rock riprap using the Manning-Strickler equation (Equation 7-5). Equation 7-5 (Chang, 1992) does not apply to grouted rock riprap or to very shallow flow. Table 7-4 provides Manning roughness coefficients for standard rock riprap classifications based on the Manning-Strickler method.

Equation 7-5. Manning-Strickler Equation

$$n=0.0395d_{50}^{1/6}$$

where:

n = Manning roughness coefficient (dimensionless)

d_{50} = median stone diameter (feet)

P: A4.12

where ...

n = Manning roughness coefficient (dimensionless); and

d_{50} = median stone diameter (feet).

Table 5-4 Standard Rock Riprap Gradations

##Rock Gradation [a]	Median Stone Weight (W_{50}) [c]	Median Stone Diameter (d_{50}) [d]	Manning n (Ungrouted) [e]
No. 3 Backing	5 lb	0.4 ft	0.034
No. 2 Backing	25 lb	0.7 ft	0.037
No. 1 Backing [b]	75 lb	1.0 ft	0.039
Light	200 lb	1.3 ft	0.041
¼ Ton	500 lb	1.8 ft	0.044
½ Ton	1000 lb	2.3 ft	0.045
1 Ton	2000 lb	2.9 ft	0.047
2 Ton	4000 lb	3.6 ft	0.049

(a) Except for 2 ton rock, classification is based upon Caltrans Method B Placement, which allows dumping of the rock and spreading by mechanical equipment. Local surface irregularities shall not vary from the planned grade by more than 1 foot, measured perpendicular to the slope. Two-ton rock requires special placement, see Caltrans (2002) or Greenbook for more information. (b) No. 1 Backing has same gradation as Facing Riprap. (c) per Caltrans (2002). (d) Assumes specific weight of 165 lb/ft3. The designer shall take care to apply a unit weight that is applicable to the type of riprap specified for the project, and adjust their calculations when necessary. (e) Based on Manning-Strickler relationship (Chang, 1988).

Where hydraulic radius is less than or equal to two times the maximum rock size, the roughness coefficient will be greater than indicated by Equation 5-5. In these cases, the design engineer shall use the method outlined in Section 5.7.17 to calculate the roughness of the channel. Appendix A (Table A-3) provides recommended Manning roughness coefficient (n) for grouted riprap applications. A 20% roughness coefficient reduction (N grouted = 0.80 N ungrouted) for grouted rip-rap shall be required for velocity-based design for energy dissipation/scour minimization measures applications. For channel capacity design, the roughness coefficients in Table 5-4 and Appendix A shall be used.

5.7.3 Low Flow and Trickle Channels

Riprap-lined channels conveying a 100-year peak runoff of 20 cfs or less do not require trickle channels. The design engineer shall evaluate the factors such as drainage slope, flow velocity, soil type, and upstream impervious area, and specify a trickle channel when needed based on their engineering judgment. Low-flow channels shall be designed in accordance with Section 5.5.3.2.

5.7.4 Bottom Width

The selection of the over-all channel bottom width shall consider factors such as ultimate conveyance requirements, constructability, channel stability, and maintenance.

5.7.5 Freeboard and Flow Depth

Riprap-lined channels shall meet the minimum freeboard requirements outlined in Section 5.3.7. Excessive depths and high velocities shall be avoided whenever practicable to maintain public safety. Section 5.3.9 discusses access and safety for open channels, including thresholds for flow depth and velocity.

5.7.6 Side Slopes

The side slopes of riprap-lined channel shall not ordinarily be steeper than 2H:1V, except in cases where an embankment stability analysis can justify a steeper side slope. The stability analysis should be completed in consultation with a soils engineer, and consider such factors such as: soil

Table 5-6 Minimum Riprap Layer Thickness

Rock Gradation	Minimum Layer Thickness (ft)
Placement Method A	
Rock Gradation	**Minimum Layer Thickness (ft)**
8 ton	8.50
4 ton	6.80
2 ton	5.40
1 ton	4.30
½ ton	3.40
Placement Method B	
Rock Gradation	**Minimum Layer Thickness (ft)**
1 ton	5.40
½ ton	4.30
¼ ton	3.30
Light	2.50
Facing	1.80
Backing No. 1	1.80
Backing No. 2	1.25
Backing No. 3	0.75

Note: used Rip Rap SDRSD D40

Minimum layer thickness for Placement Method A is 1.50d_{50} and 1.875d_{50} for Placement Method B. These thickness calculations assume a specific weight of 165 lb/ft^3. The designer shall take care to apply a unit weight that is applicable to the type of riprap specified for the project, and adjust their calculations when necessary.

underwater, the riprap thickness shall be increased by at least 50 percent. The total thickness of a riprap installation is the sum of individual layer thicknesses (see Section 5.7.11).

