FLOOD INSURANCE STUDY

VOLUME 2 OF 4

LOS ANGELES COUNTY, CALIFORNIA

AND INCORPORATED AREAS

Community Name	Community Number	Community Name	Community Number	Community Name	Community Number	Community Name	Community Number
LOS ANGELES COUNTY, UNINCORPORATED AREAS	065043	DIAMOND BAR, CITY OF	060741	LAWNDALE, CITY OF*	060134	SAN DIMAS, CITY OF	060154
AGOURA HILLS, CITY OF	065072	DOWNEY, CITY OF	060645	LOMITA, CITY OF*	060135	SAN FERNANDO, CITY OF*	060628
ALHAMBRA, CITY OF*	060095	DUARTE, CITY OF	065026	LONG BEACH, CITY OF	060136	SAN GABRIEL, CITY OF*	065055
ARCADIA, CITY OF	065014	EL MONTE, CITY OF*	060658	LOS ANGELES, CITY OF	060137	SAN MARINO, CITY OF*	065057
ARTESIA, CITY OF*	060097	EL SEGUNDO, CITY OF	060118	LYNWOOD, CITY OF	060635	SANTA CLARITA, CITY OF	060729
AVALON, CITY OF	060098	GARDENA, CITY OF	060119	MALIBU, CITY OF	060745	SANTA FE SPRINGS, CITY OF	060158
AZUSA, CITY OF	065015	GLENDALE, CITY OF	065030	MANHATTAN BEACH, CITY OF	060138	SANTA MONICA, CITY OF	060159
BALDWIN PARK, CITY OF*	060100	GLENDORA, CITY OF	065031	MAYWOOD, CITY OF*	060651	SIERRA MADRE, CITY OF	065059
BELL GARDENS, CITY OF	060656	HAWAIIAN GARDENS, CITY OF*	065032	MONROVIA, CITY OF	065046	SIGNAL HILL, CITY OF*	060161
BELL, CITY OF*	060101	HAWTHORNE, CITY OF*	060123	MONTEBELLO, CITY OF	060141	SOUTH EL MONTE, CITY OF*	060162
BELLFLOWER, CITY OF	060102	HERMOSA BEACH, CITY OF	060124	MONTEREY PARK, CITY OF*	065047	SOUTH GATE, CITY OF	060163
BEVERLY HILLS, CITY OF*	060655	HIDDEN HILLS, CITY OF	060125	NORWALK, CITY OF	060652	SOUTH PASADENA, CITY OF*	065061
BRADBURY, CITY OF	065017	HUNTINGTON PARK, CITY OF*	060126	PALMDALE, CITY OF	060144	TEMPLE CITY, CITY OF	060653
BURBANK, CITY OF	065018	INDUSTRY, CITY OF	065035	PALOS VERDES ESTATES, CITY OF	060145	TORRANCE, CITY OF	060165
CALABASAS, CITY OF	060749	INGLEWOOD, CITY OF*	065036	PARAMOUNT, CITY OF	065049	VERNON, CITY OF*	060166
CARSON, CITY OF	060107	IRWINDALE, CITY OF*	060129	PASADENA, CITY OF	065050	WALNUT, CITY OF	065069
CERRITOS, CITY OF	060108	LA CANADA FLINTRIDGE, CITY OF	060669	PICO RIVERA, CITY OF	060148	WEST COVINA, CITY OF	060666
CLAREMONT, CITY OF	060109	LA HABRA HEIGHTS, CITY OF	060701	POMONA, CITY OF	060149	WEST HOLLYWOOD, CITY OF	060720
COMMERCE, CITY OF	060110	LA MIRADA, CITY OF	060131	RANCHO PALOS VERDES, CITY OF	060464	WESTLAKE VILLAGE, CITY OF	060744
COMPTON, CITY OF	060111	LA PUENTE, CITY OF*	065039	REDONDO BEACH, CITY OF	060150	WHITTIER, CITY OF	060169
COVINA, CITY OF	065024	LA VERNE, CITY OF	060133	ROLLING HILLS ESTATES, CITY OF*	065054		
CUDAHY, CITY OF	060657	LAKEWOOD, CITY OF	060130	ROLLING HILLS, CITY OF	060151		
CULVER CITY, CITY OF	060114	LANCASTER, CITY OF	060672	ROSEMEAD, CITY OF	060153		

*Non-floodprone community

REVISED
January 6, 2016

Federal Emergency Management Agency

FLOOD INSURANCE STUDY NUMBER
06037CV002B

NOTICE TO
FLOOD INSURANCE STUDY USERS

Communities participating in the National Flood Insurance Program have established repositories of flood hazard data for floodplain management and flood insurance purposes. This Flood Insurance Study (FIS) may not contain all data available within the repository. It is advisable to contact the community repository for any additional data.

Part or all of this FIS may be revised and republished at any time. In addition, part of this FIS may be revised by the Letter of Map Revision process, which does not involve republication or redistribution of the FIS. It is, therefore, the responsibility of the user to consult with community officials and to check the community repository to obtain the most current FIS components.

Initial Countywide FIS Effective Date: September 26, 2008

Revised Countywide FIS Date: January 6, 2016

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3.0 ENGINEERING METHODS

For the flooding sources studied by detailed methods in the community, standard hydrologic and hydraulic study methods were used to determine the flood-hazard data required for this study. Flood events of a magnitude that is expected to be equaled or exceeded once on the average during any 10-, 50-, 100-, or 500-year period (recurrence interval) have been selected as having special significance for floodplain management and for flood insurance rates. These events, commonly termed the 10-, 50-, 100-, and 500-year floods, have a 10-, 2-, 1-, and 0.2-percent chance, respectively, of being equaled or exceeded during any year. Although the recurrence interval represents the long-term, average period between floods of a specific magnitude, rare floods could occur at short intervals or even within the same year. The risk of experiencing a rare flood increases when periods greater than 1 year are considered. For example, the risk of having a flood that equals or exceeds the 1-percent-annual-chance flood in any 50-year period is approximately 40 percent (4 in 10); for any 90-year period, the risk increases to approximately 60 percent (6 in 10). The analyses reported herein reflect flooding potentials based on conditions existing in the community at the time of completion of this study. Maps and flood elevations will be amended periodically to reflect future changes.

3.1 Hydrologic Analyses

Hydrologic analyses were carried out to establish the peak discharge-frequency relationships for the flooding sources studied in detail affecting the County. A summary of peak discharge-drainage area relationships for streams studied by detailed methods is shown in Table 6, "Summary of Peak Discharges." A summary of breakout discharges is shown in Table 7, “Summary of Breakout Discharges.” Elevations for floods of the selected recurrence intervals on the Pacific Ocean are shown in Table 8, “Summary of Elevations.”

3.1.1 Methods for Flooding Sources with New or Revised Analyses in Current Study

Discharges for the Las Virgenes Creek Study were computed for the 10-, 2-, 1-, and 0.2-percent-annual-chance floods in the USACE HEC-HMS ver. 3.5 flood hydrograph program. Basin hydrographs were generated using the Los Angeles County Design Storm Unit Hyetograph and S-graphs developed for Los Angeles County by USACE and routed downstream using modified Puls routing. Land use was collected from the National Land Cover Dataset website and soil information was collected from the NRCS Soil Data Mart Website. Precipitation data and 24-hour frequency depths at 6 gages were provided by Los Angeles County Department of Public Works and incorporated into the HEC-HMS model using Thiessen weights.

Peak discharges for Unnamed Stream Main Reach, Unnamed Stream Tributary 1, and Unnamed Stream Tributary 2 were computed for the 10-, 2-, 1-, and 0.2-percent-annual-chance storm events using regional regression equations from the United States Geological Survey (USGS) contained in the report entitled “The Nationwide Summary of U.S. Geological Survey Regression Equation for Estimating Magnitude and Frequency of Floods for Ungaged Sites dated 1993 (Water Resources Investigations Report 94-4002”. The study area’s urbanization and location within the South Coast Region determined the regression equations used.

Table 6: Summary of Peak Discharges

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second)			
		10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
3,500 feet Northeast of the Intersection of Via Montana and Country Club Drive	0.7	--	--	600	--
At the Intersection of Alameda Avenue and Main Street	1.2	--	--	750	--
At the Intersection of Chestnut and Lake Streets	1.3	--	--	670	--
AMARGOSA CREEK					
At Outlet of Ritter Ranch Detention Pond	23.8	--	--	1,856	--
At Vineyard Ranch	26.5	--	--	2,063	
At Elizabeth Lake Ford Crossing	28.6	--	--	2,288	--
At 25th Street West Bridge	30.0	--	--	2,341	--
At 10th Street West	32.0	--	--	2,364	--
AMARGOSA CREEK TRIBUTARY					
Intersection of Avenue L and 3rd Street East	2.4	150	420	560	1,000
Intersection of Avenue I and Spearman Avenue	7.2	310	900	1,220	2,400
Avenue M and Valleyline Drive	1.8	120	340	460	850

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second)			
		10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
ANAVERDE CREEK					
1.85 Miles Downstream of California Aqueduct	15.66	--	--	3,630	--
1.47 Miles Downstream of California Aqueduct	12.79	--	--	3,200	--
Antelope Freeway	16.35	--	--	3,730	--
1.85 miles Downstream of California Aqueduct	15.66	--	--	3,630	--
1.47 miles Downstream of California Aqueduct	12.79	--	--	3,200	--
0.75 miles Downstream of California Aqueduct	11.79	--	--	3,050	--
California Aqueduct	8.25	--	--	2,440	--
NAVERDE CREEK TRIBUTARY					
Division Street between Avenue P and Avenue P-8	1.4	300	1,100	1,600	3,000
ANTELOPE VALLEY					
Amargosa Creek at 90th Street West	6.9	580	2,000	3,100	4,500
Amargosa Creek Approximately Midway between 20th Street West and 10th Street West	32.7	1,800	3,300	5,000	10,100
West of Antelope Valley Freeway North of Avenue H	147	2,000	5,600	8,400	18,000

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second)			
		10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
East of Antelope Valley Freeway North of Avenue H	206	3,000	9,000	13,000	30,000
Avenue F at Sierra Highway	206	3,000	9,000	13,000	30,000
Anaverde Creek East of Antelope Valley Freeway	16	700	2,100	3,000	6,400
West of Sierra Highway at Avenue P-8	19	700	2,100	3,100	6,600
West of 136th Street East at Avenue W-8	2.4	440	1,500	1,900	3,900
165th Street East Approximately 4,000 feet South of Pearblossom Highway	1.0	370	1,300	1,600	3,100
3,000 feet East of 165th Street East and 4.000 feet South of Pearblossom Highway	7.3	500	1,700	2,300	4,700
Acton Canyon Road, Escondido Canyon Road, and Crown Valley Road	20.3	--	--	3,421	6,052
Acton Canyon at Intersection of Crown Valley Road and Acton Avenue	20.3	--	--	3,421	6,052
Agua Dulce Canyon Approximately 5,600 feet Upstream of Darling Road	10.3	--	--	3,509	6,360
Agua Dulce Canyon Approximately 800 feet Upstream of Escondido Canyon Road	14.3	--	--	4,401	7,977

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second) 10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
Sand Canyon Approximately 800 feet Upstream of Placerita Canyon Road	6.4	--	--	4,371	5,961
Sand Canyon Approximately 2,900 feet Downstream of Placerita Canyon Road	7.3	--	--	4,908	6,693
Sand Canyon Approximately 250 feet Downstream of Iron Canyon Confluence	10.1	--	--	6,372	8,689
Iron Canyon Approximately 2,000 feet Upstream of Sand Canyon Road	2.8	--	--	2,078	2,833
Oak Springs Canyon Approximately 100 feet Upstream of Union Pacific Railroad (former Southern Pacific Railroad)	5.7	--	--	2,703	4,054
At intersection of Sixth Street and Quincy Avenue	1.0	271	598	763	1,194
AVALON CANYON					
At Cross Section A	3.65	859	1,895	2,419	3,785
At Cross Section G	1.83	440	971	1,239	1,938
BALLONA CREEK CHANNEL					
At intersection of Adams Boulevard and Genesee Avenue	16.7	2,100	4,700	6,000	9,400

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second) 10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
BIG ROCK WASH					
At mouth, Southwest	23.0	--	--	15,000	---
CHATSWORTH AREA					
Vicinity of Santa Susanna Pass Road and Santa Susanna Avenue	1.46	450	990	1,300	2,000
CHESEBORO CREEK					
1,100 feet Upstream of Driver Avenue	7.6	2,169	4,779	6,088	9,551
HACIENDA CREEK					
Cross Section A	1.46	626	1,381	1,762	2,758
HARBOR AREA					
North of Carson Street Between Vermont and Berendo Avenues	0.35	74	164	209	327
HIDDEN SPRINGS AREA					
Mill Creek (Cross Section B)	14.8	2,274	5,019	6,405	10,024
INDUSTRY AREA					
Vicinity of Brea Canyon Road and Lycoming Street	3.85	952	2,102	2,682	4,197

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second)			
		10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
IRON CANYON					
Approximately 2,000 feet Upstream of Sand Canyon Road	2.8	--	--	2,078	2,833
KAGEL CANYON AREA					
Kagel Canyon Channel (Cross Section A)	2.04	490	1,081	1,380	2,159
Little Tujunga Wash Approximately 3,000 feet Upstream of the City of Los Angeles Corporate Limits	17.9	2273	5,019	6,405	10,022
LA MIRADA AREA					
Mystic Street, Vicinity of Parkinson Avenue	0.31	81	179	228	357
LA MIRADA CREEK					
At Ocaso Avenue	4.6	610	1,340	1,700	2,670
Approximately 1100 feet Downstream of La Mirada Boulevard	5.0	610	1,350	1,720	2,690
LADERA HEIGHTS AREA					
Vicinity of La Cienega Boulevard and Slauson Avenue	0.53	138	305	389	609

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second)			
		10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
LAS VIRGENES CREEK					
Approximately 1500 feet downstream of the confluence of Stokes Canyon	24.3	9,230	13,678	15,521	18,704
Approximately 250 feet downstream of the confluence of Stokes Canyon	24.3	9,228	13,673	15,515	18,811
At the confluence of Stokes Canyon	19.7	9,193	13,766	15,646	19,340
Just downstream of Mulholland Highway	19.1	6,873	10,346	11,929	14,853
At the confluence of Liberty Canyon	16.6	6,871	10,348	11,935	15,210
Approximately 1500 feet upstream of the confluence of Liberty Canyon	16.5	5,862	8,799	10,069	12,755
Approximately 4000 feet upstream of the confluence of Liberty Canyon	16.2	5,783	8,676	9,913	12,554
Approximately 1800 feet downstream of Lost Hills Road	15.0	5,414	8,112	9,246	11,714
Just downstream of Lost Hills Road	15.0	5,420	8,133	9,281	11,764
Just downstream of Meadow Creek Lane	14.9	5,414	8,124	9,269	11,751
Approximately 1600 feet upstream of Meadow Creek Lane	13.3	4,860	7,211	8,197	10,356

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second)			
		10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
Just downstream of Agola Road	12.7	4,783	7,040	8,005	10,076
Just downstream of US Highway 101	10.4	3,830	5,644	6,419	8,137
Just downstream of Las Virgenes Road	10.2	3,787	5,577	6,340	8,044
LINDERO CANYON					
700 feet Downstream of Thousand Oaks Boulevard	4.1	1,369	3,024	3,858	6,037
At Reyes Adobe Road	3.4	1,290	2,847	3,632	5,685
LITTLE ROCK WASH					
Little Rock Reservoir	48.0	--	--	20,000	--
LOCKHEED DRAIN CHANNEL					
Approximately 150 feet Downstream of Hollywood Way	0.9	--	--	965	--
Approximately 300 feet Upstream of Lima Street	1.44	--	--	1,635	--
At Ontario Street	1.82	--	--	2,054	--
Approximately 100 feet Downstream of Naomi Street	1.89	--	--	2,026	--

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second)			
		10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
Approximately 300 feet Downstream of Victory Place	2.48	--	--	2,410	--
--Data Unknown					
Approximately 100 feet Downstream of Burbank Boulevard	3.73	--	--	2,910	--
LOPEZ CANYON AREA					
Lopez Canyon Channel (Cross Section A)	1.78	682	1,506	1,922	3,007
LOS ANGELES RIVER					
At Compton Creek	808	92,900	133,000	142,000	143,000
At Imperial Highway	752	89,400	126,000	140,000	156,000
MALIBU AREA					
Trancas Creek Upstream of Pacific Coast Highway (Cross Section A)	8.6	2,499	5,518	7,040	11,106
Zuma Canyon (Cross Section A)	8.9	2,024	4,469	5,705	8,925
Zuma Canyon (Cross Section W)	8.4	2,079	4,590	5,858	9,167
Ramirez Canyon (Cross Section B)	3.3	1,066	2,352	3,000	4,696
Ramirez Canyon (Cross Section I)	2.8	1,150	2,540	3,240	5,070

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second)			
		10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
Escondido Canyon (Cross Section B)	3.2	958	2,116	2,700	4226
Escondido Canyon (Cross Section F)	1.7	986	2176	2778	4,346
Malibu Creek (Cross Section A)	109.6	14183	31,648	40,544	63,934
Malibu Creek (Cross Section A)	109.2	14,183	31,648	40,544	63,934
Unnamed Canyon (Serra Retreat Area) (Cross Section C)	0.4	281	619	791	1,237
Las Flores Canyon (Cross Section F)	4.1	1,758	3,882	4,954	7,752
Topanga Canyon (Cross Section H)	19.6	4,095	9,040	11,537	18,054
Topanga Canyon (Cross Section M)	15.0	5,404	11,930	15,223	23,882
Topanga Canyon (Cross Section Q)	14.5	5,208	11,499	14,672	22,960
Topanga Canyon (Cross Section T)	7.3	2,560	5,656	7,215	11,289
Topanga Canyon (Cross Section V)	7.0	2,364	5,222	6,601	10,422
Topanga Canyon (Cross Section X)	5.5	1,862	4,113	5,247	8,210
Topanga Canyon (Cross Section AG)	0.3	259	572	729	1,141
Santa Maria Canyon (Cross Section C)	3.1	1,070	2,333	3,016	4,719
Old Topanga Canyon (Cross Section E)	1.7	567	1,253	1,597	2,499

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second)			
		10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
Old Topanga Canyon (Cross Section H)	0.8	251	554	706	1,104
Garapito Canyon (Cross Section A)	2.9	996	2,171	2,807	4,392
Garapito Canyon (Cross Section E)	2.0	675	1,470	1,910	2,974
Cold Creek (Cross Section A)	8.1	2,280	5,019	6,406	10,023
Cold Creek (Cross Section C)	7.8	2,280	5,041	6,432	10,066
Cold Creek (Cross Section G)	5.7	1,734	3,826	4,881	7,640
Dark Canyon (Cross Section A)	1.2	753	1,600	2,118	3,314
Lobo Canyon (Cross Section A)	3.8	1,572	3,473	4,429	6,932
Lobo Canyon (Cross Section A)	2.5	1,625	3,588	4,579	7,166
Stokes Canyon (Cross Section B)	2.9	1,089	2,403	3,067	4,799
Stokes Canyon (Cross Section B)	2.4	934	2,062	2,631	4,117
Dry Canyon (Cross Section C)	1.1	527	1,104	1,484	2,323
Dry Canyon (Cross Section M)	0.8	490	1,083	1,382	2,162
Dry Canyon (Cross Section T)	0.4	242	534	681	1,065
Cheseboro Creek (Cross Section B)	7.6	2,169	4,779	6,088	9,551
Palo Comado Creek (Cross Section E)	4.1	1,159	2,562	3,268	5,113

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second) 10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
Palo Comado Creek (Cross Section J)	3.5	1,074	2,374	3,028	4,738
Palo Comado Creek (Cross Section K)	3.2	1,032	2,279	2,908	4,551
Liberty Canyon (Cross Section E)	1.4	938	2,072	2,645	4,140
Medea Canyon (Cross Section B)	24.6	5,794	12,788	16,319	25,537
Medea Canyon (Cross Section H)	23.0	6,174	13,628	17,389	25,537
Medea Canyon (Cross Section K)	22.2	6,363	14,074	17,925	28,049
Medea Canyon (Cross Section P)	6.3	2,558	5,647	7,204	11,272
Lindero Canyon (Cross Section C)	6.7	1,725	3,809	4,860	7,604
Lindero Canyon (Cross Section E)	4.1	1,369	3,024	3,858	6,037
Lindero Canyon (Cross Section H)	3.8	1,343	2,965	3,783	5,920
Lindero Canyon (Cross Section M)	3.4	1,290	2,847	3,632	5,685
Lindero Canyon (Cross Section N)	3.1	1,258	2,776	3,542	5,545
Triunfo Creek (Cross Section B)	28.7	4,781	11,396	14,898	24,298
Triunfo Creek (Cross Section E)	28.3	4,846	11,544	15,090	24,606
Malibu Lake	64.6	11,859	26,556	34,043	53,712

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second)			
		10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
MEDEA CREEK					
Downstream of Venture Highway	6.3	2,560	2,645	7,200	11,270
Approximately 950 feet Upstream of Canwood Street	--	--	--	6,720	--
Approximately 1,100 feet Upstream of Kanan Road	--	--	--	5,960	--
At Thousand Oaks Boulevard	--	--	--	5,946	--
Approximately 1,700 feet Downstream of Laro Drive	4.1	--	--	5,320	
Approximately 575 feet Downstream of Fountainwood Street	3.9	--	--	5,240	--
Just Upstream of Fountainwood Street	3.4	--	--	4,700	--
MINT CANYON					
Downstream of Sierra Highway Crossing	29.3	--	--	8,300	14,581
Downstream of Vasquez Canyon Road	26.8	--	--	7,896	14,179

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second)			
		10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
Approximately 2,600 feet Downstream of Davenport Road	19.9	--	--	6,691	12,604
NEWHALL CANYON					
Approximately 800 feet Upstream of Railroad Canyon	5.2	--	--	3,224	4,396
Approximately 650 feet Upstream of Railroad Canyon	6.2	--	--	3,390	5,424
Approximately 650 feet Downstream of Railroad Canyon	7.3	--	--	3,892	6,228
OAK SPRINGS CANYON					
Approximately 100 feet Upstream of Union Pacific Railroad (former Southern Pacific Railroad)	5.7	--	--	2,703	4,054
OVERLAND FLOW					
North of Florence Avenue and East of Pioneer Boulevard	1.34	270	596	760	1,190

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second)			
		10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
North of Lakeland Road, 1000 feet East of Bloomfield Avenue	0.42	68	151	192	301
Marquardt Avenue, 1400 feet North of Rosecrans Avenue	2.09	411	907	1,158	1,812
PALO COMADO CREEK					
At Fairview Place	3.5	1,074	2,374	3,028	4,738
PLACERITA CREEK					
Approximately 575 feet Downstream of San Fernando Road	9.3	--	--	5,321	7,981
Approximately 2,900 feet Upstream of San Fernando Road	8.6	--	--	4,988	7,482
Approximately 2,000 feet Upstream of Quigley Canyon Road	7.1	--	--	4,085	6,313
Approximately 850 feet Downstream of Antelope Valley Freeway	6.3	--	--	3,546	5,673
PONDING					
At Intersection of Mines Avenue and Taylor Avenue	0.5	120	250	330	510

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second)			
		10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
Savage Creek at Intersection of York Avenue and Mar Vista Street	0.9	260	570	730	1,150
Turnbull Canyon at intersection of Painter Avenue and Camilla Street	1.0	250	540	690	1,080
PORTAL RIDGE WASH					
Intersection of Avenue H and Antelope Valley Freeway	147.0	1,600	5,000	7,200	16,000
RIO HONDA					
At Stewart and Gray Road	132	35,600	41,000	39,300	40,200
At Beverly Boulevard	113	33,800	37,50	38,000	38,400
At Outflow from Whittier Narrows Dam	110	33,500	36,500	36,500	36,500
SAN FERNANDO VALLEY DISTRICT					
San Fernando Pacoima Wash, Approximately 150 feet Downstream of Shablow Avenue	31.07	1,900	5,600	8,100	12,100
Lockheed Drain Channel, Approximately 450 feet Upstream of Clybourn Avenue	0.42	278	--	448	--

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second)			
		10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
LAKEVIEW TERRACE					
Little Tujunga Canyon, Approximately 1,600 feet Upstream of Foothill Boulevard	20.29	2,700	6,000	7,700	12,200
Kagel Canyon,Approximately 650 feet Upstream of Osborne Avenue	2.04	490	1,100	1,400	12,200
SUNLAND					
Big Tujunga Canyon, Approximately 1,200 feet Upstream of Foothill Boulevard and Tujuna Valley Street	34.57	8,100	24,700	36,500	62,600
Big Tujunga Canyon, Upstream of Wheatland Avenue	43.25	9,300	26,800	38,900	66,000
SYLMAR					
East Side of Golden State Freeway South of Sierra Highway	0.22	50	120	150	240
Weldon Canyon, Approximately 1,570 feet Downstream of Sierra Highway and San Fernando Road	1.47	410	900	1,150	1,800

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second)			
		10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
VAN NUYS					
Victory Boulevard, Vicinity of Hayvenhurst Avenue	0.73	90	200	250	390
PORTER RANCH					
Mayerling Street, Northwest of Shoshone Avenue	0.19	40	100	120	190
Vicinity of Sesnon Boulevard	0.10	30	60	70	120
GRANADA HILLS					
Superior Street, West of Paso Robles Avenue	0.53	90	200	260	400
Vicinity of Balboa Boulevard and Citronia Street	0.53	90	200	260	400
SEPULVEDA					
Roscoe Boulevard at Haskell Avenue	0.84	160	360	460	720
Haskell Avenue North of Union Pacific Railroad (former Southern Pacific Railroad)	1.0	230	500	640	1,000
CHATSWORTH					
Vicinity of Chatsworth Street and Corbin Avenue	0.85	220	480	610	960

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second)			
		10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
Vicinity of Variel Avenue and Chatsworth Street	13.43	2,100	4,700	6,000	9,300
Vicinity of Canoga Avenue and Devonshire Street	0.77	230	510	650	1,000
Vicinity of Valley Circle Boulevard and Lassen Street	0.75	220	480	600	950
Vicinity of Topanga Canyon Boulevard and Lassen Street	0.25	50	120	150	230
Vicinity of Farrolone Avenue and Lassen Street	0.42	100	220	280	440
Vicinity of Topanga Canyon Boulevard and Santa Susana Place	0.10	20	50	60	100
Vicinity of Santa Susana Pass Road and Santa Susana Avenue	1.46	450	990	1,300	2,000
WOODLAND HILLS					
Vicinity of Mulholland Drive and Ventura Freeway	2.27	490	1,100	1,400	2,200
Vicinity of Saltillo Street and Canoga Avenue	0.32	100	250	300	500
SHERMAN OAKS					
Magnolia Boulevard at Haskell Avenue	1.23	360	800	1,000	1,600

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second)			
		10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
SAN GABRIEL RIVER					
Whittier Narrows Flood Control Basin At Siphon Road	524.0	--[2]	--[2]	90,000	--[3]
SAND CANYON					
Approximately 250 feet Downstream of Confluence with Iron Canyon	10.1	--	--	6,372	8,689
Approximately 2,900 feet Downstream of Placerita Canyon Road	7.3	--	--	4,908	6,693
Approximately 800 feet Upstream of Placerita Canyon Road	6.4	--	--	4,371	5,961
SAND CANYON LATERAL					
At Robinson Ranch Road	0.9	--	--	1,480	--

-- Data Unknown
[2] Discharge not determined because 1% Annual Chance Flood is contained within Whittier Narrows Flood Control Basin
[3] Not Required by the Federal Insurance Administration

SANTA CLARA RIVER					
Approximately 2,600 feet Upstream of Los Angeles Aqueduct	235.4	--	--	15,182	26,369

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second) 10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
At Sand Canyon Road	179.4	--	--	8,408	13,849
SANTA CLARITA VALLEY					
Santa Clara River Approximately 3,500 feet Upstream of Arrastre Canyon Road	67.7	--	--	8,408	13,849
Santa Clara River 7,600 feet Upstream of Oak Springs Canyon	172.7	--	--	13,412	22,588
Santa Clara River at Sand Canyon Road	179.4	--	--	13,934	23,467
Mint Canyon 3,600 feet Downstream of Vasquez Canyon Road	26.8	--	--	7,896	14,179
Mint Canyon 1,600 feet Downstream of Sierra Highway Crossing	29.3	--	--	8,300	14,581
Mint Canyon Approximately 2,600 feet Downstream of Davenport Road	19.9	--	--	6,691	12,604
Vasquez Canyon Approximately 1,373 feet Upstream of Vasquez Canyon Road	4.2	--	--	2,851	5,009
Bouquet Canyon Approximately 4,500 feet Upstream of Vasquez Canyon Road	38.6	--	--	11,303	23,161
Placerita Creek Approximately 850 feet Downstream of Antelope Valley Freeway	6.3	--	--	3,546	5,673

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second)			
		10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
Placerita Creek Approximately 2,000 feet Upstream of Quigley Canyon Road	7.1	--	--	4,085	6,313
Placerita Creek Approximately 2,900 feet upstream of Quigley Canyon Road	8.6	--	--	4,988	7,482
Placerita Creek Approximately 575 feet Upstream of San Fernando Road	9.3	--	--	5,321	7,981
Newhall Creek Approximately 800 feet Downstream of Sierra Highway	5.2	--	--	3,224	4,396
Newhall Creek Approximately 650 feet Upstream of Railroad Canyon	6.2	--	--	3,390	5,424
Newhall Creek Approximately 650 feet Downstream of Railroad Canyon	7.3	--	--	3,892	6,228
Railroad Canyon Approximately 350 feet upstream of San Fernando Road	1.2	--	--	835	1,253
South Fork Santa Clara River Approximately 600 feet Downstream of Golden State Freeway	12.8	--	--	8,417	13,596
Wildwood Canyon Approximately 600 feet Upstream of Intersection of Valley Street and Maple Street	0.23	--	--	172	279

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second)			
		10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
South Fork Santa Clara River Approximately 500 feet Downstream of Wiley Canyon Road	12.9	--	--	8,,483	13,704
Santa Clara River Approximately 2,600 feet Upstream of Los Angeles Aqueduct	235.4	--	--	15,182	26,369
Approximately 1,800 feet South of Intersection of San Fernando Road and Magic Mountain Parkway	1.9	--	--	1,437	2,495
Bouquet Canyon Approximately 2,600 feet Upstream of Bouquet Canyon Road	32.1	--	--	11,117	22,707
Plum Canyon Approximately 2,350 feet Upstream of Bouquet Canyon Road	3.4	--	--	1,942	3,453
Haskell Canyon Approximately 1,300 feet Downstream of Headworks	6.7	--	--	5,363	10,516
Haskell Canyon Approximately 6,400 feet Upstream of Confluence with Bouquet Canyon	10.4	--	--	7,268	14,072
Dry Canyon Approximately 2,000 feet Upstream of San Francisquito Road	5.5	--	--	5,235	10,470
San Martinez-Chiquito Canyon Approximately 1,000 feet Upstream of Chiquito Canyon Road (Lower Crossing)	4.7	--	--	4,659	8,607

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second)			
		10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
San Martinez-Chiquito Canyon Approximately 400 feet Upstream of Chiquito Canyon Road (Upper Crossing)	3.1	--	--	3,112	5,705
San Martinez-Chiquito Canyon Approximately 250 feet Downstream of Verdale Street	1.1	--	--	1,205	2,208
Halsey Canyon Approximately 1,150 feet Downstream of Halsey Canyon Road	7.3	--	--	5,544	10,163
Halsey Canyon Approximately 550 feet Downstream of Romero Canyon Road	5.9	--	--	4,523	8,292
Castaic Creek Approximately 2,100 feet Upstream of Confluence with Charlie Canyon	16.8	--	--	11,805	22,326
Violin Canyon Approximately 2,000 feet Downstream of Interstate Highway 5	10.5	--	--	9,421	17,818
Gorman Creek Approximately 250 feet North of Interstate Highway 5 Overcrossing Gorman Road	3.8	--	--	1,713	3,221
Elizabeth Canyon Approximately 2,300 feet Downstream of Elizabeth Lake Pine Canyon Road	7.7	--	--	3,455	7,176

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second)			
		10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
Pine Canyon Approximately 1,200 feet Upstream of Lake Hughes Road	6.4	--	--	2,969	6,166
Dowd Canyon at Calle Corona Extended	3.9	--	--	2,982	5,963
San Francisquito Canyon at Spunky Road	2.7	--	--	2,140	4,281
SANTA FE SPRINGS AREA					
Vicinity of Rivera Road and Vicki Drive	0.38	80	176	225	352
SHALLOW FLOODING					
Turnbull Canyon in the Vicinity of Broadway and Alta Drive	1.0	250	540	690	1,080
At intersection of Ripley Avenue and Rindge Lane	N/A	61	135	172	270
At Gould Avenue between Ford and Goodman Avenues	0	66	146	186	291
At intersection of Vincent Street and South Irena Avenue	N/A	68	149	190	298
At intersection of Camino Real and South Juanita Avenue	10	50	111	141	221
At intersection of Avenue H and Massena Avenue	5[1]	154	340	434	679

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second) 10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
SOUTH FORK SANTA CLARA RIVER					
Approximately 500 feet downstream of Wiley Canyon Road	12.9	--	--	8,483	13,704
Approximately 600 feet downstream of Golden State Freeway	12.8	--	--	8,417	13,596
Surface Runoff at Intersection of Garfield Avenue and Beverly Boulevard	2.9	820	1,810	2,310	3,610
Vicinity of Rosewood Avenue and Huntley Drive West Los Angeles and Central Districts	1.06	670	1,479	1,888	3,329
Happy Lane	1.73	640	1,400	1,800	2,800
Laurel Canyon Boulevard at Hollywood Boulevard	1.91	600	800	1,160	2,100
WEST HOLLYWOOD					
Genesse Avenue North of Hollywood Boulevard	1.00	370	820	1,000	1,600

[1] Pump Capacity

--Data Unknown

Third Street, Vicinity of La Cienga Boulevard	5.10	1,600	3,500	4,500	7,200

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second) 10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
Fifth Street, Vicinity of Orlando Avenue	5.66	1,600	3,600	4,500	7,100
Beverly Boulevard, Vicinity of Spaulding Avenue	4.02	730	1,600	2,100	2,900
Third Street, Vicinity of Fairfax Avenue	6.13	1,500	3,200	4,100	6,800
HOLLYWOOD					
Santa Monica Boulevard, Vicinity of Mariposa Avenue	2.79	940	2,100	2,700	4,200
South of Hollywood Freeway, Vicinity of Kenmore Avenue	3.20	830	1,800	2,300	3,700
Third Street at Kenmore Avenue	3.43	800	1,800	2,300	3,500
Madison Avenue at Monroe Street	0.54	160	350	440	690
SILVER LAKE					
Griffith Park Boulevard at Tracy Street	0.64	220	490	620	970
Between Hyperion Avenue and Griffith Park Boulevard, North of Fountain Avenue	0.91	290	650	830	1,300
Myra Avenue, Vicinity of Del Mar Avenue	1.80	490	1,110	1,400	2,200
Silver Lake Boulevard East of Virgil Avenue	1.27	420	900	1,100	1,800

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second)			
		10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
WESTLAKE					
Vicinity of Wilshire Boulevard West of Hoover Street	1.40	360	790	1,000	1,600
HANCOCK PARK					
Sixth Street, Vicinity of Alexandria Avenue	8.09	2,100	4,600	5,900	9,200
Lucerne Boulevard at Francis Avenue	0.26	70	160	200	320
Olympic Boulevard at Hudson Avenue	0.56	130	290	370	570
Vicinity of Western Avenue and 11th Street	3.48	670	1,300	1,600	2,500
Vicinity of Bronson Avenue and Country Club Drive	18.07	3,700	7,900	9,600	14,000
Vicinity of West Boulevard and Dockweiler Street	18.76	3,600	7,600	9,300	13,600
Vicinity of San Vicente and Pico Boulevards	18.91	3,500	7,400	9,000	13,100
Vicinity of Highland Avenue and St. Elmo Drive	20.21	3,600	7,700	9,300	13,700
Arlington Avenue, Vicinity of 37th Place	0.73	440	990	1,400	2,500
Victoria Avenue, Vicinity of Jefferson Boulevard	1.17	320	1,100	1,400	2,600

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second) 10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
Chesapeake Avenue, Vicinity of Exposition Boulevard	7.97	1,100	2,400	3,000	3,700
Harcourt Avenue, Vicinity of Westhaven Street	0.53	160	350	450	700
PARK LA BREA					
Wilshire Boulevard, Vicinity of Crescent Heights Avenue	6.62	1,500	3,300	4,200	6,600
Vicinity of Orange Drive and Pickford Street	24.67	4,400	9,500	11,800	17,700
Vicinity of Whitworth Drive and La Cienega Boulevard	17.13	3,400	7,600	9,700	15,200
Venice Boulevard, Vicinity of Fairfax Avenue	18.44	3,400	7,500	9,500	14,900
Redondo Boulevard, Vicinity of Santa Monica Freeway	1.16	300	670	860	1,300
Redondo Boulevard, Vicinity of Roseland Street	14.53	2,000	4,400	5,700	9,100
Houser Boulevard, Vicinity of La Cienega Boulevard	14.76	1,900	4,300	5,500	8,800
Fairfax Avenue, Vicinity of La Cienga Boulevard	16.67	2,100	4,700	6,000	9,600

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second)			
		10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
WEST LOS ANGELES					
Balsam Avenue, Vicinity of Olympic Boulevard	1.19	290	550	660	940
Manning Avenue, Vicinity of Tennessee Avenue	3.40	530	1,300	1,700	2,600
Between Westwood Boulevard and Overland Avenue, Vicinity of Exposition Boulevard	4.00	190	1,200	1,500	2,700
Roundtree Road, Vicinity of Manning Avenue	0.72	500	740	840	1,100
CENTURY CITY					
Northwest of Santa Monica Boulevard and Avenue of the Stars	0.49	400	590	700	900
BEL AIR ESTATES					
Stone Canyon Road South of Somma Way	0.66	480	710	800	1,100
Stone Canyon Road South of Bellagio Road	1.02	630	940	1,100	1,400
Beverly Glen Boulevard North of Sunset Boulevard	1.18	700	1,000	1,200	1,600

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second)			
		10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
BRENTWOOD					
North of San Vicente Boulevard, West of Westgate Avenue	0.21	60	140	180	280
Northeast of Sunset Boulevard and Barrington Avenue	0.24	230	340	390	520
PACIFIC PALISADES					
Rustic Canyon, Approximately 1,030 feet Downstream (South) of Sunset Boulevard	5.67	700	1,500	2,000	3,100
WESTCHESTER					
Approximately 300 feet East of Sepulveda Boulevard and 1,300 feet North of 74th Street	1.39	310	690	880	1,400
Sepulveda Boulevard South of San Diego Freeway	1.39	310	690	880	1,400
Arizona Avenue North of Arizona Circle	1.65	340	740	950	1,500
HYDE PARK					
Halldale Avenue, Vicinity of 65th Street	1.20	300	660	850	1,300
Wilton Place, Vicinity of Gage Avenue	3.29	770	1,600	1,900	3,000