5.7.11 Bedding Requirements

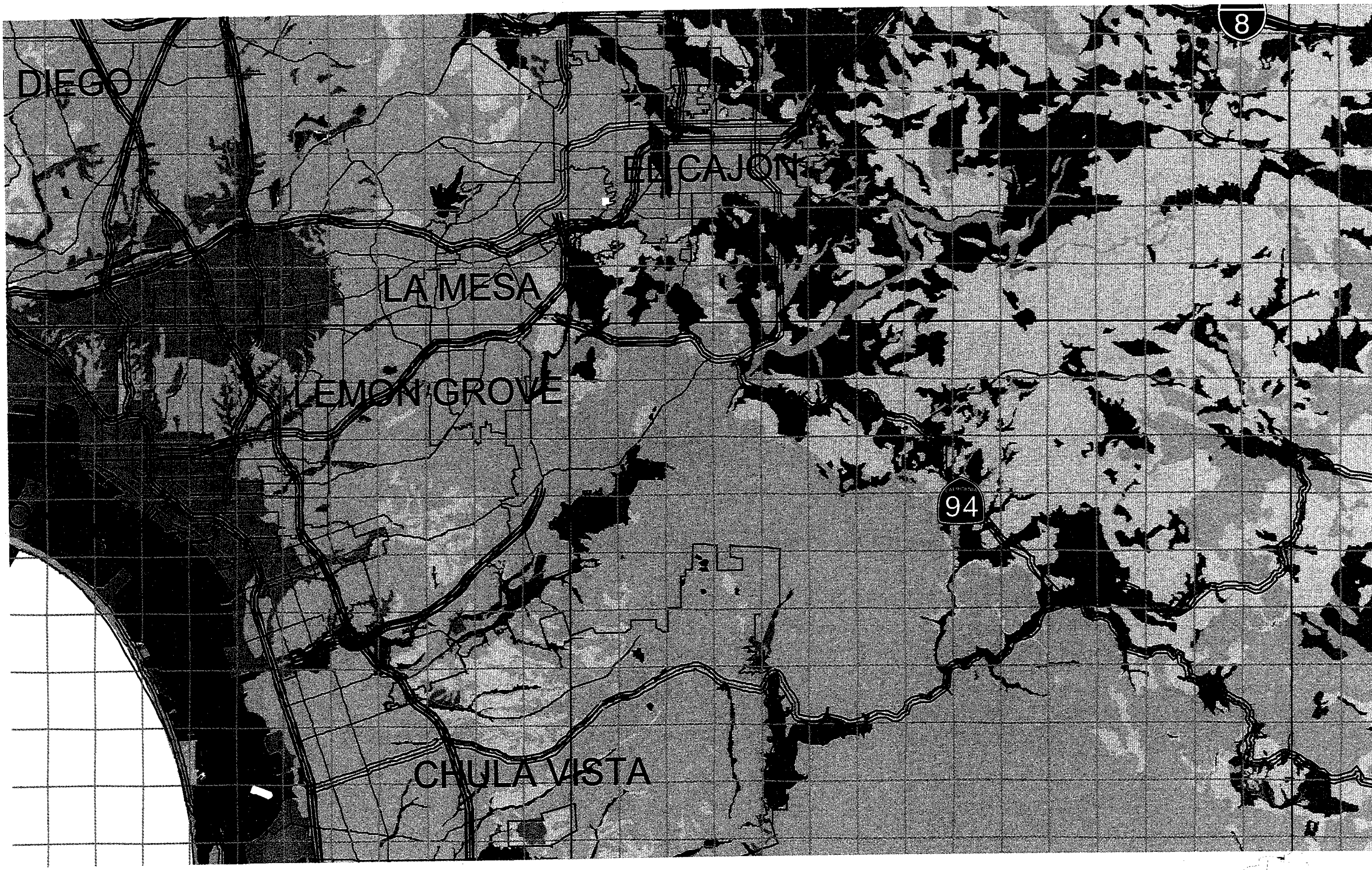
The long-term stability of riprap erosion protection is strongly influenced by proper bedding conditions. Properly designed bedding provides a buffer of intermediate-sized material between the channel bed and the riprap to prevent movement of soil particles through the voids in the riprap. Three types of bedding are in common use: generic single-layer granular bedding, multiple-layer granular bedding, and filter fabric.

Standard riprap installations include an outside layer, one or more inner layers, a backing layer, and filter fabric. Section 5.7.9 describes the determination of gradation of the outside rock layer. The composition of the inner layer(s), backing layer, and filter fabric are design to be progressively smaller to prevent migration of material through voids of the layers. Table 5-7 summarizes the appropriate layers for standard riprap installations. Alternate designs for riprap bedding are acceptable when accompanied by appropriate design calculations.