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second)			
		10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
South of Southwest Drive, Vicinity of Van Ness Avenue	4.15	730	1,600	2,100	3,200
HARBOR DISTRICT					
Harbor Lake, Southeast of Vermont Avenue and Pacific Coast Highway	18.97	3,200	7,000	8,900	14,000
Denker Avenue, Vicinity of 204th Street	0.28	60	130	170	260
UNNAMED STREAM MAIN REACH					
At Pacific Ocean	1.2	353	724	917	1,400
Downstream of Confluence of Tributary 3	1.1	338	692	876	1282
Upstream of Confluence of Tributary 1	0.37	146	290	361	523
Upstream of Confluence of Tributary 2	0.65	229	462	580	865
UNNAMED STREAM TRIBUTARY 1					
At Confluence with Main Reach	0.21	97	191	236	381
UNNAMED STREAM TRIBUTARY 1 (continued)					
Downstream of Confluence of Tributary 1	0.58	209	421	527	787

Table 6: Summary of Peak Discharges (continued)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (cubic feet per second) 10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
UNNAMED STREAM TRIBUTARY 2					
At Confluence with Main Reach	0.44	164	331	413	600
At Via Zurita	0.38	144	290	361	525
WEST HOLLYWOOD AREA					
Vicinity of Rosemead Avenue and Huntley Drive	1.06	670	1,479	1,888	3,329
Vicinity of Pan Pacific Auditorium	4.02	730	1,600	3,600	4,500
WHITTIER AREA					
Vicinity of Turnbull Canyon Road	1.0	246	543	692	1,084
Whittier Narrows Flood Control Basin	524	--[2]	--[2]	90,000	--[3]
WINSOR HILLS AREA					
Vicinity of La Brea and Slauson Avenues	0.25	67	147	188	294

-- Data Unknown

[1] Pump Capacity

[2] Discharge not determined because 1% Annual Chance Flood is contained within Whittier Narrows Flood Control Basin

[3] Not Required by the Federal Insurance Administration

Table 7: Summary of Breakout Discharges

Flooding Source and Location	Breakout Discharges (cubic feet per second)			
	10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
COMPTON CREEK				
Upstream of the Confluence of Compton Creek and Los Angeles River, Right Overbank	--	--	14,800	--
LOS ANGELES RIVER	--	--	75,200	--
At Fernwood Avenue	--	--	57,000	--
Left Overbank	--	--	18,200	--
Right Overbank	--	--	45,400	--
At Wardlow Road	--	--	14,200	--
Left Overbank	--	--	31,200	--
Right Overbank	--	--	75,200	--
RIO HONDA				
At Beverly Boulevard, Left Overbank	--	--	13,700	--
At Stewart and Gray Road	--	--	2,790	--
Left Overbank	--	--	1,395	--
Right Overbank	--	--	1,395	--

Table 7: Summary of Breakout Discharges (continued)

Flooding Source and Location	Breakout Discharges (cubic feet per second)			
	10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
UPPER LOS ANGELES RIVER				
At Broadway, Left Overbank	--	--	100	--

-- Data Unknown

Table 8: Summary of Elevations

Flooding Source and Location	10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
Los Angeles River	7.3	7.8	9.9	15.6
Los Cerritos Channel	6.9	7.5	8.7	12.2
Pacific Ocean				
San Pedro Bay	7.4	7.9	10.0	15.7
San Pedro Bay	7.0	7.6	8.8	12.3
San Pedro Bay	8.9	--	8.9	--
Alamitos Bay	7.0	7.6	8.8	12.3
Swimming Lagoon	7.4	7.9	10.0	15.7
At King Harbor	6.9	6.9	6.9	8.3
At Pleasure Pier	8.9	--	8.9	--
At Pleasure Pier	10.3	11.2	11.6	12.3
Ponding 600 feet East of Bloomfield Avenue North of Lakeland Road	139.8	142.8	143.8	143.8
Ponding 1,000 feet East of Bloomfield Avenue North of Lakeland Road	116.8	148.3	148.8	149.8

Table 8: Summary of Elevations (continued)

Flooding Source and Location	10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
Ponding at Marquardt Avenue 1,400 feet North of Rosecrans Avenue	83.8	85.8	86.8	88.8
Ponding from Savage Creek				
Intersection of York Avenue and Mar Vista Street	382.8	382.8	382.8	382.8
Ponding from Turnbull Canyon				
Intersection of Painter Avenue and Camilla Street	411.8	419.8	420.8	421.8
San Gabriel River				
At Whittier Narrows Flood Control Basin	213.8	222.8	222.8	231.8
Shallow Flooding				
Intersection of Ripley Avenue and Rindge Lane	--	62.9	64.9	68.9
At Gould Avenue between Ford and Goodman Avenues	83.4	91.4	95.9	105.9
Intersection of Vincent Street and South Irena Avenue	81.9	82.9	83.6	84.9
Intersection of Camino Real and South Juanita Avenue	120.5	121.9	122.9	124.3

Table 8: Summary of Elevations (continued)

Flooding Source and Location	10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
Intersection of Avenue H and Massena Avenue	61.4	64.4	65.4	67.4
Surface Runoff – Deep Ponding Area				
Southwest of the Intersection of Carson Street and Madrona Avenue	60.1	66.1	68.8	74.8
Intersection of Doris Way and Reese Road	61.6	64.8	65.8	67.7
Surface Runoff – Ponding Area				
Intersection of Anza Avenue and Spencer Street	82.6	83.4	83.8	84.9
Northeast of Sepulveda Boulevard and Madrona Avenue	77.3	78.4	78.8	79.5
Intersection of California Street and Alaska Avenue	78.7	80.1	80.8	81.6
Intersection of Mines Avenue and Taylor Avenue	186.7	188.8	188.8	188.8

-- Data Unknown

3.1.2 Methods for Flooding Sources Incorporated from Previous Studies

Many of the incorporated community within, and the unincorporated areas of Los Angeles County, have a previously printed FIS report. The hydrologic analyses described in those reports have been compiled and are summarized below.

Because many of the communities affected by the Los Angeles River and its tributaries were removed from the regulatory floodplain based on completion of the Los Angeles County Drainage Area (LACDA), the discussion in this FIS for numerous communities is based on the revised analyses conducted by the Corps of Engineers, and reviewed and certified by the USACE and FEMA, for that project. Information on the methods used to determine peak discharge-frequency relationships for the streams restudied as part of this countywide FIS is shown below.

Depending on the availability of hydrologic data, numerous different approaches were used throughout the County. These are discussed in the following paragraphs.

Los Angeles County Unincorporated and Incorporated Areas

For the 2008 countywide study, hydrologic analyses were carried out to establish peak discharge frequency relationships for each flooding source studied by detailed and approximate methods affecting the community.

Discharges for the 1-percent-annual-chance recurrence interval for all new enhanced approximate and approximate study streams in Neshoba County were determined using the Rural-East Region USGS regression equations for Mississippi as described in the USGS Water-Resources Investigations report 94-4002 (Reference 8).

Drainage areas along streams were determined using a flow accumulation grid developed from the USGS 10 meter digital elevation models and corrected National Hydrologic Data (NHD) stream coverage. Flow points along stream centerlines were calculated using the regression equations in conjunction with accumulated area for every 10 percent increase in flow along a particular stream.

Los Angeles County

Antelope Valley (not including the communities of Lancaster and Palmdale).

The USACE, Los Angeles District, developed discharge-frequency relationships for the Antelope Valley. The USACE using the log-Pearson Type III frequency analysis computed the 1-percent annual chance peak flow rates for Little Rock Creek and Big Rock Creek. The gage for Little Rock Creek, located at Little Rock Reservoir, has operated since 1931 and records flow from a drainage area of approximately 48 square miles. The gage located at the mouth of Big Rock Creek has been operated since 1923 and records flow from a drainage area of approximately 23 square miles.

The remaining streams tributaries to the Antelope Valley are ungaged. Therefore, discharge-frequency curves were developed by the USACE from the Little Rock Creek and Big Rock Creek curves. An average of the two curves was developed using standard deviation and average skew coefficient of the two gages. The USACE Standard Project Flood peak discharge at the concentration points was used as the basis for transposing the frequency curves to ungaged streams.

For the summer peak discharges in the Antelope Valley desert region, the USACE determined from gages on nine streams that the major events were independent with relatively short records. Therefore, the peak discharges were considered collectively as a single flood record representative of the region.

To develop a summer storm discharge-frequency curve at any ungaged location, the Standard Project Flood was used as the basis for transposing the frequency curves.

The Los Angeles County Flood Control District employed the USACE study as a data base to develop yield-versus-area curves for the 10-, 2-, 1-, and 0.2-percent annual chance frequency flow rates for the concentration points. These curves were used to determine the peak flow rates for intermediate points along the major watercourses and for adjacent watersheds.

Santa Clarita Valley (not including the City of Santa Clarita)

Much of the hydrologic data for this portion of the County was also supplied by the USACE. For watersheds greater than 20 square miles, the USACE formula for the geometric mean flood was used to predict 1-percent annual chance frequency peak flow rates. For drainage areas less than 20 square miles, this formula was modified slightly to yield runoff values more closely related to observed values using engineering judgment. This modification was reviewed by the Los Angeles District office of the USACE.

Malibu Area

Streams in the Malibu area that have Los Angeles County Flood Control District gage records sufficient for frequency analysis are Malibu Creek, Station F130-R; Zuma Creek, Station F53-R; and Topanga Canyon, Station F548-R. The peak flow rates were computed at these locations using log-Pearson Type III frequency analysis. Following this analysis, the peak flow rates were also computed using the Regional Runoff Frequency Equations developed by the Los Angeles County Flood Control District. These regional runoff frequency equations were developed through the multiple-linear regression analysis of the peak flow data of 48 gaging stations in Los Angeles County. Comparison of the results obtained indicated that the log-Pearson Type III analysis of the stream gages in the Malibu area produced higher peak flow rates than the Regional Runoff Frequency Equations. Therefore, the ratio of the flow rates predicted by the two methods was computed at each gage. Flow rates were then computed for the remaining points in the watershed by multiplying the regional equation flow rate by the appropriate ratio. The ratio used was determined by comparing the watershed being analyzed to those analyzed by the log- Pearson Type III analysis to determine which one was most similar.

Los Angeles Basin

The remaining portions of unincorporated territory are located in the Los Angeles basin and were analyzed in conjunction with the incorporated cities on a drainage area basis. For streams with gages of sufficient length of reliable record, log-Pearson Type III analysis was used to determine 1-percent annual chance flood flow rates. The flow rates for the remaining streams were calculated by the Regional Runoff Frequency Equations developed by the District.

The flow rates used in the Los Angeles County study do not reflect the substantial amount of mud and debris flows which can be generated by a burned watershed. Therefore, it should be emphasized that the results of the study do not reflect the true degree of flood and mudflow hazard to the community.

Due to the configuration of the channels and overbanks, storage can cause floods to pond or break away from the channels resulting in an inverse discharge-drainage area relationship to exist along portions of Zuma, Ramirez, Escondido, Topanga, and Lobo Canyons, and Medea and Triunfo Creeks.

Analyses were carried out to establish the peak elevation-frequency relationships for each flooding source studied in detail.

Coastal flood hazard areas subject to inundation by the Pacific Ocean were determined on the basis of water-surface elevations established from regression relations defined by Thomas. These regression relations were defined as a practical method for establishing inundation elevations at any site along the southern California mainland coast. They were defined through analysis of water-surface elevations established for 125 locations in a complex and comprehensive model study by Tetra Tech, Inc. The regression relations establish wave run-up and wave set-up elevations having 10-, 1-, and 0.02-percent chances of occurring in any year and are sometimes referred to as the 10-, 100-, and 500-year flood events, respectively.

Wave run-up elevations were used to determine flood hazard areas for sites along the open coast that are subject to direct assault by deep-water waves. Runup elevations range with location and local beach slope and were computed at 0.5-mile intervals, or more frequently in areas where the beach profile changes significantly over short distances. Areas with ground elevations 3.0 feet or more below the 1- percent annual chance wave run-up elevation are subject to velocity hazard.

Wave setup elevations determined from the regression equations on the basis of location along the coast were used to identify flood hazard areas along bays, coves, and areas sheltered from direct action of deep-water waves.

City of Agoura Hills

Streams in the Malibu area that have Los Angeles County Flood Control District gage records sufficient for frequency analysis are Malibu Creek, Station F130-R; Zuma Creek, Station F53-R; and Topanga Canyon, Station F548-R. The peak flow rates were computed at these locations using log-Pearson Type III frequency analysis (U.S. Water Resources Council, March 1976). Following this analysis, the peak flow rates were also computed using the Regional Runoff Frequency Equations developed by the Los Angeles County Flood Control District (Los Angeles County Flood Control District, November 1977). These regional runoff frequency equations were developed through the multiple-linear regression analysis of the peak flow data of 48 gaging stations in Los Angeles County. Comparison of the results obtained indicated that the log-Pearson Type III analysis of the stream gages in the Malibu area produced higher peak flow rates than the Regional Runoff Frequency Equations. Therefore, the ratio of the flow rates predicted by the two methods was computed at each gage. Flow rates were then computed for the remaining points in the watershed by multiplying the regional equation flow rate by the appropriate ratio. The ratio used was determined by comparing the watershed being analyzed to those analyzed by the log- Pearson Type III analysis to determine which one was most similar.

The flow rates used in this study do not reflect the substantial amount of mud and debris flows which can be generated by a burned watershed. Therefore, it should be emphasized that the results of the study do not reflect the true degree of flood and mudflow hazard to the community.

The 1-percent annual chance flood discharges used for the 1998 revision to the Agoura Hills FIS were developed by the Los Angeles County Flood Control District (Los Angeles County, Construction Drawings PM 100203, September 6, 1979 and Construction Drawings PM 7982, August 17, 1979) and Simons, Li & Associates, Inc., using Los Angeles County "Capital Flood" methodology (Simons, Li & Associates, Inc., October 7, 1992).

City of Avalon

There are no gaged streams in the Avalon watershed; therefore, regional run-off frequency equations developed by the Los Angeles County Flood Control District were used to calculate flow rates based on runoff frequency. These regional runoff frequency equations were developed through the multiple-linear regression analyses of the peak flow data of 48 stream gaging stations within the county. Runoff data from the 48 gaging stations were first analyzed to obtain peak flows of the selected recurrence intervals at the gage sites. These peak values were then regressed against a n umber of physical parameters of the drainage basins.

Two of the important parameters included in the regional runoff frequency equations are rainfall intensity and runoff coefficients.

Rainfall records maintained by the City of Avalon, Harbor Department, for the period from 1947 through 1973 were used in the rainfall analysis for this study. A log-Pearson probability distribution analysis of the rainfall records was used to arrive at the 2-percent annual chance flood, 24-hour amount. This value is 5.02 inches and is similar to rainfall in the J rainfall zone. The analysis indicated that the distribution of rainfall at the Avalon gage over a 24-hour period is similar to the J rainfall zone distribution; therefore, the J rainfall zone intensity-duration curves were used to arrive at the 2-percent annual chance flood, 1- hour duration intensity. This value is 0.75 inch per hour and was used in the regional runoff frequency equation.

The district categorized and experimentally established runoff coefficient graphs for numerous areas of homogeneous runoff characteristics. To apply the appropriate runoff coefficients for this study, it was first necessary to determine the characteristics of the watersheds tributary to Avalon.

The study contractor was provided with a Soil Conservation Survey map for the eastern end of Santa Catalina Island. The survey specifically covered the Avalon watershed area. Watershed areas were categorized by soil type, texture, permeability, effective depth, and erodibility.

Examination of the soil map indicates that the tributary watersheds are composed of medium texture topsoil of moderate to shallow effective depth, low to moderately low infiltration rates, and moderate erodibility. The runoff characteristics of these watersheds compare very closely with watersheds found on the county mainland along the Santa Monica Mountain Range. This area is described as rough, broken, and stony, nonagricultural land, and is classified as Soil Type No. 022, for which the study contractor has runoff coefficient graphs. The graph was used to obtain the runoff coefficient of 0.624 at a rainfall intensity of 2 i nches per hour. This value was used in the regional runoff frequency equations. The rest of the parameters used in the regional run-off frequency equation were obtained from topographic maps and other information on f ile with the Los Angeles County Flood Control District, and are in accordance with standard practice.

Coastal flood hazard areas in Avalon were analyzed using a co mplex hydrodynamic model which considered the effects of storm generated waves/swells and their transformation due to shoaling, refraction and frictional dissipation. Limited fetch distances preclude the City of Avalon from being directly exposed to severe storm-induced surge flooding. Locally generated storm waves combined with astronomical tide is the major cause of flooding along coastal areas in the vicinity of Avalon. Analysis of wave effects included a statistical analysis of historical local wind data to obtain the 10-, 2-, 1-, and 0.2-percent annual chance floods maximum wind magnitudes. Wave characteristics were then computed for the various wind recurrence intervals. Using the methodology cited above, the wave runup and setup elevations were calculated based on the wave characteristics. The wave runup and setup elevations were then statistically combined with the astronomical tide to yield the final coastal flooding conditions.

Wave runup elevations were used to determine flood hazard areas for sites along the open coast that are subject to direct assault by deep-water waves. Runup elevations range with location and local beach slope. Areas with ground elevations 3.0 feet or more below the 1-perecent annual chance wave runup elevation are subject to velocity hazard.

Wave setup elevations, determined on the basis of location along the coast, were used to identify flood hazard areas along bays, coves, and areas sheltered from direct action of deep-water waves. For this study, no wave setup elevations are shown.

Cities of Bellflower, Carson, Compton, Downey, Gardena, Lakewood, Long Beach (flooding from terrestrial sources only), Lynwood, Paramount, Pico Rivera, Santa Fe Springs, South Gate, Whittier

Hydrologic data for the Los Angeles River and the Rio Hondo were obtained from the USACE. The basis of the hydrologic data was HEC-1 and HEC-5 computer models. The HEC-1 model was calibrated for each subbasin using observed flow data where applicable. In addition, frequency-discharge calculations were made to compare the USACE results. The results were based on statistical analysis of stream gage data obtained from the LACFCD. The data were analyzed using the criteria in Bulletin 17- B.

The 1-Percent Annual Chance breakout hydrology for the Los Angeles River lower reach and the Rio Hondo were also obtained from the USACE. The peak values given in the LACDA report were used for hydraulic calculations in the overbank areas.

The timing of the breakouts on the left levee of the Rio Hondo at Beverly Boulevard and Stewart and Gray Road and the left levee of the Los Angeles River at Fernwood Avenue (Century Freeway) was also considered in determining the peak flow rate in the left overbank downstream of the Century Freeway. The USACE has determined that the peaks on the Rio Hondo breakouts do not occur at the same time as the peak on the Los Angeles River breakout. Therefore, downstream of the Century Freeway, the peak flow rate in the left overbank from the Rio Hondo breakouts is not combined with the peak flow rate from the breakout near the Century Freeway. Only the peak flow from the Los Angeles River breakout is used since it has a larger magnitude.

City of Burbank

Regional Runoff Frequency Equations developed by the Los Angeles County Flood Control District were used to calculate flow rates for the Burbank Western Flood Control Channel in the City of Burbank, based on runoff frequency for the ungaged flood sources. These Regional Runoff Frequency Equations were developed through the multiple-linear regression analyses of the peak flow data of 48 gaging stations operated by the Los Angeles County Flood Control District within Los Angeles County. Runoff data from these stations were first analyzed to obtain peak flows of the selected recurrence intervals at the gage sites. These peak values were then regressed against a number of physical parameters of the drainage basins.

The Los Angeles River Flood Control Channel, which traverses the city's southern corporate limits, and the Burbank Western Flood Control Channel are the only gaged streams in the Burbank study area. The 1-percent annual chance peak flow rate for the Los Angeles River Flood Control Channel was computed using the log-Pearson Type III frequency analysis, and discharges associated with this event were found to be contained within the channel within the City. One of the 48 gaging stations operated by the Los Angeles County Flood Control District within Los Angeles County is located at Tujunga Avenue on the Burbank Western Flood Control Channel. It has been operated since 1950 and has a drainage area of approximately 401 square miles. The gage records for this location were considered inaccurate for frequency analysis purposes because of the residential development that has occurred in the watershed over the past 20 years. Therefore, Regional Runoff Frequency Equations developed by the Los Angeles County Flood Control District were used to calculate flow rates based on runoff frequency, and 1-percent annual chance flood discharges were found to be contained within the channel.

The flow rates used in this study do not include the substantial amount of mud and debris flows which could be generated from a burned watershed.

For the January 20, 1999 revision, the USACE HEC-1 computer program (U.S. Department of the Army, Corps of Engineers, Hydrologic Engineering Center, September 1990) was used to establish peak discharges having recurrence intervals of 10- and 1-percent annual chance. The parameters used were developed based on site conditions and in accordance with the guidelines contained in Natural Resources Conservation Service (NRCS) (formerly the Soil Conservation Service) Technical Release No. 55, "Urban Hydrology For Small Watersheds" (U.S. Department of the Interior, 1976).

Drainage areas were delineated on U.S. Geological Survey (USGS) 7.5-minute series topographic maps at a scale of 1:24,000, with a contour interval of 40 feet (U.S. Department of the Interior, 1966, Photorevised 1972), of the area based on previous studies by the LACFCD (Los Angeles County Flood Control District, August 1982).

The NRCS dimensionless unit-hydrograph option within HEC-1 was used. Times of concentration and lag were determined using NRCS methodology and criteria. Losses were determined using the NRCS curve-number method, in accordance with Technical Release No. 55 guidelines. Land use was determined from City of Burbank mapping and field reconnaissance. A 24-hour nested balanced storm was used with precipitation values determined from statistics developed by the California DWR (California Department of Water Resources, 1986) for the Burbank Valley Pump recording rain gage. The 1-percent annual chance precipitation for this gage ranged from 0.40 inch for 5 minutes to 1.51 inches for 1 hour to 7.44 inches for 24 hours.

Flows were routed and combined using the channel-storage (modified-Puts) and Muskingum-Cunge channel-routing methods within the HEC-1 model.Discharges were determined for 10- and 1- percent annual chance return periods. The 10-percent annual chance discharges were compared with discharges determined by the LACFCD and loss rates were adjusted so the discharges would agree within 1 to 5 percent. The 1-percent annual chance discharges within the channel are limited by channel capacity.

City of Culver City

The gaged streams tributary to Culver City are the Ballona Creek Channel and the Sawtelle-Westwood Storm Drain Channel. The 1-percent annual chance peak flow rates for these streams were computed using the log-Pearson Type III frequency analysis. The USACE, Los Angeles District, performed the analysis of Ballona Creek Channel. The gage, located at Sawtelle Boulevard, has been operated since 1927 and records flows from a drainage area of approximately 89 square miles. The flow rates were modified due to cultural changes in the watershed (i.e., agricultural to urbanized). The study contractor performed frequency analysis for the gage on Sawtelle-Westwood Channel. The gage, located at Culver Boulevard, has been operated since 1951 and records flows from a drainage area of approximately 23 square miles. Benedict Canyon Channel is completely underground through Culver City.

The remaining streams tributary to Culver City are ungaged. Therefore, regional runoff frequency equations developed by the Los Angeles County Flood Control District were used to calculate flow rates based on runoff frequency. These regional runoff frequency equations were developed through the multiple linear regression analyses of the peak flow data of 48 stream gaging stations within Los Angeles County. Runoff data from the 48 gaging stations were first analyzed to obtain peak flows of the selected recurrence intervals at the gage sites. These peak values were then regressed against a number of physical parameters of the drainage basins.

As a result of these analyses, it was determined that the 1-percent annual chance flood discharges for Ballona Creek Channel, Sawtelle-Westwood Storm Drain Channel, Benedict Canyon Channel, and Centinela Creek Channel were contained in the channels except for Ballona Creek Channel in the vicinity of the northeast corporate limits near Washington Boulevard. The 0.2-percent annual chance flood event was not studied for channel segments that contain the 1-percent annual chance flood peak discharge.

City of La Mirada

There are no gaged streams in the watersheds tributary to La Mirada Creek; therefore, regional runoff frequency equations developed by the study contractor were used to calculate flow rates based on runoff frequency. These regional runoff frequency equations were developed through the multiple-linear regression analyses of the peak flow data of 48 stream gaging stations within Los Angeles County. Runoff data from the 48 gaging stations were first analyzed to obtain peak flows of the selected recurrence intervals at the gage sites. These peak values were then regressed against a number of physical parameters of the drainage basins.

City of Lancaster

The USACE, Los Angeles District, developed discharge-frequency relationships for streams in the Antelope Valley and the City of Lancaster. The 1-percent annual chance peak flow rates for Little Rock Creek and Big Rock Creek were computed using log-Pearson Type III frequency analyses. The analysis for Little Rock Creek was based on the stream gage located at Little Rock Reservoir, south of the City of Palmdale, which has been in operation since 1931 and records streamflow from a drainage area of approximately 49 square miles. The gage located at the mouth of Big Rock Creek, southwest of the City of Palmdale, has been in operation since 1923 and records flows from a drainage area of approximately 23 square miles.

Amargosa Creek, Amargosa Creek Tributary, and Portal Ridge Wash are ungaged. Therefore, discharge- frequency curves were developed by the USACE from the Little Rock Creek and Big Rock Creek frequency curves. An average of the two curves was developed using standard deviation and average skew coefficient of the two gages. The USACE Standard Project Flood peak discharge at the concentration points was used as the basis for transposing the frequency curves to ungaged streams originating in the San Gabriel Mountains.

For the summer peak discharges in the Antelope Valley desert region, the USACE determined from the gages of nine streams that the major events were independent with relatively short gage records. Therefore, the peak discharges recorded at each of the gages were considered collectively as a single flood record representative of the region. To develop a summer storm discharge-frequency curve at any engaged location, the Standard Project Flood was used as the basis for transposing the frequency curves.

The Los Angeles County Flood Control District employed the USACE study as a data base to develop yield versus area curves for the 10-, 2-, 1-, and 0.2-percent annual chance flow rates for the concentration points. These curves were used to determine the peak flow rates for intermediate points along the major watercourses and for adjacent watersheds.

City of Long Beach (Coastal Flooding only; terrestrial flooding covered under Cities of Bellflower, et al., above)

Coastal flooding in the City of Long Beach, as analyzed for the original study of the City, originates from San Pedro and Alamitos Bays. This flooding is attributed to the following mechanisms:

1. Swell runup from intense offshore winter storms in the Pacific
2. Tsunamis from the Aleutian-Alaskan and Peru-Chile Trenches
3. Runup from wind waves generated by landfalling storms
4. Swell runup from waves generated off Baja California by tropical cyclones
5. Effects of landfalling tropical cyclones

The influence of the astronomical tides on coastal flooding is also incorporated in each of the previously mentioned mechanisms. A flood producing event from any of these mechanisms is considered to occur with a random phase of the astronomical tide. Each of these mechanisms is considered to act alone, so that the joint occurrence of any combination of the above mechanisms in a flooding event is considered to be irrelevant to the determination of flood elevations with return periods of less than 0.02-percent annual chance.

For each mechanism, the frequency of occurrence of causative events, as well as the probability distribution of flood elevations at a given location due to the ensemble of events, were determined using methods discussed in "Methodology for Coastal Flooding in Southern California." A brief outline follows.

Winter Swell

The statistics of flooding due to winter swell runup were determined using input data provided by the Navy Fleet Numerical Weather Center (FNWC). These input data consist of daily values of swell heights, periods, and directions at three deep water locations beyond the continental shelf bordering the study area. The data are inclusive from 1951 to 1974, and were computed by FNWC using input from ship observations, meteorological stations, and synoptic surface meteorological charts of the Pacific Ocean. For the original study, the incoming swells provided by FNWC were classified into 12 direction sectors of 10 degrees band width each. (Exposure of the study area to winter swells was confined to a 120 degree band, from directions 220° to 340°T). Within each sector, 10 days of swell height and period values were selected from the 24 years of FNWC data to represent extreme flood producing days. The selection criteria were guided by Hunts formula for runup. The 120 days at each of the three deepwater stations were merged to obtain a master list of 161 extreme runup producing days. For each of 161 days, the input swell provided by FNWC was refracted across the continental shelf and converted to runup at selected locations in the study area. The techniques used and data required are described in Section 3.2. Of the 161 days, a number of groups of consecutive days could be identified.

Each such group of days is considered to represent one event only; the largest runup from each group of days was selected as the maximum runup for that event. As a result of refraction and island sheltering effects, a number of the input swells produced no significant runup at certain locations. Therefore, the number of extreme runup events is less than 161. The average number of events in the study area is approximately 40. For each location in the study area, the runup for the extreme events were fitted to a Weibull distribution to obtain a probability distribution of runup from winter swell. The Weibull distribution was found to be best suited for representing runup statistics. Because extreme winter swell runup lasts for at least one day, the maximum runup must be considered to coexist with the maximum high tide.

Regarding the extreme runup values as a statistical sample only, the influence of the astronomical tides was included by convolving the probability distribution of runup with the probability distribution of daily "high tides. The latter was obtained from standard tide prediction procedures using the harmonic constants at the nearest available tide gage for which such data exists as supplied by the Tidal Prediction Branch of the National Oceanic and Atmospheric Administration. At each location, the frequency of occurrence of extreme events is determined by the number of runup values used in the Weibull curve fit. The number of years over which these occur is 24. The product of the frequency occurrence with the complement of cumulative probability distribution of the runup-plus-tide (convolved) distribution gives the exceedence frequency curve for flood elevations due to winter swell runup.

Tsunamis

Elevation-frequency curves for tsunami flooding were obtained from information supplied by the USACE's Waterways Experiment Station (WES). The use of the results of the WES study were directed by FEMA.

In the WES study, the statistics of tsunami elevations along the coastline were derived by synthesizing data on tsunami source intensities, source dimensions, and frequencies of occurrence along the Aleutian- Alaskan and Peru-Chile Trenches. As a result, 75 different tsunamis, each with a known frequency of occurrence, were generated and propagated across the Pacific Ocean using a numerical hydrodynamic model of tsunamis. At a number of locations in the study area, these 75 tsunami time signatures were each added to the tidal time signature at the nearest tide gage location for which harmonic constants for tide computations are available. One year of tidal signature was generated from the harmonic constants. A given tsunami signature was then combined with the tide signature and the maximum of tsunami plus tide for the combination recorded. To simulate the occurrence of the tsunami at random phases of the tide, the tsunami signature was repeatedly combined to the tide signature starting at random phases over the entire year of the tide signature. Each combination produces a maximum tsunami-plus tide elevation with a frequency of occurrence equal to the frequency of occurrence of the particular tsunami signature used, divided by the total number of such combinations for that particular tsunami. The process was repeated for all 75 tsunamis and the elevation frequency curve for tsunami flooding was thus established.

Wind Waves From Landfalling Storms

The source of data for wind waves is the same as that for winter swell, the FNWC (1951 through 1974) data. The stations for which daily height, period, and direction data are available are also the same as for winter swells. The FNWC wind-wave data are directly correlated to local wind speeds. For obtaining runup statistics, the FNWC daily wave data were converted to daily runup data using the method outlined in Section 3.2. The daily runup data were then fitted to a Weibull distribution and convolved with the tide in the same manner as for winter swells.

Tropical Cyclone Swell

Runup from swell generated by tropical cyclones off Baja California was computed using the techniques discussed in Section 3.2. To establish the statistics of hurricane swell runup, the following procedure was used. Data concerning tropical cyclone tracks were obtained from the National Climatic Center (NCC). The data comprise 12-hourly positions of eastern North Pacific tropical cyclones from 1949 to 1974. This was supplemented by data on tropical cyclone tracks from the period 1975 to 1978, as reported in the Monthly Weather Review.

Besides position data, storm intensities at each 12-hourly position are also given. The intensity classifications are based on estimated maximum wind speeds. The intensity categories are tropical depression (less than 35 knot winds), tropical storm (less than 65 knot winds), and hurricane (at least 65 knot winds). Storms with tropical depression status were considered to generate negligible swell and omitted from this study. Data on actual maximum wind speeds were available from the NCC only from 1973 to 1977. These were used as the basis for obtaining values to represent maximum wind speeds from each of the two intensity classifications associated with the track data. Data on storm radii were derived from North American Surface Weather Charts by analysis of pressure fields of tropical cyclones off Baja California. These were used to define typical radius of maximum winds for each of two relevant intensity classes. For each tropical cyclone between 1949 and 1918, the hurricane wind waves were computed using the mean radius and maximum wind speeds established for each intensity class along with the track data. The swell and resultant runup were computed using the techniques described in Section 3.2. For each tropical cyclone and each location of interest in the study area, a time history of swell runup was determined. These were added to time histories of the local astronomical tide in a procedure analogous to that used in determining tsunami plus tide effects. The exceedence frequencies of tropical cyclone swell runup were computed in a manner similar to that used for tsunamis.

Landfalling Tropical Cyclones

The frequency of landfalling tropical cyclones in southern California is extremely low. During those years covered by the NCC tape of eastern North Pacific tropical cyclones (1949 to 1974), no tropical cyclone hit southern California. A longer period of record was used to estimate the frequency of an event such as the Long Beach 1939 storm. A study by Pyke was used to compile a list of landfalling tropical cyclones along the coast of southern California. The study was a r esult of extensive investigation of historical records such as precipitation and other weather and meteorological data. The study spanned the period from 1889 to 1977 and showed only 5 or 6 identifiable landfalling tropical cyclones, of which the 1939 Long Beach event was the strongest, and only one in the tropical storm category. The others were all weak tropical depressions (with maximum winds of less than 35 knots). The low frequency event, once in 105 y ears over approximately 360 m iles of coastline, coupled with an impact diameter of approximately 60 miles, implies that for any given location, the return period of a landfalling tropical cyclone is about 600 years. Therefore, landfalling tropical cyclones were not considered in the original study.

At each location within the study area, the exceedence frequencies at a given elevation due to the various flood producing mechanisms were summed to give the total exceedence frequency at the flood elevation.

City of Los Angeles

The following streams within the City of Los Angeles have Los Angeles County Flood Control District records sufficient for frequency analysis purposes: Aliso Creek, Station F 1 52B-R, at Nordhoff Street; Big Tujunga Wash, Station F213-R, located 2 m iles above the mouth of the canyon; Los Angeles River, Station F300-R, located at Tujunga Avenue and Station F57C-R, located at the confluence with Arroyo Seco; Sawtelle Channel, Station F301-R, located 141 feet upstream of Culver Boulevard; Ballona Creek, Station F38C-R, located 530 feet upstream of Sawtelle Boulevard; and Compton Creek, Station F37B-R, located at Greenleaf Boulevard. The 1-percent annual chance frequency peak flow rates for these streams were computed using the log-Pearson Type III frequency analyses.

The remaining streams in the Los Angeles study area are ungaged; therefore, regional runoff frequency equations developed by the Los Angeles County Flood Control District were used to calculate flow rates based on runoff frequency. These regional runoff frequency equations were developed through the multiple-linear regression analyses of the peak flow data of 48 stream-gaging stations within Los Angeles County. Runoff data from the 48 gaging stations were first analyzed to obtain peak flows of the selected recurrence intervals at the gage sites. These peak values were then regressed against a n umber of physical parameters of the drainage basins.

The flow rates used in the Los Angeles study do not include the substantial amount of mud and debris flows that could be generated from a burned watershed. Therefore, it should be-emphasized that the results of this study may not reflect the true degree of flood hazard in the community.

Coastal flood hazard areas subject to inundation by the Pacific Ocean were determined on the basis of water-surface elevations established from regression relations defined by Thomas. These regression relations were defined as a practical method for establishing inundation elevations at any site along the southern California mainland coast. They were defined through analysis of water-surface elevations established for 125 locations in a complex and comprehensive model study by Tetra Tech, Inc. The regression relations establish wave runup and wave setup elevations that have 10-, 1-, and 0.02-percent chances of occurring in any year and are sometimes referred to as the 10-, 100-, and 500-year flood events, respectively.

Wave runup elevations were used to determine flood hazard areas for sites along the open coast that are subject to direct assault by deep-water waves. Runup elevations range with location and local beach slope and were computed at 0.5-mile intervals, or more frequency in areas where the beach profile changes significantly over short distances. Areas with ground elevations 3.0 feet or more below the 1- percent annual chance wave runup elevation are subject to velocity hazard.

Wave setup elevations determined from the regression equations on the basis of location along the coast were used to identify flood hazard areas along bays, coves, and areas sheltered from direct action of deep-water waves.

City of Montebello

The only gaged stream in the Montebello study area is located on Drainage District Improvement No. 23, upstream of the Rio Hondo Channel. In the original study, this gage was found unsatisfactory for frequency analysis purposes due to diversions in the watershed, substantial residential development, and the effect of backwater from the Rio Hondo Channel. Therefore, Regional Runoff Frequency Equations developed by the LACFCD were used to calculate flow rates based on runoff frequency. These Regional Runoff Frequency Equations were developed through the multiple-linear regression analyses of the peak flow data of 48 stream gaging stations within Los Angeles County. Runoff data from the 48 gaging stations were first analyzed by obtaining peak flows of the selected recurrence intervals at the gage sites. These peak values were then regressed against a number of physical parameters of the drainage basins.

The flow rates used in the original study do not include the substantial amount of mud and debris flow that could be generated from a burned watershed. Therefore, it should be emphasized that the results of the study do not reflect the mud and debris flow hazard in the community.

For the areas of the City of Montebello affected by the Los Angeles River/Rio Hondo system, hydrology was generated using the methodologies outlined in the section on the Cities of Bellflower, et al., above.

The timing of the breakouts on the left levee of the Rio Hondo at Beverly Boulevard and Stewart and Gray Road and the left levee of the Los Angeles

River at Fernwood Avenue (Century Freeway) was also considered in determining the peak flow rate in the left overbank downstream of the Century Freeway. The USACE has determined that the peaks on the Rio Hondo breakouts do not occur at the same time as the peak on the Los Angeles River breakout. Therefore, downstream of the Century Freeway, the peak flow rate in the left overbank from the Rio Hondo breakouts is not combined with the peak flow rate from the breakout near the Century Freeway. Only the peak flow from the Los Angeles River breakout is used since it has a larger magnitude.

City of Palmdale

Discharge-frequency relationships for the City of Palmdale were developed by the USACE, Los Angeles District. In their study, the 1-percent annual chance peak flow rates for Little Rock Wash and Big Rock Wash were computed using the log-Pearson Type III frequency analysis. The gage located at Little Rock Reservoir, south of Palmdale, has operated since 1931 and records reflect flow from a drainage area of approximately 48 square miles. The gage located at the mouth of Big Rock Wash, southwest, has been operated since 1923 and records flows from a drainage area of approximately 23 square miles.