Filter fabric can provide adequate bedding for channel linings along uniform mild sloping channels where leaching forces are primarily perpendicular to the fabric. The design engineer shall use care in specifying using filter fabrics, and shall note appropriate construction methods on their plans and specifications. Some of the design considerations and limitations of filter fabric include:

A 4.14

Soil Type per County of San Diego Hydrology Manual

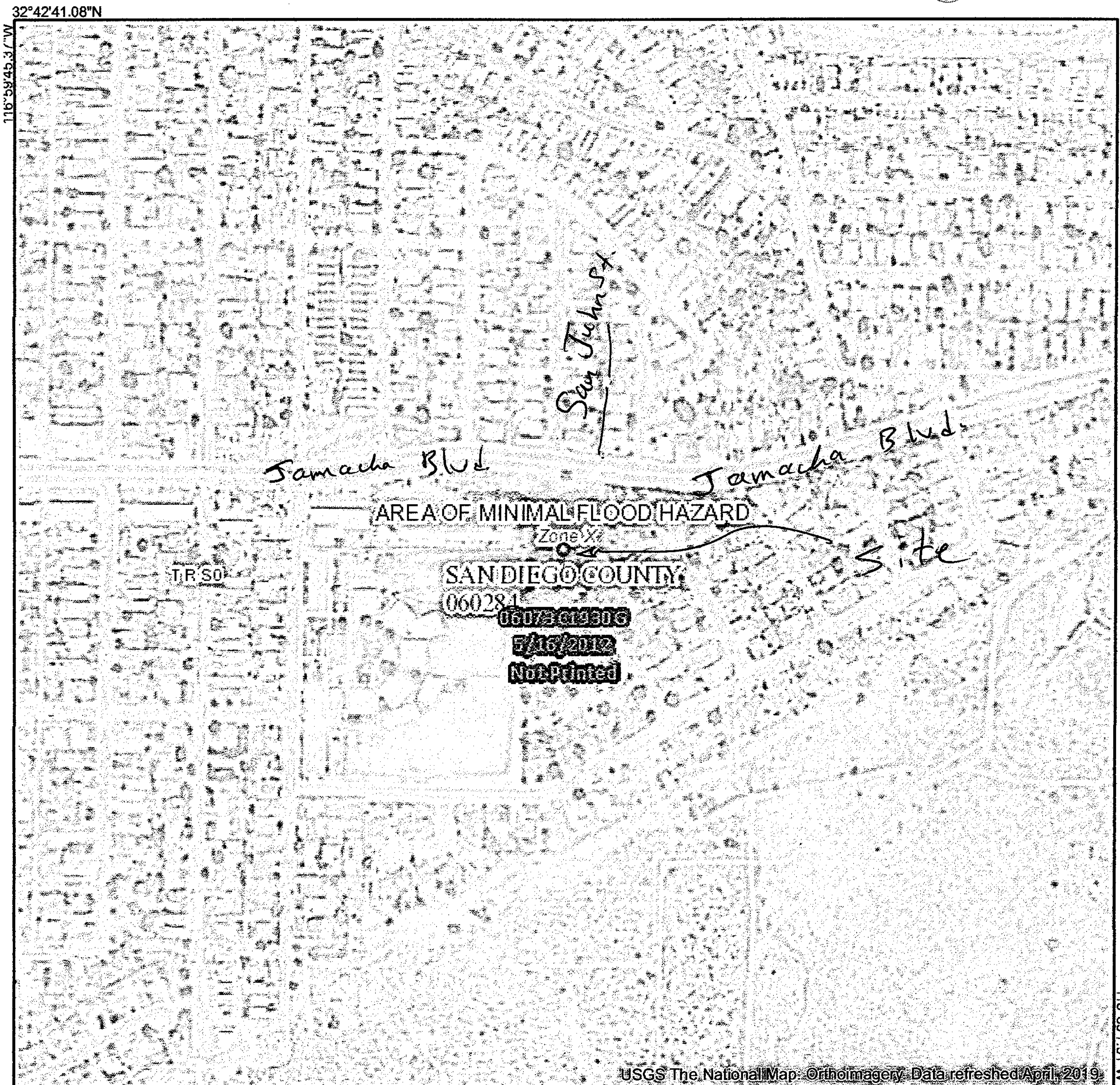


Appendix A-5

Flood Map (by FEMA)

National Flood Hazard Layer FIRMette

FEMA



Feet 0 250 500 1,000 1,500 2,000

1:6,000

Legend

SEE FIS REPORT FOR DETAILED LEGEND AND INDEX MAP FOR FIRM PANEL LAYOUT

SPECIAL FLOOD HAZARD AREAS
- Without Base Flood Elevation (BFE) *Zone A, V, A99*
- With BFE or Depth *Zone AE, AO, AH, VE, AR*
- Regulatory Floodway

OTHER AREAS OF FLOOD HAZARD
- 0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
- Future Conditions 1% Annual Chance Flood Hazard *Zone X*
- Area with Reduced Flood Risk due to Levee. See Notes. *Zone X*
- Area with Flood Risk due to Levee *Zone D*