Amargosa Creek, Amargosa Creek Tributary, Anaverde Creek, and Anaverde Creek Tributary are ungaged. Therefore, discharge-frequency curves were developed by the USACE from Little Rock Wash and Big Rock Wash curves. An average of the two curves was developed using the standard deviation and average skew coefficient of the two gages. The USACE Standard Project Flood peak discharge at the concentration points was used as the basis for transposing the frequency curves to ungaged streams.

For the summer peak discharges in the Antelope Valley desert region, the USACE determined from gages on nine streams that the major events were independent with relatively short records. Therefore, the peak discharges were considered collectively as a single flood record representative of the region. To develop a summer storm discharge-frequency curve at any ungaged location, the Standard Project Flood was used as the basis for transposing the frequency curves.

The LACFCD used the USACE study as a data base to develop yield-versus-area curves for the 10-, 2-, 1-, and 0.2-percent annual chance flow rates for the concentration points. These curves were used to determine the peak flow rates for intermediate points along the major watercourses and for adjacent watersheds.

For the March 30, 1998 revision, the 1-percent annual chance discharges were calculated using regional regression equations developed by FEMA. The FEMA regression equation for the 1-percent annual chance discharges is:

$$Q = 660\,A^{0.62}$$

where A is the total contributing watershed in square miles.

This equation was developed from data for 41 g aging stations in the South Lohonton-Colorado Desert (SLCD) region, as defined in the U.S. Geological Survey (USGS) Water Resources Investigations 77-2 1, "Magnitude and Frequency of Floods in California" (U.S. Department of the Interior, Geological Survey, June 1977). Anaverde Creek is in the SLCD region. The above equation is applicable for estimating flood discharges for Anaverde Creek because three gaging stations in the vicinity of Anaverde Creek were included in the regression analysis.

City of Redondo Beach

The watersheds of Redondo Beach are relatively small and there are no g aged streams in the study area. Therefore, the 1-percent annual chance peak flow rates were determined by use of the Los Angeles County Flood Control District Primary Regional Run-Off Frequency Equation for ungaged streams. Where 1-percent annual chance flood discharges exceeded the drain capacities, a field review and calculations of street capacities were made. At several locations, localized sumps were found where the existing drains do not adequately convey the 1-Percent Annual Chance flows or where drains do no t exist. The excess flows create ponding conditions and the Los Angeles County Flood Control District Regional Normalized Hydrograph Equations were used to determine the volumes of ponding water. Where necessary, the volumes were reduced by reservoir routing the flows through the ponding areas.

The principal source of coastal flooding in Redondo Beach is from the Pacific Ocean and its landward intrusions such as Alamitos and Marina del Rey.

Coastal flooding is attributed to the following mechanisms:

1. Swell runup from intense offshore winter storms in the Pacific
2. Tsunamis from the Aleutian-Alaskan and Peru-Chile trenches
3. Runup from wind waves generated by landfalling storms
4. Swell runup from waves generated off Baja California by tropical cyclones
5. Effects of landfalling tropical cyclones

The influence of the astronomical tides on coastal flooding is also incorporated in each of the above mechanisms. A flood-producing event from any of the above mechanisms is considered to occur with a random phase of the astronomical tide. Each of the above mechanisms is considered to act alone. This is the joint occurrence of any combination of the above mechanisms in a flooding event is considered to be irrelevant to the determination of flood elevations with return periods of less than 0.2-percent annual chance.

For each mechanism, the frequency of occurrence of causative events as well as the probability distribution of flood elevations at a given location due to the ensemble of events was determined according to the methodology given in "Methodology for Coastal Flooding in Southern California." A brief outline of it is presented in the section on the City of Los Angeles, above.

City of Santa Clarita

Much of the hydrologic data used in this FIS study for the City of Santa Clarita was taken from a report prepared by the USACE. For watersheds greater than 20 square miles, the USACE formula for the geometric mean flood was used to predict 1-percent annual chance peak flow rates. For drainage areas less than 20 square miles, this formula was modified slightly to yield runoff values more closely related to observed values and engineering judgment. This modification was reviewed by the Los Angeles District Office of the USACE.

City of Santa Fe Springs

Floods impacting the City of Santa Fe Springs are generated from watersheds on the southwesterly side of the Puente Hills, located to the north of Santa Fe Springs. The only gaged streams in the Santa Fe Springs study area are the San Gabriel River and Coyote Creek (both located outside the corporate limits). The 1-percent annual chance peak flow rates for these streams were computed using log-Pearson Type III frequency analyses.

The analysis of the San Gabriel River is based on the Los Angeles County Flood Control District Stream Gage No. F 262E-R, which is located approximately 1400 feet upstream of Florence Avenue near the western corporate limits. This gage has a drainage area of 216 square miles and 43 years of record. However, only the past 16 years of record were used for the frequency analysis, and they were compiled following completion of the Santa Fe and Whittier Narrows Dams, which are major flood control facilities located 15 miles and 5 miles upstream of the gage, respectively. The 1-percent annual chance peak discharge for the San Gabriel River at Florence Avenue was determined to be 13,000 cubic feet per second (cfs). The design capacity of the channel at this location is 19,000 cfs. Therefore, it was determined that no flooding from the San Gabriel River affects the city. The analysis for Coyote Creek - North Fork was based on the Los Angeles County Flood Control District Stream Gage No. 3208, which is located on the main branch of Coyote Creek at Centralia Street. This gage is located 4 miles downstream of Santa Fe Springs, has a drainage area of 110 square miles, and has 34 years of record. The 1-percent annual chance peak discharge is approximately 10,000 cfs as compared to design capacity of 42,000 cfs for Coyote Creek downstream of the City of Santa Fe Springs. It was also determined that no flooding from Coyote Creek and Coyote Creek - North Fork affect the city.

The remaining streams in the Santa Fe Springs study area are ungaged; therefore, regional runoff- frequency equations developed by the Los Angeles County Flood Control District were used to calculate flow rates based on runoff frequency. These regional runoff-frequency equations were developed through the multiple-linear regression analyses of the peak flow data of 48 stream gaging stations within Los Angeles County. Runoff data from the 48 gaging stations were first analyzed to obtain peak flows of the selected recurrence intervals at the gage sites. These peak values were then regressed against a number of physical parameters of the drainage basins.

City of Torrance

Flood conveyance channels within the City of Torrance are relatively small, and stormflows either accumulate in numerous small sumps, drain directly into the Pacific Ocean or are tributary to Dominguez Channel. Dominquez Channel is the only gaged watershed in the City of Torrance. However, the gage has an insufficient length of record for frequency analysis purposes. Dominquez Channel was analyzed through a comparison with Compton Creek, a gaged stream in an adjacent watershed outside of the corporate limits with similar hydrologic and hydraulic characteristics. The 1-percent annual chance peak flow for Compton Creek was computed using the log-Pearson Type III frequency analysis method. The ratio of the 1-percent annual chance peak flow for Compton Creek to the peak flow recorded in Compton Creek during the major storm of 1969 was applied to the 1969 peak flow in Dominguez Channel to obtain an approximate 1-percent annual chance peak flow for Dominguez Channel. This peak flow was estimated to be 12,500 cubic feet per second (cfs). Because the available channel capacity is 17,000 cfs, it was concluded that Dominguez Channel has ample capacity to convey the 1-percent annual chance discharge, and no further analysis was necessary.

The remaining watersheds tributaries to the City of Torrance are ungaged. Therefore, regional runoff frequency equations developed by the Los Angeles County Flood Control District were used to calculate flow rates based on runoff frequency. These regional runoff frequency equations were developed through the multiple-linear regression analyses of the peak flow data of 48 stream gaging stations within Los Angeles County. Runoff data from the 48 gaging stations were first analyzed to obtain peak flows of the selected recurrence intervals at the gage sites. These peak values were then regressed against a number of physical parameters of the drainage basins.

City of West Hollywood

Regional runoff frequency equations developed by the Los Angeles County Flood Control District were used to calculate peak discharges for the City of West Hollywood.

City of Whittier

There are no gaged streams in the watersheds draining the City of Whittier; therefore, Regional Runoff Frequency Equations developed by the Los Angeles County Flood Control District were used to calculate flow rates based on runoff frequency. These Regional Runoff Frequency Equations were developed through the multiple-linear regression analyses of the peak-flow data of 48 gaging stations operated by the Los Angeles County Flood Control District within Los Angeles County. Runoff data from these stations were first analyzed in order to obtain peak flows of the selected recurrence intervals at the gage sites. These peak values were then regressed against a number of physical parameters of the drainage basins.

The flow rates used in this study do not include the substantial amount of mud and debris flows which could be generated from a burned watershed. Therefore, it should be emphasized that the study does not reflect this type of flood hazard in the community.

Peak inflow volumes determined for the ponding areas studied by detailed methods in Torrance are shown in Table 9, "Summary of Inflow Volumes."

Table 9: Summary of Inflow Volumes

Flooding Source and location	Drainage Area (Sq. miles)	Peak Inflows (cfs)			
		10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
SURFACE RUNOFF – DEEP PONDING AREA					
Southwest of the intersection of Carson Street and Madrona Avenue	0.3	50	110	140	210
At intersection of Doris Way and Reese Road	0.5	160	350	450	700
SURFACE RUNOFF – PONDING AREA					
At intersection of Anza Avenue and Spencer Street	0.1	10	20	25	40
Northwest of Sepulveda Boulevard and Madrona	0.3	60	140	180	280
At intersection of California Street and Alaska Avenue	0.7	190	250	270	330
At intersection of Amsler Street and Dormont Avenue	6.2	1,330	2,960	3,760	5,880

3.2 Hydraulic Analyses

Analyses of the hydraulic characteristics of flooding from the sources studied were carried out to provide estimates of the elevations of floods of the selected recurrence intervals. Users should be aware that flood elevations shown on the FIRM represent rounded whole-foot elevations and may not exactly reflect the elevations shown on the Flood Profiles or in the Floodway Data tables in the FIS report. Flood elevations shown on the FIRM are primarily intended for flood insurance rating purposes. For construction and/or floodplain management purposes, users are cautioned to use the flood elevation data presented in this FIS in conjunction with the data shown on the FIRM.

The elevations have been determined for the 10-, 2-, 1-, and 0.2-percent annual chance floods for the flooding sources studied by detailed methods.

Cross sections were determined from topographic maps and field surveys. All bridges, dams, and culverts were field surveyed to obtain elevation data and structural geometry. All topographic mapping used to determine cross sections are referenced in Section 4.1.

Locations of selected cross sections used in the hydraulic analyses are shown on the Flood Profiles (Exhibit 1). For stream segments for which a floodway was computed (Section 4.2), selected cross section locations are also shown on the FIRM (Exhibit 2).

Roughness coefficients (Manning's "n") were chosen by engineering judgment and based on field observation of the channel and floodplain areas

Table 10, “Summary of Roughness Coefficients,” contains the channel and overbank "n" values for the streams studied by detailed methods.

Table 10: Summary of Roughness Coefficients

Flooding Source	Channel	Overbanks
Amargosa Creek	0.04	0.04
Anaverde Creek	0.04	0.04
Avalon Canyon	0.030 – 0.050	0.030 – 0.050
Big Rock Wash	0.05	0.05
Cheseboro Creek	0.05	0.03
Cold Creek	0.05	0.03
Dark Canyon	0.05	0.03
Dry Canyon	0.05 – 0.06	0.03
Escondido Canyon	0.05	0.03
Flow along Empire Avenue	0.014 – 0.050	0.014 – 0.050
Flowline No. 1	0.030	0.030
Garapito Creek	0.05	0.03
Hacienda Creek	0.06	0.03
Kegal Canyon	0.035 – 0.065	0.035 – 0.065
La Mirada Creek	0.025 – 0.030	0.025 – 0.030
Lake Street Overflow	0.014 – 0.050	0.014 – 0.050
Las Flores Canyon	0.05	0.03
Las Virgenes Creek	0.012-0.040	0.050-0.130
Liberty Canyon	0.05	0.03
Lindero Canyon above Confluence with Medea Creek	0.05	0.03
Lindero Canyon above Spillway above Lake Lindero	0.05	0.03
Little Rock Wash – Profile A	0.05	0.03
Little Rock Wash – Profile B	0.05	0.03
Little Rock Wash – Profile C	0.05	0.03
Lobo Canyon	0.05	0.03
Lockheed Drain Channel	0.014 – 0.050	0.014 – 0.050
Lopez Canyon Channel	0.06	0.03

Table 10. Summary of Roughness Coefficients (continued)

Flooding Source	Channel	Overbanks
Los Angeles River Left Overbank Path 2	0.05 – 0.15	0.016
Los Angeles River Right Overbank Path 1	0.05 – 0.15	0.016
Los Angeles River Right Overbank Path 2	0.05 – 0.15	0.016
Malibu Creek	0.05	0.03
Medea Creek	0.05	0.03
Medea Creek (above Ventura Freeway)	0.03	0.05
Mill Creek	0.06	0.03
North Overflow	0.014 – 0.050	0.014 – 0.050
Old Topanga Canyon	0.05	0.03
Overflow Area of Lockheed Drain Channel	0.030 – 0.040	0.030 – 0.040
Overflow Area of Lockheed Storm Drain	0.014 – 0.050	0.014 – 0.050
Palo Comando Creek	0.05	0.03
Ramirez Canyon	0.05	0.03
Rio Honda Left Overbank Path 3	0.05 – 0.15	0.05 – 0.15
Rio Honda Left Overbank Path 5	0.05 – 0.15	0.05 – 0.15
Rio Honda Left Overbank Path 6	0.05 – 0.15	0.05 – 0.15
Rustic Canyon	0.035 – 0.065	0.035 – 0.065
Santa Maria Canyon	0.05	0.03
Stokes Canyon	0.05	0.03
Topanga Canyon	0.05	0.03
Trancas Creek	0.05	0.03
Triunfo Creek	0.05	0.03
Unnamed Canyon (Serra Retreat Area)	0.05	0.03
Unnamed Stream Main Reach	0.015-0.04	0.015-0.12
Unnamed Stream Tributary 1	0.015-0.045	0.015-0.11
Unnamed Stream Tributary 2	0.015-0.045	0.015-0.11
Upper Los Angeles River Left Overbank	0.05 – 0.15	0.05 – 0.15
Weldon Canyon	0.035 – 0.065	0.035 – 0.065
Zuma Canyon	0.05	0.03

The hydraulic analyses for this study were based on unobstructed flow. The flood elevations shown on the Flood Profiles (Exhibit 1) are thus considered valid only if hydraulic structures remain unobstructed, operate properly, and do not fail.

3.2.1 Methods for Flooding Sources with New or Revised Analyses in Current Study

In the City of Calabasas, hydraulic cross section geometries for Las Virgenes Creek were obtained from LIDAR data, and supplemented with surveyed hydraulic data at various locations throughout the detailed studied reach. All hydraulic structures were field measured to obtain elevation data and structural geometry. Water-surface elevations for the 10-, 2-, 1-, and 0.2-percent annual chance exceedance discharges were computed for the detailed method models with the unsteady analysis feature of the Army Corps of Engineers' HEC-RAS computer program, version 4.1.

The Manning's n-value chosen for the channel ranges from 0.012-0.04. An overbank n-value range of 0.05-0.130 was selected to represent the varying topography in the headwaters near the City of Calabasas, downstream through Malibu Creek State Park.

The downstream boundary condition chosen for Las Virgenes Creek was normal depth. The upstream boundary condition utilized known water surface elevations from LOMC 99-09-334P-060749 at Las Virgenes Road. ArcGIS version 9.3 SP1 was utilized for the refinement of cross-section placement, Manning's n-value selection, and reach length development.

For Unnamed Stream Main Reach, Unnamed Stream Tributary 1, and Unnamed Stream Tributary 2 in Palos Verdes Estates, hydraulic models were developed in HEC-RAS 3.1.3. Flow capacities were estimated for the storm sewer networks using Manning's equation. Surface flows to delineate the floodplains were calculated by subtracting storm drain pipe capacities from the overall peak discharges determined in the hydrologic analysis. Based on the topography and discussions with the City of Palos Verdes Estates, it was determined that there are potential overflow locations at roadway crossings of the study streams. Flows from the study reaches at these roadway crossings have the potential to be diverted as sheet flow along the roadways due to the downward gradient away from stream crossings. Except for the Via Coronel Road crossing of the Main Reach, the diverted flows were not modeled separately as part of this Flood Insurance Study.

At the Via Coronel Road crossing of the Unnamed Stream Main Reach, flow from the Main Reach would be diverted north towards Unnamed Stream Tributary 1 along Via Coronel Road. The flows that would be diverted were calculated using the FlowMaster software from Bentley Systems, Inc. The diverted flows from Unnamed Stream Main Reach were added to the surface flows of Unnamed Stream Tributary 1.

3.2.2 Methods for Flooding Sources Incorporated from Previous Studies

Los Angeles County Unincorporated and Incorporated Areas

Analyses of the hydraulic characteristics of flooding from the sources studied by enhanced approximate and approximate methods were carried out to provide estimates of the elevations of floods of the selected recurrence intervals.

Water-surface profiles were computed for enhanced approximate and approximate study streams through the use of the USACE HEC-RAS version 3.1.2 computer program (Reference 11). Water-surface profiles were produced for the 1-percent-annual-chance storms for enhanced approximate and approximate studies.

The enhanced approximate and approximate study methodology used Watershed Information System (WISE) (Reference 12) as a preprocessor to HEC-RAS. Tools within WISE allowed the engineer to verify that the cross section data was acceptable. The WISE program was used to generate the input data file for HEC-RAS. Then HEC-RAS was used to determine the flood elevation at each cross section of the modeled stream. No floodway was calculated for streams studied by approximate methods.

Santa Clarita and Antelope Valley

Preliminary flood elevations were determined by routing peak discharges through the county using the boundaries of the alluvial fans, historical records, and field reviews. Topographic and cross section data were compiled from existing topographic maps and from topographic maps prepared by the County Engineer for use in the Antelope Valley Flood Study. Features that cause changing flow depths, such as changing ground slope or obstructions, were considered. In all cases, the changes in flow depth caused by these features were deemed to be insignificant, and backwater calculations were not used. Roughness coefficients (Manning's "n") for overland flow conditions were estimated by field inspection of the areas under investigation. The Manning's "n" values ranged from 0.03 in the channels to 0.06 in the overbanks.

The preliminary flood elevations were field reviewed for verification of actual field conditions. Features such as local obstructions or depressions which would affect flood elevations or depths were noted, and flood elevations were revised accordingly, based on engineering judgment. Average depths of flooding were assigned based on standard normal-depth calculations through irregular cross sections. In many instances, the assigned average depth is not representative of the true degree of flood hazard. This occurs when average depths are based on a wide cross section which encompasses one or more low-flow drainage courses. The actual depth of flooding and, consequently, the true flood hazard will be greater adjacent to the drainage course. In some locations in the Santa Clarita Valley, the low-flow drainage course has been designated Zone A to reflect both the more severe hazard and that no development will take place. The adjacent flood plain is then given a shallow flooding designation based on average depth across the entire cross section.

Water-surface profiles were not prepared because the 1-percent annual chance flooding in the Antelope and Santa Clarita Valleys are not readily associated with channel flooding and flood profiles. Therefore, flooding limits were established through the use of available topography and field reviews.

Flood elevations for flooding sources in areas of little existing development and low potential for future development were determined by approximate methods based on Flood Hazard Boundary Maps, field reviews, and historical records.