OTHER AREAS
- NO SCREEN Area of Minimal Flood Hazard *Zone X*
- Effective LOMRs
- Area of Undetermined Flood Hazard *Zone D*

GENERAL STRUCTURES
- Channel, Culvert, or Storm Sewer
- Levee, Dike, or Floodwall

OTHER FEATURES
- 20.2 / 17.5 Cross Sections with 1% Annual Chance Water Surface Elevation
- Coastal Transect
- 513 Base Flood Elevation Line (BFE)
- Limit of Study
- Jurisdiction Boundary
- Coastal Transect Baseline
- Profile Baseline
- Hydrographic Feature

MAP PANELS
- Digital Data Available
- No Digital Data Available
- Unmapped

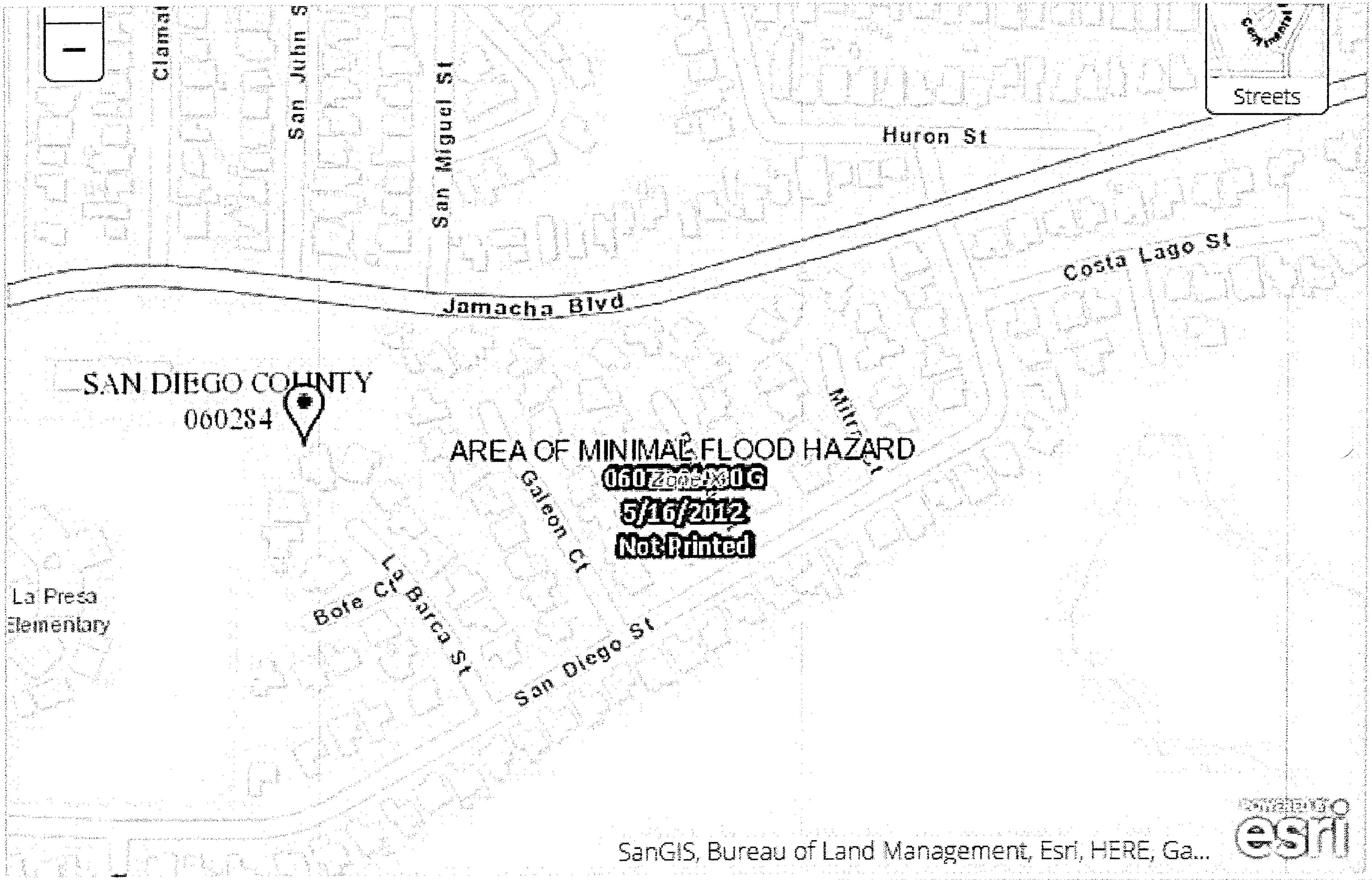
N

The pin displayed on the map is an approximate point selected by the user and does not represen an authoritative property location.

This map complies with FEMA's standards for the use of digital flood maps if it is not void as described below. The basemap shown complies with FEMA's basemap accuracy standards

The flood hazard information is derived directly from the authoritative NFHL web services provided by FEMA. This map was exported on 3/8/2020 at 5:02:07 AM and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time.

This map image is void if the one or more of the following map elements do not appear: basemap imagery, flood zone labels, legend, scale bar, map creation date, community identifiers, FIRM panel number, and FIRM effective date. Map images for unmapped and unmodernized areas cannot be used for regulatory purposes.


San Juan S
San Miguel St
Huron St
Streets
Jamacha Blvd
Costa Lago St
SAN DIEGO COUNTY
060284
AREA OF MINIMAL FLOOD HAZARD
5/16/2012
Not Printed
Galeon Ct
La Presa
Elementary
Bole Ct
La Barca St
San Diego St
SanGIS, Bureau of Land Management, Esri, HERE, Ga...
esri

Show all

Exhibit A:

Pre- Development Drainage Map

And

Exhibit B:

Post- Development Drainage Map

Legends:

- - - - - Exist. Counter line

⟶ Direction of flow of Stormwater.

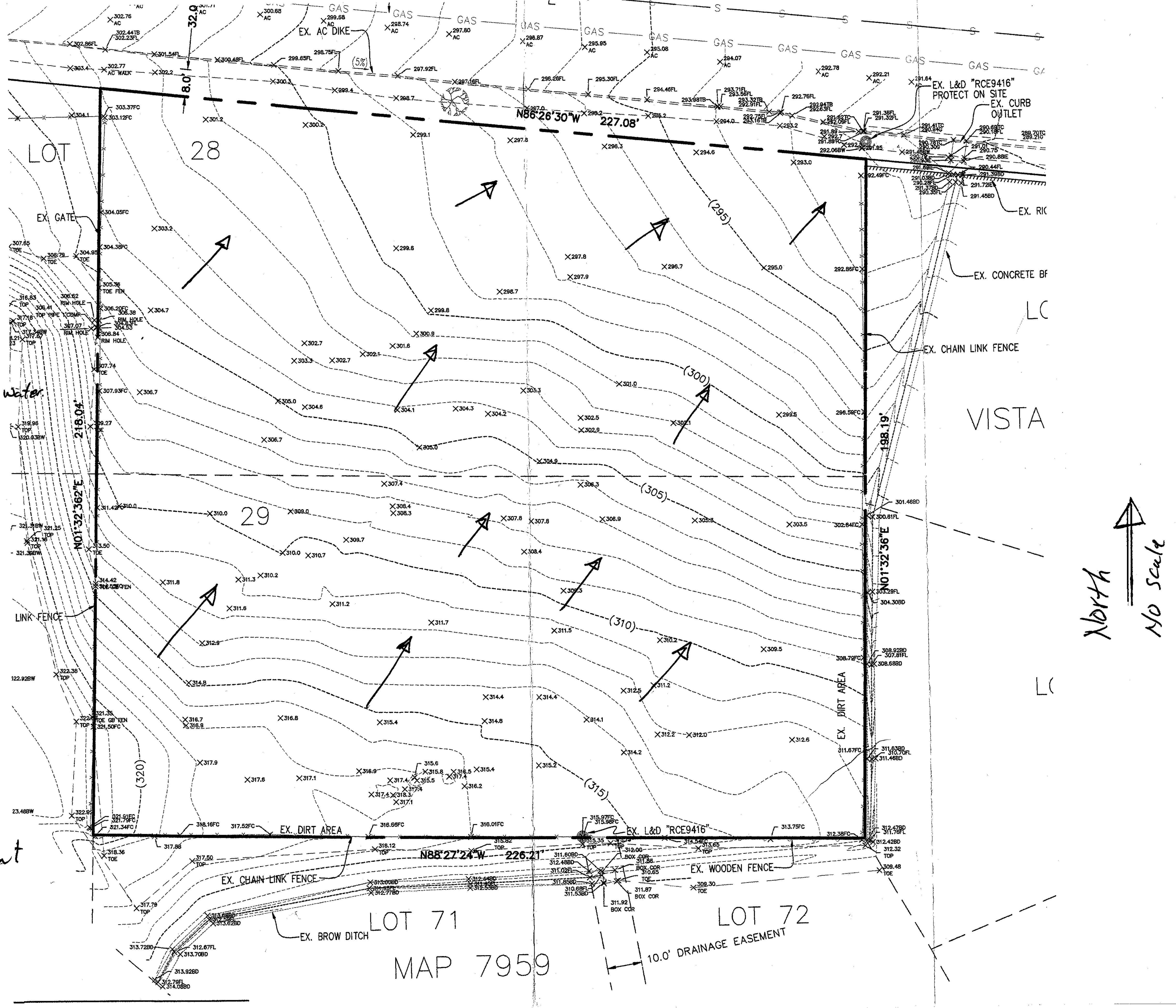
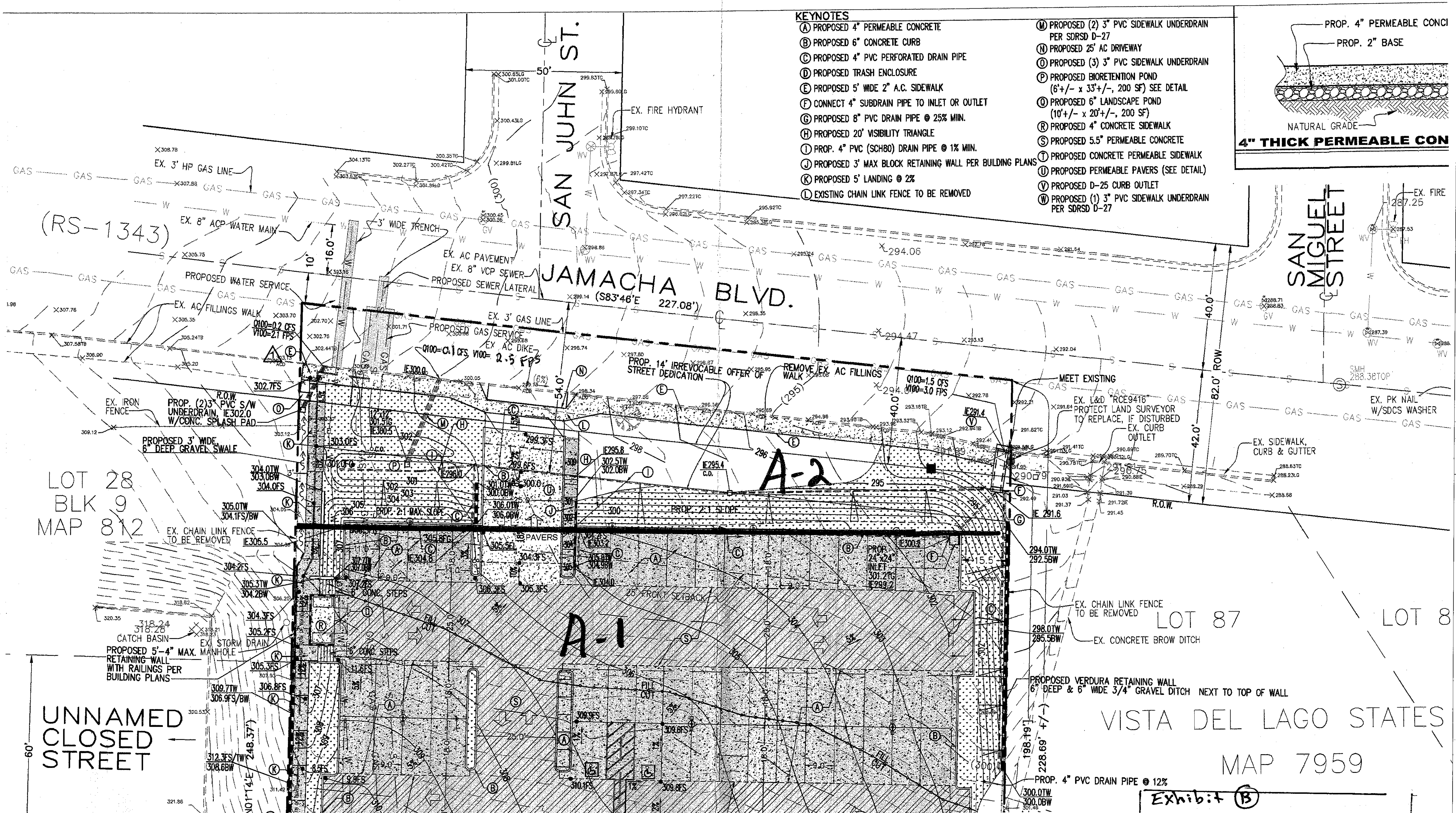