Malibu Area

Flooding sources in the Malibu area typically are well-incised streams with relatively high velocities. Flood profiles have been prepared for all flooding sources studied in detail except for the downstream portion of Malibu Creek. In this instance, shallow flooding designations were assigned in. accordance with FEMA criteria.

Peak discharges were routed through the Malibu area considering the capacities of existing flood control systems. Capacities of these systems were obtained from design records or were computed using Manning's Equation. Topographic and cross section data were compiled from existing topographic maps and field surveys. Features which cause change in flow depths, such a changing ground slope or obstructions, were considered in determining water-surface elevations. Roughness coefficients (Manning's "n") were estimated by field inspection of the areas under investigation. Manning's "n" values ranged from 0.03 in the channels to 0.05 in the overbanks.

Los Angeles Basin

The pockets of unincorporated territory within Los Angeles County were analyzed with the various city Flood Insurance Studies on a drainage-area basis. Where applicable, flood profiles were prepared using the same procedure as for the Malibu area of the study. With the exception of Kagel Canyon Channel, Mill Creek, Lopez Canyon Channel, and Hacienda Creek, most flooding in these areas consists of shallow flooding in developed areas. Flow depths for shallow flooding areas were calculated using available topographic maps, street plan data, and field surveys. The flow depths were determined using Manning's Equation based on normal-depth assumptions. Features such as changing ground slope or obstructions were considered.

Because the effectiveness of the calculated cross sections is reduced by the presence of obstructions such as buildings or walls, a "wetted perimeter reduction factor" was used in heavily developed areas. This factor is a measure of the percentage of blockage across the cross sectional area and has the effect of reducing the flow-carrying capacity of the cross section. This has the effect of raising the calculated water-surface elevation. Manning's "n" values for Kagel Canyon Channel, Mill Creek, Lopez Canyon Channel, and Hacienda Creek ranged from 0.03 in the channels to 0.06 in the overbanks. For shallow flooding areas, a Manning's "n" value of 0.03 was used.

Throughout the county, ponding conditions and reservoirs were analyzed using the Los Angeles County Flood Control District Regional Normalized Hydrograph Equation. This equation determines the volume of water generated by 1-percent annual chance flood discharges. Where necessary, the volumes were reduced by reservoir routing flood flows through ponded areas.

Starting water-surface elevations used in the study were determined from normal-depth calculations adjusted to field conditions.

Locations of selected cross sections used in the hydraulic analyses are shown on the Flood Profiles (Exhibit 1).

City of Agoura Hills

Peak discharges were routed through the area considering the capacities of existing flood control systems. Capacities of these systems were obtained from design records or were computed using Manning's Equation. Topographic and cross section data were compiled from existing topographic maps (Los Angeles County Flood Control District, 1968 and U.S. Department of the Interior, Geological Survey, 1967) and field surveys. Features which cause changes in flow depths such as changing ground slope or obstructions were considered in determining water-surface elevations.

Roughness coefficients (Manning's "n") were estimated by field inspection of the areas under investigation. Manning's "n" values ranged from 0.03 in the channels to 0.05 in the overbanks.

Locations of selected cross sections used in the hydraulic analyses are shown on the Flood Profiles (Exhibit 1).

Starting water-surface elevations used in this study were determined from normal-depth calculations adjusted to field conditions.

For the 1998 revision to the Agoura Hills FIS, the water-surface elevations for the 1-percent annual chance flood event were computed through the use of the USACE HEC-2 computer program (U.S. Department of the Army, Corps of Engineers, September 1990) and manual calculations.

At the downstream end of the restudy, from approximately 1,040 feet downstream of Kanan Road to the concrete channel downstream of Kanan Road, the HEC-2 model was developed using cross section information developed for the previous Flood Insurance Study for the City of Agoura Hills (Federal Emergency Management Agency, December 18, 1986), including cross section data and workmaps obtained from Los Angeles County (Los Angeles County Department of Public Works, September 4, 1979 and September 25, 1979) and as-built construction drawings provided by Los Angeles County (Los Angeles County, Construction Drawings PM 100203, September 6, 1979 and Construction Drawings PM 7982, August 17, 1979).

For the reinforced-concrete channel from downstream of Kanan Road to Thousand Oaks Boulevard, the 1-percent annual chance discharges are contained under supercritical flow conditions as supported by design calculations submitted by the Los Angeles County Public Works Department, which were prepared by Hale, Haaland & Associates, Inc. (Hale, Haaland & Associates, Inc., February 1979).

For the restudy area upstream of Thousand Oaks Boulevard to the Ventura County line, the analyses were primarily based on the USACE HEC-2 computer model prepared by Simons, Li & Associates, Inc., for the Medea Creek Rehabilitation as part of the Morrison Ranch Project (U.S. Department of Housing and Urban Development, 1978). The as-built-conditions HEC-2 model provided by the City of Agoura Hills was also used (City of Agoura Hills, December 6, 19 93). The model was extended downstream approximately 600 feet to tie into the upstream end of the concrete channel at Thousand Oaks Boulevard. This extension was based on t he Los Angeles County as-built construction drawings (Los Angeles County, Construction Drawings PM 100203, September 6, 1979 and Construction Drawings PM 7982, August 17, 1979). The downstream starting water-surface elevation was based on the Los Angeles County design water-surface elevation at the upstream end of the supercritical reinforced-concrete-lined section.

Roughness coefficients (Manning's "n" values) used in the hydraulic analyses along Medea Creek ranged from 0.015 to 0.070 for the channel and from 0.040 to 0.070 for the overbank areas. Roughness coefficients were assigned based on the assumption of little or no channel maintenance.

City of Avalon

Topographic and cross section data were compiled from existing topographic maps, street plan data, and by field survey work. Topographic maps were obtained from the city at scales of 1:2,400, with contour intervals of 2 and 5 feet and 1:6,000, with a contour interval of 10 feet. Plans of all bridges and culverts were reviewed to determine elevation data, hydraulic characteristics, and structural geometry.

Locations of selected cross sections used in the hydraulic analyses are shown on the Flood Profiles (Exhibit 1).

Design capacities of storm drains and channels were derived from existing design data for each facility. Where design data were lacking, drain capacities were determined using Manning's Equation based on normal-depth assumptions.

Overland flows were routed through the community considering capacities of all existing drainage facilities. In those areas where storm discharges of the selected recurrence intervals exceeded drain capacities, surface flows existed and field cross sections were used to determine flood depths. Features which cause changing flow depths, such as changing ground slope or obstructions, were considered. In all cases, the changes in flow depth caused by these features were deemed to be insignificant and calculations for backwater were not warranted; therefore, uniform flow characteristics do exist and normal-depth analysis was used.

However, because the hydraulic effectiveness of the cross section is reduced by the presence of many obstructions, such as structures and walls, a w etted perimeter reduction factor was applied to appropriate cross sections. The factor is a measure of the percentage of blockage across the cross sectional area and has the effect of reducing the flow-carrying capacity of the cross section, thus increasing the water- surface elevation of peak discharges.

For determining depths and limits of flooding, the floodplain was divided into 3 study sections: the open area upstream of Tremont Street; the densely developed area between Tremont and Beacon Streets; and the section downstream of Beacon Street.

The section upstream of Tremont Street is characterized by sparse development, and hydraulic calculations were based on this condition. The section between Tremont and Beacon Streets is densely developed, but has a f ew vacant lots scattered throughout the area. The effect of these vacant lots on the depth of flooding throughout the overall area is negligible. Therefore, the vacant lots were assumed improved, and the wetted perimeter reduction factor was uniformly applied throughout this section. The section downstream of Beacon Street includes a large, open plaza area which was considered as open space in the hydraulic calculations.

Roughness coefficients (Manning's "n") for overland flow conditions were estimated by field inspection at the locations under investigation and ranged from 0.030 to 0.050.

Cities of Bellflower, Carson, Compton, Downey, Gardena, Lakewood, Long Beach (terrestrial flooding sources only), portions of Los Angeles affected by Los Angeles River, Lynwood, Paramount, Pico Rivera, South Gate

Cross section data developed for the backwater analysis of floods affecting these cities were obtained from aerial photogrammetry. The channel cross sections in the upper reaches of the Los Angeles River were developed from as-built plans obtained from the USACE. Elevation data for interstate highways crossing the channel and floodplain were obtained from the USACE and CALTRANS.

The roughness factor (Manning's "n") of 0.0 16 used for the channel was chosen based on engineering judgment of the design parameters and field observation of the concrete channel.

The roughness factors (Manning's "n") in the overbank areas were adjusted to compensate for the urbanized areas in the floodplain. The adjustment is based on the percentage of blockage parallel and perpendicular to the direction of flow. This factor has the effect of reducing the flow-carrying capacity of the cross section, thus raising the calculated water-surface elevation. The overbanks were divided into industrial and residential for this analysis. Industrial developed cross sections indicated a roughness factor of 0.05 with residential ranging from 0.10 to 0.15. A weighted average was used for cross sections comprised of industrial and residential development.

CALTRANS provided geometrical information for the backwater-producing structures in the lower reach. They include Interstates 405, 91, 710, and 105. Spot elevation data points in conjunction with aerial cross sections were used to determined weir elevations of the SPRR, the Union Pacific Railroad (UPRR), the Atchison Topeka and Santa Fe Railroad (ATSFRR) and ridges of high ground which separate flow paths in the overbank areas.

Expansion and contraction coefficients of 0.3 to 0.5, respectively, were used upstream and downstream of highways and railroads where flows were constricted to underpasses or limited crossing areas. A 1:1 contraction of flow upstream and a 4:1 expansion of flow downstream of the structures were used to define the effective flow areas and non-effective hydraulic "shadows". Cross sections were modified by the use of encroachment routines and/or modification of cross section geometry to describe ineffective flow areas.

Starting water-surface elevations used in the USACE computer program, HEC-2, for the overbank areas were based on critical depth, normal depth or depths over weirs.

The 1-percent annual chance peak overbank flow rates developed by the USACE and documented in the LACDA report for the Los Angeles River lower reach and the Rio Hondo were used to determine potential overbank water-surface elevations and floodplain limits.

Locations of selected cross sections for the entire study used in the hydraulic analyses are shown on the Flood Profiles (Exhibit 1).

The hydraulic analyses for this study were based on unobstructed flow. The flood elevations shown on the profiles are thus considered valid only if hydraulic structures remain unobstructed, operate properly, and do not fail.

The following information refers to different flow paths. These flow paths are limited to smaller reaches than the profile flow paths and the names differ from those used to label the profiles.

Los Angeles River Left Overbank

The left overbank of the Lower Los Angeles River is divided into two areas. The first area floods as a result of a levee failure on the Los Angeles River near the

Century Freeway. The second area floods as a result of levee failure near Wardlow Road.

The first area extends from the Century Freeway to the Pacific Ocean east of Signal Hill. According to the LACDA report the left levee of the Los Angeles River fails at Fernwood Avenue. The LACDA report assumes that the Century Freeway is not in place. The location of levee failure did not take into account the new freeway. However, recent correspondence with the USACE confirms that the levee failure location should not change significantly with the inclusion of the Century Freeway. Therefore, for the purposes of this Flood Insurance Study, the Century Freeway will be considered "in place." The magnitudes and locations of breakout are given in the LACDA report. The Fernwood Avenue breakout is assumed to be downstream of the Century Freeway. The peak flow rate is reduced through this reach due to attenuation as was done in the LACDA report.

The floodplain analysis in the first area includes three different flow paths. For the reach between the Century Freeway and just upstream of the Artesia Freeway the entire breakout is modeled in one flow path with a discharge of 57,000 cfs. Just upstream of the Artesia Freeway the overbank is divided into two paths. The main flow path with a discharge of 39,700 cfs is west of the UPRR and the second flow path with a discharge of 17,000 cfs is east of the railroad.

Downstream of the Artesia Freeway, the UPRR and Paramount Boulevard are elevated above the adjacent ground and form a barrier for flows draining in the east or west direction. Water may only flow in those directions when it has ponded high enough on either side to flow over the top. In order to analyze this area two separate flow paths have been modeled. The main flow path is west of the UPRR. The secondary flow path is east of Paramount Boulevard. In the second flow path, flow is limited by high ground at Clark Avenue on the east and Paramount Boulevard on the west. The HEC-2 split flow option was used to simulate weir flow over Paramount Boulevard and the UPRR. The weir extended from the Artesia Freeway for approximately 2,500 feet. Downstream of this reach oil berms and high ground block any additional transfer of flow. The flow in the second flow path continues south but is limited from spreading west by the UPRR. Downstream of Del Amo Boulevard the flow paths are permanently divided by Signal Hill. The secondary flow path is prevented from spreading east beyond the high ground near Bellflower Boulevard until it reaches Del Amo Boulevard. Downstream of Del Amo Boulevard the HEC-2 split flow option is used to simulate the transfer of flows east toward the San Gabriel River. Normal depth outflow through the streets is assumed. This area where flow is transferred east to the San Gabriel River is designated as an AO Zone. Between Carson Street and Monlaco Road high ground prevents the further transfer of flow and an island is formed. A separate flow path is modeled adjacent to the San Gabriel River using the results of the split flow analysis. Downstream of the island an effective flow line is used to simulate the spread of the recombining of the flows. The total combined flow then continues south to Los Alamitos Bay.

As previously discussed, the main flow path carries its flow adjacent to the Los Angeles River at the Artesia Freeway. Between the Artesia Freeway and the oil tank berms additional flows are added from the secondary flow path. Downstream of this location the main flow path is confined on the east by the UPRR. Downstream of Washington Street the UPRR turns and runs diagonally toward the Los Angeles River. Because the railroad is elevated, it forces water back in the river. The Los Angeles River levees are assumed to remain in place therefore water is forced over the levees into the river. Critical depth was assumed as the starting water-surface elevation. A constriction is formed just downstream of where the UPRR crosses the Los Angeles River which prevents any additional overbank flows. This constriction is caused by Signal Hill.

The second area of the left overbank of the Los Angeles River is flooded downstream of the San Diego Freeway (Interstate 405) due to a levee failure and a breakout discharge of 14,200 cfs in the vicinity of Wardlow Road. Downstream of this breakout the levee is assumed to remain in place and flows are attenuated as described in the LACDA report.

HEC-2 backwater runs were made from the ocean to the San Diego Freeway. These runs indicate that it is possible for water to pond high enough to overtop the Los Angeles River levee and flow back into the main channel. The split flow option (weir flow) in HEC-2 was used to allow water to flow over the levee back into the channel.

Los Angeles River Right Overbank

In the right overbank of the lower Los Angeles River upstream of Del Amo Boulevard, water-surface elevations were determined using HEC-2 and the 1-percent annual chance peak flow rates developed by the USACE for the LACDA report for the breakout at Fernwood Avenue. The actual breakout of 18,200 cfs will be downstream of the Century Freeway as discussed for the Los Angeles River left overbank. Floodplain limits extend upstream of the actual breakout location due to backwater effects. Starting water-surface elevations were determined from critical depth at the Compton Creek levees and the results of the downstream studies at Del Amo Boulevard.

The reach downstream of Del Amo Boulevard to Interstate 405 is affected by breakouts at two different locations: the Compton Creek breakout from the north and the Wardlow Road breakout from the east. The water-surface elevations were determined at each street intersection in the reach between the Los Angeles River and the SPRR assuming normal depth and using Manning's equation. A trial and error process was used to balance the flows going to and from each intersection. Two outflow locations exist for this area. The first is Interstate 405 where flows drain south through the underpasses. The outflow at these underpasses was determined from normal depth calculations. The second outflow location is the SPRR where flows drain west over the SPRR to the Dominguez sink area. The Dominguez sink area is a natural depression with a capacity of approximately 20,000 acre-feet at elevation 20 feet. The outflow over the SPRR was determined from weir flow calculations.

Two separate inflow locations to the Dominguez Sink were analyzed. The first source is the weir flow over the SPRR between Del Amo Boulevard and Interstate 405. The second source of flow to the Dominguez Sink is from a constricted section downstream of Interstate 405, just east of Wilmington Avenue. Weir flow calculations were used to determine the amount of flow to the Dominguez Sink from this source. Water does not pond high enough in the sink to allow flows to drain out of the sink area during the 1-Percent Annual Chance flood.

For the reach upstream of Interstate 405 between the SPRR and the Dominguez Sink the depth of water was determined by using the 1-percent annual chance peak flow rate over the SPRR (with the exception of what drains through Wilmington Avenue). This flow was distributed across the available area resulting in a shallow flooding area with a depth of 3 feet.

The remainder of flow which does not go to the Dominguez Sink continues downstream to the Pacific Ocean. The flow rates obtained by the analyses described above do not result in the same flow rates obtained by the USACE in the LACDA report. The USACE did not take into account the second source of flow to the Dominguez Sink from the constricted section downstream of Interstate 405. Therefore, the flow rates used in this Flood Insurance Study are less than those obtained by the USACE. Once the final peak flow rates were determined, the HEC-2 computer program was used to determine the water-surface elevations.

Rio Hondo Left Overbank

The left overbank of the Rio Hondo extends from the Whittier Narrows Dam to the Century Freeway. Just downstream of Whittier Narrows Dam the overbank floods as a result of the levee failure at Beverly Boulevard. A portion of the breakout is confined to spreading grounds on both sides of the channel and is considered ineffective. The remainder of the flow, 9,020 cubic feet per second (cfs), drains south to the UPRR where it crosses through underpasses at Rosemead Boulevard, Lexington Avenue and Whittier Boulevard. The peak flow rate is reduced throughout this reach due to attenuation as was done in the LACDA report. Percolation basins adjacent to the Rio Hondo and the San Gabriel River are considered ineffective flow areas since these basins may be full at the time of a flood event.

Downstream of the UPRR to the ATSFRR, the overbank is divided into three separate flow paths. One flow path is bounded by the Rio Hondo on the west and a ridge near Rosemead Boulevard on the east. A second flow path is bound by the ridge near Rosemead Boulevard on the west and another ridge near Passons Boulevard on the east. The third flow path is bound by the ridge near Passons Boulevard on the west and the San Gabriel River on the east. High ground between these flow paths prevents the overbank flows from spreading unhindered to the east. The HEC-2 split flow option for weir flow was used to determine the amount of flow which crosses east over the ridges between each cross section and continues south in the overbank.

Most of the water that spreads east to the third flow path, adjacent to the San Gabriel River, overtops the river levees and escapes to the channel. This is possible since these levees are often lower than the adjacent overbank. Along with the HEC-2 split flow option, hand calculations were used to determine the amount of flow which enters the San Gabriel River. Based on the LACDA report and conversations with the USACE, it was determined that adequate capacity existed in the San Gabriel River, above the 1- percent annual chance flows releases from Whittier Narrows Dam, to allow the flows from the Rio Hondo overbank to enter the channel. A total of almost two-thirds of the breakout flows from the Rio Hondo overtop the levees between the dam and the Century Freeway with most of the flows escaping upstream of the ATSFRR.

Once the final flow rates in each path were determined the HEC-2 computer program was used to determine water-surface elevations and floodplain limits. Normal depth calculations were used to determine the depths in the shallow flooding areas.

At the ATSFRR, all the flow remaining in the left overbank crosses at the Rosemead Boulevard underpass. This water then flows south between the Rio Hondo and a ridge of high ground at approximately Passons Boulevard to Interstate 5. At Burke Street, downstream of Slausen Avenue, a small portion of the flow escapes east over the ridge as determined by the HEC-2 split flow weir analysis. The water that flows east over the high ground at Burke Street continues east toward the San Gabriel River and flows over the river levees near the ATSFRR. The San Gabriel River levees in this reach are lower than the adjacent ground which is sloping eastward toward the river. The area between Passons Boulevard and the San Gabriel River is zoned as a shallow flooding area with average depths of 1 foot. This depth was based on normal depth calculations using the elevations of the streets in the direction of flow.

Downstream of Interstate 5 to the Century Freeway a total of three flow paths exist with high ground separating each flow path. The main flow path is adjacent to the Rio Hondo and extends from Interstate 5 to the Century Freeway. At Stewart and Gray Road additional breakouts from the Rio Hondo join the left overbank flows. The second flow path is immediately east of the main flow path between Florence Avenue and the SPRR. A portion of the flows from the first flow path escapes to the second flow path at Florence Avenue. The third flow path begins at Gallatin Road where flows from the first flow path begin to flow over high ground. Flow paths two and three combine downstream of the SPRR. The combined flow from the second and third flow paths extend to the Century Freeway and is adjacent to the San Gabriel River.

At Interstate 5 all flow passes through the openings at Paramount and Lakewood Boulevards. This water then flows south adjacent to the Rio Hondo in the main flow path. Between Interstate 5 and Gallatin Road a small portion of the flow crosses east over high ground near Lakewood Boulevard into the third path. The amount of flow crossing over the high ground was determined using weir flow of the split flow option in the HEC-2 hydraulic model. At Florence Avenue a portion of the flow from the main flow path escapes east into the second flow path. This amount of flow was determined using normal depth calculations for the available street capacity at the known water-surface elevation (from the main flow path HEC-2 runs). Due to high ground adjacent to Burke Street and the southeastern slope of the land, none of the flow that escapes east from the main flow path returns. At Stewart and Gray Road the discharge is reduced to account for attenuation. At this location the additional breakout flows of 1,395 cfs are also added as determined by the USACE. Between the Imperial Highway and the Century Freeway the UPRR crosses diagonally through the main flowpath. The railroad is elevated on fill. This reach was analyzed for two conditions. The first condition assumes the railroad embankment fails and water distributes evenly across the floodplain in one flow path. The second condition assumed the embankment remains in place and flows east of the railroad must pond to the elevation of the railroad embankment before it can cross over to the west. The amount of flow that crosses over the railroad was determined using weir flow of the split flow option in the HEC-2 hydraulic model with the railroad embankment elevations used for the weir crest elevations. HEC-2 backwater runs were made to determine water-surface elevations for the entire main flow path using the flow rates determined above. The HEC-2 runs that resulted in the greater water-surface elevations were used in mapping the floodplains. The starting water-surface elevation used at the Century Freeway was the water-surface elevation obtained from the downstream study of the Los Angeles River left overbank.

In the second flow path the water is confined between high ground to the east and west until it gets downstream of the SPRR. At this point the flows between the second flow path begin combining with the flows in the third flow path. HEC-2 backwater runs were made to determine the water-surface elevations in the second flow path. The starting water-surface elevation was determined using normal depth calculations. In the transition between flow paths 2 and 3 a shallow flooding zone occurs with water depths varying from one to two feet as determined from spot elevations.

Flows from the main path adjacent to the Rio Hondo begin entering the third flow path downstream of Interstate 5. These flows are prevented from continuing east to the San Gabriel River until upstream of Firestone Boulevard. At this point the high ground is reduced and the flows are free to drain to the east and flow against the San Gabriel River levees. Further downstream water from flow path two enters the third flow path and also continues east to the San Gabriel River levees. The HEC-2 backwater analysis indicates that the water-surface elevation is high enough at this point to allow a portion of the flows to flow over the San Gabriel River levee. This was determined using the HEC-2 split flow option for weir flow and the as-built levee elevations on the San Gabriel River levee for the weir elevations. The remaining flow in the overbank continues south to the Century Freeway.

At the Century Freeway the flows in the third flow path (which includes the flows from the second flow path) run into the depressed freeway section and drain west toward the Los Angeles River where they combine with flows from the main flow path and cross over into the left overbank adjacent to the Los Angeles River. At this same location another breakout occurs on the Los Angeles

River. The magnitude of the breakout of the Los Angeles River is much greater than that of the Rio Hondo breakouts. The peaks of the two breakouts occur at different times according to the USACE, therefore, only the larger breakout amount from the Los Angeles River is used to analyze the floodplain limits and depths downstream of the Century Freeway.

Rio Hondo Right Overbank

Upstream of the Los Angeles River-Rio Hondo confluence a triangle is formed which is flooded from a breakout of the right Rio Hondo levee at Stewart and Gray Road. The Los Angeles River levees upstream of the confluence are certified by the USACE.

In order for water to get back into the channels (Rio Hondo or Los Angeles River) it must pond behind the levees at the confluence then flow over them. Water-surface elevations were determined using the HEC-2 model.

City of Burbank

In order to compute water-surface elevations within the City of Burbank, peak discharges were routed through the community considering capacities of existing flood control facilities. At locations where peak discharges exceeded the available drainage system capacity, field reviews and cross section data were used to determine depths of the overland flows. Capacities of channels and storm drains were obtained from design records or were derived from available data using Manning's equation based on normal depth assumptions. Topographic and cross section data were compiled from existing topographic maps, field reviews, and street plan data on file at the Los Angeles County Flood Control District.

Water-surface profiles were not prepared because the 1-percent annual chance flooding in Burbank is not readily associated with channel flooding and flood profiles.

Roughness coefficients (Manning's "n") for overland flow conditions were estimated by field inspection of the areas under investigation, and values ranging from 0.014 to 0.050 were used.

Country Club Drive in Sunset Canyon acts as a channel for storm runoff, and depths calculated are based on normal depth assumptions indicating supercritical flow. However, it was concluded that the combined effects of variations in channel roughness, short-radius curves, and debris will cause the flows to be at critical depth and, therefore, the flooding limits in Sunset Canyon were based on critical depth calculations.

Features which cause changing flow depths, such as changing around slope or obstructions, were considered. In all cases, the changes in flow depth caused by these features were deemed to be insignificant and backwater calculations were not used. However, because the effectiveness of the calculated cross sections is reduced by the presence of obstructions, such as buildings and walls, a wetted perimeter reduction factor was applied. The factor is a measure of the percentage of blockage across the cross sectional area and has the effect of reducing the flow-carrying capacity of the cross section, thus increasing the water-surface elevation of peak discharges.

To analyze ponding conditions, the Los Angeles County Flood Control District's Regional Normalized Hydrograph Equation was used to determine the volume of water generated by the 1-percent annual chance flood event. Where necessary, the volume was reduced by reservoir routing flood flows through the ponded areas.

For the January 20, 1999 revision, water-surface elevations were computed through the use of the USACE HEC-2 computer program (U.S. Department of the Army, Corps of Engineers, Hydrologic Engineering Center, September 1990). The parameters used were as follows:

1. Channel cross sections and structure dimensions were obtained from as-built plans for the Lockheed Drain Channel (Federal Works Agency, November 1944).
2. Cross sections in the overbank areas were determined from City of Burbank topographic mapping at a scale of 1 "= 100', with a contour interval of 2 f eet (Analytical Surveys, Inc., May 1988), supplemented by grading plans (City of Burbank, March 1991 and Lockheed Engineering and Science Co., October 1993) and field-reconnaissance surveys.
3. The roughness coefficient (Manning's "n" value) for various lined portions of the channel was set at 0.020. All other values were based on field inspection. Earthen channel "n" values were set at 0.035. Overbank "n" values ranged from 0.020 to 0.045, and were determined using the procedure developed by the USGS (U.S. Department of the Interior, Geological Survey, October 1977). Building blockages were estimated from the City's topographic mapping (Analytical Surveys, Inc., May 1988) and field-reconnaissance surveys. These values ranged between 0.100 and 0.150.
4. Starting water-surface elevations were calculated using the slope-area method.
5. All culverts and bridges were modeled on assumed unobstructed flow. Bridges were modeled using the HEC-2 special-culvert or normal-bridge methods. For the long pipe conduit that begins at Clybourn Avenue, an elevation discharge rating curve was determined by manual calculation and was used for the HEC-2 analyses.
6. HEC-2 split-flow routines, based on a weir discharge coefficient of 2.6, were used to determine channel overflows.

The boundaries of the 1-percent annual chance flood were delineated using the flood elevations determined at each cross section. Between cross sections, the boundaries were interpolated using aerial topographic mapping at a scal e of 1 "=100', with a contour interval of 2 f eet, that was prepared for this restudy

(Analytical Surveys, Inc., May 1988). The sheet-flow areas where flooding depths are less than 1 foot are designated Zone X. Areas where flooding depths exceed 1 foot are designated Zone AE and the calculated 1-percent-annual-chance BFEs are designated on the Flood Insurance Rate Map.

City of Culver City

Peak discharges for locations within the City of Culver City were routed through the community considering where peak discharges exceeded the available drainage system capacity, field reviews and cross section data were used to determine depths of the overland flows. Capacities of channels and storm drains were obtained from design records or were derived from available data using Manning's equation based on normal depth assumptions. Topographic and cross section data were compiled from existing topographic maps and street plan data.

Features that cause changing flow depths, such as changing ground slope or obstructions, were considered. In all cases, the changes in flow depth caused by these features were deemed to be insignificant, and backwater calculations were not used. However, because the effectiveness of the calculated cross sections is reduced by the presence of obstructions, such as buildings and walls, a wetted perimeter reduction factor was applied. The factor is a measure of the percentage of blockage across the cross sectional area and has the effect of reducing the flow-carrying capacity of the cross section, thus increasing the water-surface elevations of peak discharges.

Roughness coefficients (Manning's "n") for overland flow conditions were estimated by field inspection of the areas under investigation, and a value of 0.040 was used throughout the study.

Water-surface profiles were not prepared because the 1-percent annual chance flooding in Culver City is not readily associated with channel flooding and flood profiles. Therefore, flooding limits and depth were established through the use of available topography and field reviews.

Shallow flooding, resulting from inadequate drainage and having an average depth of 1 foot, occurs on the east side of Ballona Creek Channel in the vicinity of the intersection of Adams and Washington Boulevards. Also, shallow flooding with depths less than 1 foot occurs along the western border of Hannum Avenue, in the northeast section of the Fox Hills Mall.

City of La Mirada

The peak discharges for floods of the selected recurrence intervals within the City of La Mirada were routed through the community with consideration given to the capacities of existing flood-control facilities. At locations review and cross section data were used to determine depths of the overland flow. Capacities of channels and storm drains were obtained from design records or were derived from available data using Manning's Equation, based on normal depth assumptions. Topographic and cross section data were compiled from existing topographic maps, street plan data, and field reviews. Features which cause changing flow depths, such as changing ground slope or obstructions, were considered. In all cases, the changes in flow depth caused by these features were deemed to be insignificant and backwater calculations were not used. However, because the effectiveness of the calculated cross section is reduced by the presence of obstructions, such as buildings and walls, a wetted perimeter reduction factor was applied. The factor is a measure of the percentage of blockage across the cross sectional area and has the effect of reducing the flow-carrying capacity of the cross section, thus increasing the water- surface elevation of peak discharges.

Locations of selected cross sections used in the hydraulic analyses are shown on the Flood Profiles (Exhibit 1).

Roughness coefficients (Manning's "n") for overland flow conditions were estimated by field inspection; values ranged from 0.025 to 0.030 for both channel and overbank areas.

To analyze ponding conditions, the Regional Normalized Hydrograph Equation of the Los Angeles County Flood Control District was used to determine the volume of water generated during a 1-percent annual chance flood event. Where necessary, the volumes were reduced by reservoir-routing flood flows through the ponded areas.

Flood profiles were drawn showing computed water-surface elevations to an accuracy of 0.5 foot for floods of the selected recurrence intervals (Exhibit 1).

City of Lancaster

The preliminary flood depths within the City of Lancaster were determined by routing peak discharges through the community using the boundaries of the alluvial fans, historical records, and field reviews. Average depths of flooding were assigned based on standard hydraulic calculations through irregular cross sections. In many instances, the assigned average depth is not representative of the true degree of flood hazard. This occurs when average depths are based on a wide cross section which encompasses one or more low-flow drainage courses. The actual depth of flooding, and, consequently, the true flood hazard, will be greater adjacent to the drainage course.

Features that cause changing flow depths, such as changing ground slope or obstructions, were considered. In all cases, the changes in flow depth caused by these features were deemed to be insignificant, and backwater calculations were not used.

Topographic and cross section data were compiled from existing topographic maps and from topographic maps prepared by the County Engineer.

Roughness coefficients (Manning's "n") for overland flow conditions were estimated by field inspection of the areas under investigation, and a value of 0.04 was used throughout.

Water-surface profiles were not prepared because the 1-percent annual chance flooding in Lancaster is not readily associated with channel flooding, and flood profiles are not applicable.

City of Long Beach

Analyses of the hydraulic characteristics of flooding from oceanic sources were carried out to provide estimates of the elevations of floods of selected recurrence intervals along each of the shorelines. The discussion of flood hydraulics from terrestrial sources is covered in the section on the Cities of Bellflower, et al., above.

In order to obtain runup values for the various flood producing mechanisms, data on offshore bathymetry and beach profiles were obtained from U.S. Coast and Geodetic Survey and National Oceanic and Atmospheric Administration bathymetric charts; USGS topographic maps; surveys of beach profiles conducted by the USACE, Los Angeles District; and from aerial photographs of the study area.

City of Los Angeles

Analysis of the City of Los Angeles included all those issues related to the study of communities within the Los Angeles River watershed, and are covered under the Cities of Bellflower, at al. above. Areas outside the influence of the Los Angeles River are discussed below.

Peak discharges were routed through the City considering capacities of existing flood-control facilities. At locations where peak discharges exceeded the available drainage system capacity, field reviews and cross section data were used to determine depths of the overland flows. Capacities of channels and storm drains were obtained from design records or were derived from available data using Manning's equation based on normal-depth assumptions. Topographic and cross section data were compiled from existing topographic maps, street plan data, and field surveys.

Features that cause change in flow depths, such as changing ground slope or obstructions, were considered. In all cases, the changes in flow depth caused by these features were deemed to be insignificant, and backwater calculations were not used. However, because the effectiveness of the calculated cross sections is reduced by the presence of obstructions, such as buildings and walls, a "wetted perimeter reduction factor" was applied. The factor is a m easure of the percentage of blockage across the cross sectional area and has the effect of reducing the flow-carrying capacity of the cross section, thus increasing the water-surface elevation.

Locations of selected cross sections used in the hydraulic analyses are shown on the Flood Profiles (Exhibit 1). For stream segments for which a floodway was computed, selected cross section locations are also shown on the FIRM.

Roughness coefficients (Manning's "n") for overland flow conditions were estimated by field inspection of. the areas under investigation, and values of 0.030 and 0.040 were used throughout as appropriate. Values of 0.065, 0.055, and 0.035 were used as Manning's "n" in the hydraulic analyses of the natural watercourses.

Starting water-surface elevations were determined from normal-depth calculations.

Flood profiles were drawn showing computed water-surface elevations to an accuracy of 0.5 foot for floods of the selected recurrence intervals (Exhibit 1). No profiles are shown for Pacoima, Little Tujunga, aid Big Tujunga Washes because of the unpredictability of the location of the stream across the width of the alluvial fan.

To analyze ponding conditions, the Los Angeles County Flood Control District regional normalized hydrograph equation was used to determine the volumes of water generated by the 1-percent annual chance discharges. Where necessary, the volumes were reduced by reservoir routing flood flows through the ponded areas.

One of the mapped areas of shallow flooding is along the upper reaches of Browns Creek, which results from shallow overbank flows. During the 1-percent annual chance flood event, the water will leave the improved channel because the bridges will become plugged with debris due to the lack of a debris retention facility upstream.

Big Tujunga, Little Tujunga, and Pacoima Washes exit the San Gabriel Mountains on alluvial fans. The potential limits of flooding were delineated by determining the boundaries of the alluvial fans. The depths were assigned using mean depth at critical slope through the irregular cross sections.

Harbor Lake (previously known as Bixby Slough) was analyzed by comparing the inflow to the lake with the outflow from the lake to San Pedro Bay. Outflow is limited by the capacity of a large underground culvert, Project No. 1103.

City engineers have indicated that an inland strip along the beach, northwest of Ballona Creek outlet, has historically been subject to shallow flooding because, during major storms, the drains serving the area have not functioned at high tide.

City of Montebello

Analysis of the City of Montebello included all those issues related to the study of communities within the Los Angeles River watershed, and are covered under the Cities of Bellflower, at al. above. Areas outside the influence of the Los Angeles River are discussed below.

The 1-percent annual chance peak discharge for the original study was routed through the community considering capacities of existing flood-control facilities. At locations where peak discharges exceeded the available drainage system capacity, field reviews and cross section data were used to determine depths of the overland flows. Capacities of channels and storm drains were obtained from design records or were derived from available data using Manning's Equation based on normal depth assumptions. Topographic and cross section data were compiled from existing topographic maps.

Features that cause changing flow depths, such as changing ground slope or obstructions, were considered. In all cases, the changes in flow depth caused by these features were deemed to be insignificant, and backwater calculations were not used. However, because the effectiveness of the calculated cross sections is reduced by the presence of obstructions, such as buildings and walls, a "wetted perimeter reduction factor" was applied. The factor is a measure of the percentage of blockage across the sectional area and has the effect of reducing the flow-carrying capacity of the cross section, thus increasing the water-surface elevation of peak discharges.

Roughness coefficients (Manning's "n") for overland flow conditions were estimated by field inspection of the areas under investigation and values of 0.015 and 0.020 were used.

As a result of these calculations, it was determined that shallow flooding with depths of 1 foot and less than 1 foot would occur in the vicinity of Garfield Avenue.

To analyze ponding conditions, the LACFCD Regional Normalized Hydrograph Equation was used to determine the volume of water generated by the 1-percent annual chance discharge. Where necessary, the volume was reduced by reservoir routing flood flows through the ponded areas.

The volume of water generated by the 1-percent annual chance flood at Whittier Narrows Dam is contained within the reservoir area. The USACE has entered into lease agreements with private owners for use of the reservoir lands. These individual owners could be eligible for flood insurance; and, at the FIA's instructions, the reservoir area has been mapped showing 1-percent annual chance flood boundaries only. It was not deemed necessary to determine 0.2-percent annual chance discharges or elevations.

Field investigation was the method used to study approximate areas.

City of Palmdale

The preliminary flood depths for Amargosa Creek, Amargosa Creek Tributary, Anaverde Creek, and Anaverde Creek Tributary were determined by routing peak discharges through the community using the boundaries of the alluvial fans, historical records, and field reviews. Average depths of flooding were assigned based on standard hydraulic calculations through irregular cross sections. In many cases, the assigned average depth is not representative of the true degree of flood hazard. This situation occurs where average depths are based on a wide cross section which encompasses one or more lowflow drainage courses. The actual depth of flooding and, consequently, the true flood hazard will be greater adjacent to the drainage course.

Features that cause changing flow depths, such as changing ground slope or obstructions, were considered. In all cases, the changes in flow depth caused by these features were deemed to be insignificant, and backwater calculations were not used.

Topographic and cross section data were compiled from topographic maps prepared by the County Engineer.

Locations of selected cross sections used in the hydraulic analyses are shown on the Flood Profiles (Exhibit 1). Selected cross section locations are also shown on the Flood Insurance Rate Map.

Flood depths for Big Rock Wash and Little Rock Wash were determined utilizing the USACE HEC-2 step-backwater computer program. Cross sections used in the backwater computations were derived from photogrammetric compilation of aerial photographs, flown in November 1984 January 1985, a t a scale of 1:14,400. Topographic mapping was compiled at a scale of one (1) inch equals 400 feet, with a four foot contour interval. Bridges were field surveyed to obtain elevation data and structural geometry.