Exhibit - A

Pre-Development Drainage Map



KEYNOTES

- (A) PROPOSED 4" PERMEABLE CONCRETE
- (B) PROPOSED 6" CONCRETE CURB
- (C) PROPOSED 4" PVC PERFORATED DRAIN PIPE
- (D) PROPOSED TRASH ENCLOSURE
- (E) PROPOSED 5' WIDE 2" A.C. SIDEWALK
- (F) CONNECT 4" SUBDRAIN PIPE TO INLET OR OUTLET
- (G) PROPOSED 8" PVC DRAIN PIPE @ 25% MIIN.
- (H) PROPOSED 20' VISIBILITY TRIANGLE
- (I) PROP. 4" PVC (SCH80) DRAIN PIPE @ 1% MIN.
- (J) PROPOSED 3' MAX BLOCK RETAINING WALL PER BUILDING PLANS
- (K) PROPOSED 5' LANDING @ 2%
- (L) EXISTING CHAIN LINK FENCE TO BE REMOVED
- (M) PROPOSED (2) 3" PVC SIDEWALK UNDERDRAIN PER SDRSD D-27
- (N) PROPOSED 25' AC DRIVEWAY
- (O) PROPOSED (3) 3" PVC SIDEWALK UNDERDRAIN
- (P) PROPOSED BIORETENTION POND (6'+/- x 33'+/-, 200 SF) SEE DETAIL
- (Q) PROPOSED 6" LANDSCAPE POND (10'+/- x 20'+/-, 200 SF)
- (R) PROPOSED 4" CONCRETE SIDEWALK
- (S) PROPOSED 5.5" PERMEABLE CONCRETE
- (T) PROPOSED CONCRETE PERMEABLE SIDEWALK
- (U) PROPOSED PERMEABLE PAVERS (SEE DETAIL)
- (V) PROPOSED D-25 CURB OUTLET
- (W) PROPOSED (1) 3" PVC SIDEWALK UNDERDRAIN PER SDRSD D-27

Exhibit B

Post Development Page 1-of 2

Date: 9-19-2022

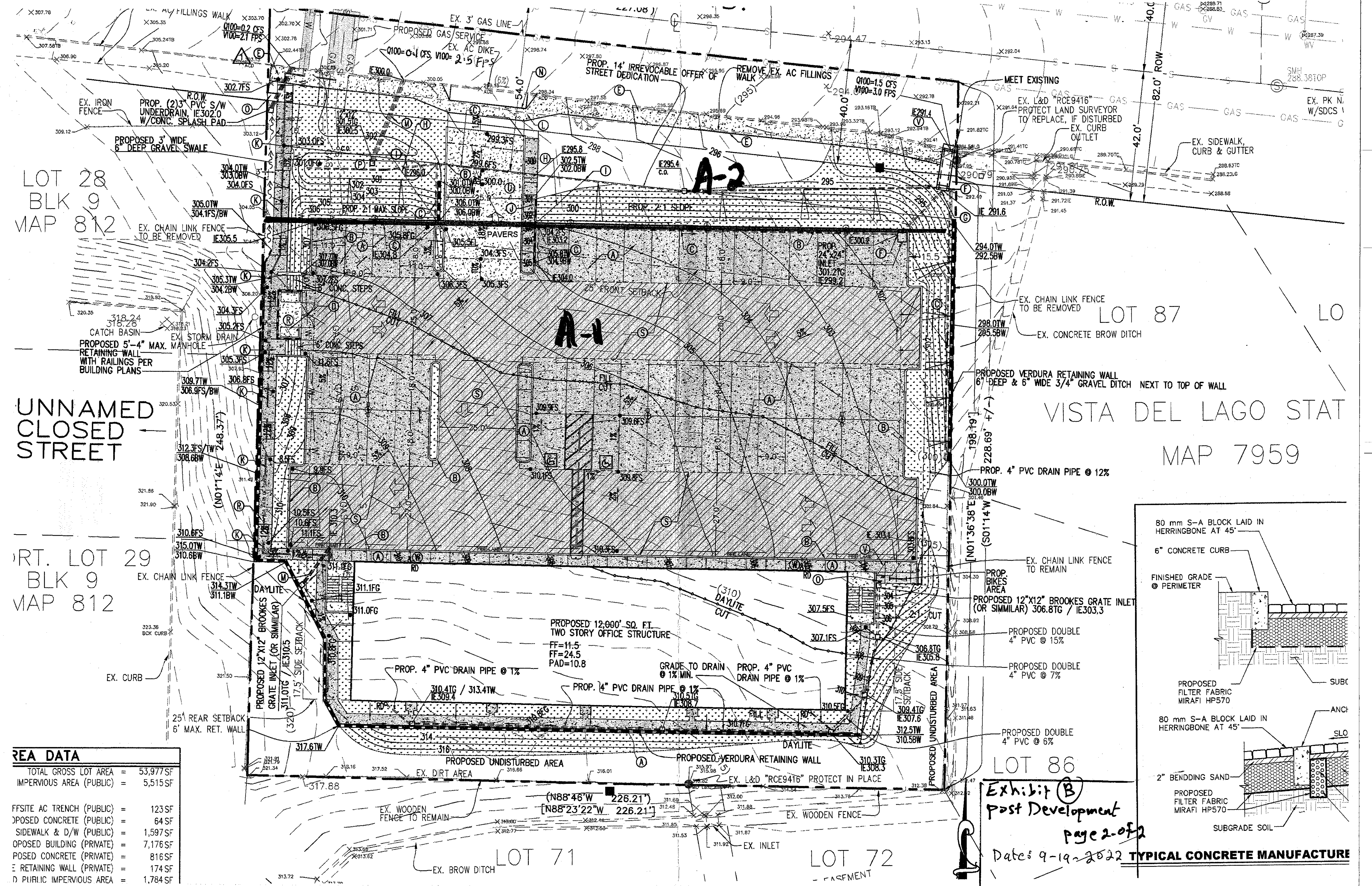


Exhibit (B)
Post Development
page 2 of 2
Dates: 9-19-2022
A-1
A-2
LOT 28
BLK 9
MAP 812
UNNAMED
CLOSED
STREET
RT. LOT 29
BLK 9
MAP 812
LOT 87
VISTA DEL LAGO STAT
MAP 7959
LOT 86
LOT 71
LOT 72
EX. 3' GAS LINE
PROPOSED GAS SERVICE
EX. AC DIKE
Q100=0.1 CFS V100=2.5 FPS
Q100=0.2 CFS V100=2.1 FPS
Q100=1.5 CFS V100=3.0 FPS
PROP. 14' IRREVOCABLE OFFER OF STREET DEDICATION
REMOVE EX. AC FILLINGS WALK
MEET EXISTING
EX. L&D "RCE9416" PROTECT LAND SURVEYOR TO REPLACE, IF DISTURBED
EX. CURB OUTLET
EX. SIDEWALK, CURB & GUTTER
EX. PK N. W/SDCS
ROW
82.0'
42.0'
40.0'
54.0'
EX. IRON FENCE
PROP. (2)3" PVC S/W UNDERDRAIN, IE302.0 W/CONC. SPLASH PAD
PROPOSED 3' WIDE, 6" DEEP GRAVEL SWALE
PROP. 2:1 MAX SLOPE
PROP. 2:1 SLOPE
PAVERS
EX. CHAIN LINK FENCE TO BE REMOVED
25' FRONT SETBACK
6" CONC. STEPS
CATCH BASIN
EX. STORM DRAIN MANHOLE
PROPOSED 5'-4" MAX. RETAINING WALL WITH RAILINGS PER BUILDING PLANS
PROP. 24"x24" INLET 301.2TG IE299.2
EX. CONCRETE BROW DITCH
PROPOSED VERDURA RETAINING WALL 6" DEEP & 6" WIDE 3/4" GRAVEL DITCH NEXT TO TOP OF WALL
PROP. 4" PVC DRAIN PIPE @ 12%
EX. CHAIN LINK FENCE TO REMAIN
PROP. BIKES AREA
PROPOSED 12"X12" BROOKES GRATE INLET (OR SIMMILAR) 306.8TG / IE303.3
PROPOSED DOUBLE 4" PVC @ 15%
PROPOSED DOUBLE 4" PVC @ 7%
PROPOSED DOUBLE 4" PVC @ 6%
PROPOSED 12,000' SQ. FT. TWO STORY OFFICE STRUCTURE
FF=11.5
FF=24.5
PAD=10.8
DAYLITE CUT
GRADE TO DRAIN @ 1% MIN.
PROP. 4" PVC DRAIN PIPE @ 1%
PROPOSED 12"X12" BROOKES GRATE INLET (OR SIMMILAR) 311.0TG / IE310.5
17.5' SIDE SETBACK
25' REAR SETBACK
6' MAX. RET. WALL
EX. CHAIN LINK FENCE
EX. CURB
PROPOSED UNDISTURBED AREA
PROPOSED VERDURA RETAINING WALL
EX. L&D "RCE9416" PROTECT IN PLACE
EX. DIRT AREA
EX. WOODEN FENCE TO REMAIN
EX. WOODEN FENCE
EX. INLET
EX. BROW DITCH
EASMENT
(N88°46'W 226.21')
[N88°23'22"W 226.21']
(N01°14'E 248.37')
[N01°36'38"E 198.19']
(S01°14'W 228.69' +/-)
AREA DATA
TOTAL GROSS LOT AREA = 53,977 SF
IMPERVIOUS AREA (PUBLIC) = 5,515 SF
FFSITE AC TRENCH (PUBLIC) = 123 SF
OPOSED CONCRETE (PUBLIC) = 64 SF
SIDEWALK & D/W (PUBLIC) = 1,597 SF
OPOSED BUILDING (PRIVATE) = 7,176 SF
POSED CONCRETE (PRIVATE) = 816 SF
E RETAINING WALL (PRIVATE) = 174 SF
D PUBLIC IMPERVIOUS AREA = 1,784 SF
80 mm S-A BLOCK LAID IN HERRINGBONE AT 45°
6" CONCRETE CURB
FINISHED GRADE @ PERIMETER
PROPOSED FILTER FABRIC MIRAFI HP570
2" BENDDING SAND
SUBGRADE SOIL
TYPICAL CONCRETE MANUFACTURE