Starting water-surface elevations were based on approximate hydraulic computations using Manning's equation. Roughness coefficients (Manning's "n") values, were estimated using S.C.S. Guidelines, field investigations, and engineering judgment. For overland flow conditions on Amargosa Creek and Tributary, Anaverde Creek and Tributary, as "n" value of 0.04 was used throughout. Big Rock Wash channel "n" value was 0.05, and an "n" value of 0.05 was used for the overbanks. The "n" values used for Little Rock Creek Wash were 0.03 for the channel, and 0.05 for the overbanks.

Flood depths in the western portion of the city resulting from the flooding of an unnamed tributary from Ritter Ridge northwest of the city and a small segment of Anaverde Creek in western Palmdale, were determined by approximate methods based on the Flood Hazard Boundary Map published by the FIA, field reviews, historical records, and the Los Angeles County Flood Overflow Maps.

For the March 30, 1998 r evision, the water-surface elevations were computed through the use of the USACE HEC-2 computer program (U.S. Army, Corps of Engineers, Hydrologic Engineering Center, November 1976, U pdated May 1984). The HEC-2 model was developed using topographic maps obtained from the California DWR (State of California, Department of Water Resources, April 9, 1990) and field measurements at road crossings.

Channel and overbank cross sections were determined from the California DWR topographic mapping at a horizontal scale of 400 f eet, with a 4-foot contour interval (State of California, Department of Water Resources, April 9, 1990), as well as field measurements.

Manning's "n" roughness values were established based on a field observations and USACE and USGS guidelines and criteria. Channel roughness values used ranged from 0.03 5 t o 0.060 and overbank roughness values used ranged from 0.035 to 0.075.

Contraction and expansion coefficients of 0.1 and 0.3 w ere used for open-channel sections. Contraction and expansion coefficients at culverts and bridges ranged from 0.4 to 0.6.

The downstream starting water-surface elevation was determined using the HEC-2 slope-area method, starting approximately 1,100 feet downstream of State Route 14, the downstream study limit.

Supercritical flow conditions can occur in some channel reaches. Subcritical analyses were conducted to determine base (1-percent annual chance flood) flood elevations (BFEs) for all stream reaches.

City of Redondo Beach

The hydraulic analyses of the small channels that exist in much of the City of Redondo Beach were performed by the methodologies discussed under the section on the City of La Mirada, above.

Hydraulic analyses of the shoreline characteristics of the flooding sources studied in detail within the City of Redondo Beach were carried out to provide estimates of the elevations of floods of the selected recurrence intervals along each of the shorelines. The limit of runup was used to designate flood zones.

To obtain runup values for the various flood-producing mechanisms, data on offshore bathymetry and beach profiles were obtained from the U.S. Coast and Geodetic Survey and the National Oceanic and Atmospheric Administration bathymetric charts, U.S. Geological Survey topographic maps, surveys of beach profiles conducted by the USACE, Los Angeles District, and from aerial photographs of the study area.

To analyze ponding conditions, the Los Angeles County Flood Control District Regional Normalized Hydrograph Equation was used to determine the volume of water generated by the 1-percent annual chance flood event. Where necessary, the volumes were reduced by reservoir routing flood flows through the ponded areas.

City of Santa Clarita

Preliminary flood elevations in the City of Santa Clarita were determined by routing peak discharges through the community using the boundaries of alluvial fans, flood overflow maps, and field reviews. Topographic and cross section data were compiled from existing topographic and floodplain boundary maps. Features that cause changing flow depths, such as changing ground slope or obstructions, were considered. In all cases, the changes in flow depth caused by these features were deemed to be insignificant, and backwater calculations were not used.

Roughness coefficients (Manning's "n") for overland flow conditions were estimated by field inspection of the areas under investigation. The Manning's "n" values used were 0.03 in the channels and 0.06 in the overbanks.

The preliminary flood elevations were field reviewed for verification of actual conditions. Features that would affect flood elevations or depths were noted, and flood elevations were revised accordingly, based on engineering judgment. Average depths of flooding were assigned based on standard normal-depth calculations through irregular cross sections. In many instances, the assigned average depth is not representative of the true degree of flood hazard. This occurs when average depths are based on a wide cross section that encompasses one or more low-flow drainage courses. The actual depth of flooding (and consequently, the true flood hazard) will be greater when located adjacent to the drainage course. In some locations in the Santa Clarita Valley, the low-flow drainage course has been designated Zone A to reflect a more severe flood hazard and to prohibit development. The adjacent floodplain is then given a shallow flooding designation based on average depth across the entire cross section.

Water-surface profiles were not prepared because the 1-percent annual chance flooding in the Santa Clarita Valley is not readily associated with channel flooding and flood profiles. Therefore, flooding limits were established using available topography and field reviews.

City of Santa Fe Springs

Peak discharges were routed through the community considering capacities of existing flood-control facilities. At locations where peak discharges exceeded the available drainage system capacity, field reviews and cross section data were used to determine depths of the overland flows. Capacities of channels and storm drains were either obtained from design records or were derived from available data using Manning's equation based on normal depth assumptions. Topographic and cross section data were compiled from existing topographic maps at a scale of 1:24,000, with a contour interval of 5 feet, street plan data, and field surveys.

Water-surface profiles were prepared for the natural watercourse north of the intersection of Pioneer Boulevard and Florence Avenue (shown as Flowline No. 1 on the map) by use of normal depth analysis. Features which cause changes in flow depths, such as changing ground slope or obstructions, were considered. In all cases, the changes in flow depth caused by these features were deemed to be insignificant and backwater calculations were not used. However, because the effectiveness of the calculated cross sections is reduced by the presence of obstructions, such as buildings and walls, a wetted perimeter reduction factor was applied. This factor is a measure of the percentage of blockage across the cross sectional area and has the effect of reducing the flow-carrying capacity of the cross section, thus increasing the water-surface elevation of peak discharges.

Locations of selected cross sections used in the hydraulic analyses are shown on the Flood Profiles (Exhibit 1).

Roughness coefficients (Manning's "n") for overland flow conditions were estimated by field inspection, and a value of 0.030 was used throughout.

Flood profiles were drawn showing computed water-surface elevations to an accuracy of 0.5 foot for floods of the selected recurrence intervals (Exhibit 1).

Starting water-surface elevations were determined by use of the broad-crested weir formula.

To analyze ponding conditions, the Los Angeles County Flood Control District's Regional Normalized Hydrograph Equation was used to determine the volumes of water generated by the 1-percent annual chance discharges. Where necessary, the volumes were reduced by reservoir routing flood flows through the ponded areas.

City of Torrance

Peak discharges were routed through the community, considering capacities of existing flood-control facilities. At locations where peak discharges exceeded the avai1able drainage system capacity, field surveys, field reviews, and cross section data were used to determine depths of the overland flow:;. Capacities of channel and storm drains were obtained from design records or were derived from available data using Manning's equation based on no rmal depth assumptions. Topographic and cross section data were compiled from existing topographic maps at scales of 1:24,000 with contour intervals of 5 and 20 feet, and 1:480, with a contour interval of 2 feet, field surveys, and street plan data.

Features that cause changing flow depths, such as changing ground slope or obstructions, were considered. In all cases, the changes in flow depth caused by these features were deemed to be insignificant, and backwater calculations were not used.

To analyze ponding conditions, the Los Angeles County Flood Control District's regional normalized hydrograph equation was used to determine the volume of water generated by the 1-percent annual chance flood peak discharge. Where necessary, the volumes were reduced by reservoir routing flood flows through the ponded areas.

Water-surface profiles were not prepared because the 1-percent annual chance flooding in Torrance is not associated with channel flooding and flood profiles.

An approximate coastal high-hazard analysis was conducted for this study. Flooding due to storm surge and wave runup was approximated by adding 3 feet to the highest tide observed in the Los Angeles area. The highest tide observed was taken from observations at Los les .titer Harbor by the U.S. Coast and Geodetic Survey, during the period from 1941 through 1959. The highest tide observed during that period was 4.9 feet. The city's coastline has been designated as beach land by the County of Los Angeles, which will preclude any substantial development of the beach below an elevation of 7.9 feet. Because there are no existing structures and no likelihood of structures being built in the future below an elevation of 7.9 feet along the Torrance coastline, only an approximate coastal high-hazard area has been shown.

City of West Hollywood

Throughout the City, ponding conditions and reservoirs were analyzed using the Los Angeles County Flood Control District Regional Normalized Hydrograph Equation. This equation determines the volume of water generated by the 1-percent annual chance flood event. Where necessary, the volumes were reduced by reservoir routing flood flows through ponded areas.

Flow depths for shallow flooding areas were calculated using available topographic maps, street-plan data, and field surveys. The flow depths were determined using Manning's Equation based on n ormal- depth assumptions. Features such as changing ground slope or obstructions were considered.

Because the effectiveness of the calculated cross sections is reduced by the presence of obstructions such as buildings or walls, a "wetted perimeter reduction factor" was used in heavily developed areas. This factor is a m easure of the percentage of blockage across the cross sectional area and has the effect of reducing the flow-carrying capacity of the cross section. This has the effect of raising the calculated water-surface elevation.

Starting water-surface elevations used in the study were determined from normal-depth calculations adjusted to field conditions. The Manning's "n" value of 0.03 was used to determine flood depths.

City of Whittier

Analyses of the hydraulic characteristics of streams in the community were carried out to provide estimates of the elevations of floods of the selected recurrence intervals along each stream studied in the community.

The 1-percent annual chance peak discharges were routed through the community considering capacities of existing flood-control facilities. At locations where peak discharges exceeded the available drainage- system capacity, field reviews and cross section data were used to determine depths of the overland flows. Capacities of channels and storm drains were obtained from design records or were-derived from available data by using Manning's equation based on normal-depth assumptions. Topographic and cross section data were compiled from existing topographic maps and street plan data.

Features which cause changing flow depths, such as changing ground slope or obstructions, were considered. In all cases, the changes in flow depth caused by these features were considered to be insignificant, and backwater calculations were not used. However, because the effectiveness of the calculated cross sections is reduced by the presence of obstructions such as buildings and walls, a wetted perimeter reduction factor was applied. The factor is a measure of the percentage of blockage across the cross sectional area and has the effect of reducing the flow-carrying capacity of the cross section, thus increasing the water-surface elevation of peak discharges.

Roughness coefficients (Manning's "n") for overland flow conditions were estimated by field inspection of the areas under investigation, and a value of 0.03 was used throughout the study. As a result of these calculations, it was determined that shallow flooding with depths of 1 foot occurs in the vicinity of Painter Avenue and Camilla Street.

Water-surface profiles were not prepared because the 1-percent annual chance flooding in Whittier is not readily' associated with channel flooding.

In order to analyze ponding conditions, the Los Angeles County Flood Control District's Regional Normalized Hydrograph Equation was used to determine the volume of water generated by the 1-percent annual chance flood discharge. Where necessary, the volume was reduced by reservoir routing flood flows through the ponded areas.

The volume of water generated by the 1-percent annual chance flood at Whittier Narrows Dam is contained within the reservoir area. The USACE has entered into lease agreements with private owners for use of the reservoir lands. These individual owners could be eligible for flood insurance; and, at the FIA's instructions, the reservoir area has been studied for the 1-percent chance flood only. It was not deemed necessary to determine 0.2-percent annual chance flood discharges or elevations.

Flood elevations for the city's landfill site, the Friendly Hills County Club golf course, and La Mirada Creek were determined by field investigation and engineering judgment.

During the analysis, 1-percent annual chance shallow flooding was determined along streets having inadequate drainage facilities.

Roughness factors (Manning’s “n”) used in the hydraulic computations were chosen by engineering judgment and were based on field observations of the streams and floodplain areas. Roughness factors for all streams studied by detailed methods are shown in Table 10, “Manning’s “n” Values.”

Refraction

Refraction computations were conducted to trace the evolution of winter swell and tropical cyclone swell from their source to the 60-foot depth contour. A large grid (200 by 250 miles) covering the coastal water of southern California with 1,000 by 1,000-foot grid spacing was used for the refraction calculations. Standard raytracing procedures were used to trace rays inward from the deep ocean grid boundaries. Ray spacing was chosen at 1,000 feet to provide adequate density of ray coverage. Wave heights at the 60-foot contour were computed using the principle of wave energy flux conservation between neighboring rays. One set of refraction computations was performed for each selected event from the list of extreme winter swells and the list of tropical cyclones off Baja California. The winter swell input values were obtained for the FNWC tape for the selected days of extreme events. The values at the three FNWC stations were the basis for linear interpolation to obtain input values in between them. For swell generated by tropical cyclones, the tropical cyclone swell procedure was used to provide input to the refraction program.

Wave Runup

Shoreward of the 60-foot contour, wave runup was determined for each beach profile of interest by adapting to composite beaches the standard empirical runup formulas valid for uniformly sloping beaches. The results of the refraction calculations were used as input. The beach profiles selected were assumed to be locally one-dimensional in order to apply the empirical runup formulas. However, the influence of incident wave directions, refraction, and shoaling effects were also taken into consideration.

Wave heights within the surf zone were also computed using empirical formulas to establish the zone where waves exceed 3 feet.

Computed elevations for wave runup, wave setup, and other inundation hazard characteristics are shown in Table 11, “Summary Elevations for Wave Runup and Wave Setup.”

Table 11: Summary of Elevations for Wave Runup and Wave Setup

Flooding Source and Location	Wave Runup Elevation[1] (feet)			Wave Runup Elevation[1] (feet)		
	10-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance	10-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
PACIFIC OCEAN						
At Will Rogers Beach, Approximately 400 feet South of the Intersection of Tramonto Drive and Porto Marina Way	14.3	19	22.1	--	--	--
At Will Rogers Beach, Approximately 300 feet South of the Intersection of Breve Way and Porta Marina Way	13.4	17.5	20.4	--	--	--
At Will Rogers Beach, at Sunset Boulevard Extended	11.3	13.9	16.5	--	--	--
At Will Rogers Beach at Temescal Canyon Road Extended	10.9	13.3	15.8	--	--	--
At Will Rogers Beach, Approximately 900 feet South of the Intersection of Beirut Avenue and Via De Las Olas	11	13.5	16	--	--	--
At Will Rogers Beach at Entrada Drive Extended	12	15.1	17.8	--	--	--
At Venice Beach at Washington Street Extended	12	15.1	17.8	--	--	--
At Marina Del Ray Entrance Channel and Ballona Creek	--	--	--	7.7	8.9	11.1
At Dockweiler Beach, at Culver Boulevard Extended	11.3	14	16.6	--	--	--
At Dockweiler Beach, at Beaumont Street Extended	11.9	14.9	17.6	--	--	--
At Dockweiler Beach, at Foutainbleau Street Extended	12.5	15.9	18.7	--	--	--
At Dockweiler Beach, at Ipswich Street Extended	13.7	18	21	--	--	--

Table 11: Summary of Elevations for Wave Runup and Wave Setup (continued)

Flooding Source and Location	Wave Runup Elevation[1] (feet)			Wave Runup Elevation[1] (feet)		
	10-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance	10-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
At Dockweiler Beach, Approximately 900 feet Northwest of the Intersection of Imperial Highway and Vista Del Mar	13.1	17.1	19.9	--	--	--
At Dockweiler Beach, Approximately 5,000 feet Northwest of the Corporate Limits	12.8	16.1	18.9	--	--	--
At Dockweiler Beach, Approximately 4,100 feet Northwest of the Corporate Limits	12	15.2	17.9	--	--	--
Along Dockweiler Beach, Approximately 3,400 feet Northwest of the Corporate Limits	11.5	14.2	16.8	--	--	--
Along Dockweiler Beach, Approximately 2,400 feet Northwest of the Corporate Limits	10.9	13.3	15.8	--	--	--
Along Dockweiler Beach, Approximately 1,000 feet Northwest of the Corporate Limits	11.5	14.3	16.9	--	--	--
Along Dockweiler Beach, Approximately 100 feet Northwest of the Corporate Limits	12.1	15.3	18.1	--	--	--
At Corporate Limits, at Royal Palms Beach, Approximately 1,000 feet Northwest of Shad Place Extended	14.1	18.7	21.7	--	--	--
At Royal Palms Beach, at Anchovy Avenue Extended	12.9	16.7	19.5	--	--	--
At Whites Point	12.3	15.7	18.4	--	--	--
At Beach, at Weymouth Avenue Extended	13.5	17.7	20.6	--	--	--

Table 11: Summary of Elevations for Wave Runup and Wave Setup (continued)

Flooding Source and Location	Wave Runup Elevation[1] (feet)			Wave Runup Elevation[1] (feet)		
	10-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance	10-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
At Point Fermin Beach, at Barbara Street Extended	12.3	15.7	18.4	--	--	--
At Point Fermin Beach, at Cabrillo Avenue Extended	13.8	18.2	21.2	--	--	--
Approximately 1,000 feet North of Point Fermin along Beach	17.4	24.7	28.3	--	--	--
At Beach, at Carolina Street Extended	16.5	22.7	26.1	--	--	--
At Beach, at Pacific Avenue Extended	15.5	21	24.3	--	--	--
At Cabrillo Beach, at 40th Street Extended	14.1	18.7	21.7	--	--	--
At Los Angeles Harbor	--	--	--	7.7	8.9	11.1
Catalina Avenue Extended at Beach	7.3	7.9	8.2	--	--	--
Approximately 1,500 feet North of Catalina Avenue Extended along Beach	8.8	10	10.7	--	--	--
At Hamilton Beach	7.9	8.8	9.2	--	--	--
At Sequit Point	11.5	14.3	16.9	--	--	--
At Arroyo Sequit Mouth	10.7	13	15.5	--	--	--
Approximately 800 feet East of Arroyo Sequit Mouth along Beach	11.5	14.3	17	--	--	--
Approximately 800 feet South of the Intersection of Nicholas Beach Road and Pacific Coast Highway	12	15.2	17.8	--	--	--
Approximately 2,400 feet West of Los Alisos Canyon Creek Mouth along Beach	14.3	19	22	--	--	--
At Los Alisos Canyon Creek Mouth	12	15.1	17.8	--	--	--

Table 11: Summary of Elevations for Wave Runup and Wave Setup (continued)

Flooding Source and Location	Wave Runup Elevation[1] (feet)			Wave Runup Elevation[1] (feet)		
	10-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance	10-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
Approximately 900 feet Southeast of the Intersection of Encinal Canyon Road and Pacific Coast Highway along Beach	12.3	15.7	18.4	--	--	--
At Encinal Canyon Creek Mouth	12.9	16.7	19.5	--	--	--
Approximately 250 feet South of the Intersection of Seal Level Drive and Roxanne Beach Road	10.9	13.3	15.8	--	--	--
At Lechuza Point	15.5	20.8	24.3	--	--	--
At Steep Hill Canyon Creek Mouth	13.1	17	19.9	--	--	--
At Trancas Creek	10.9	13.3	15.8	--	--	--
Approximately 200 feet West of Point Dume	12.4	16	18.8	--	--	--
At Point Dume	15.5	20.8	24.3	--	--	--
At Dume Cove, Approximately 500 feet Southeast of the Intersection of Dume Drive and Cliffside Drive	13.1	16.9	19.9	--	--	--
At Dume Cove, Approximately 400 feet South of the Intersection of Fernhill Drive and Cliffside Drive	12.1	15.3	18.1	--	--	--
At Dume Cove, Approximately 750 feet South of the Intersection of Grayfox Street and Cliffside Drive	13.1	16.9	19.9	--	--	--
At Paradise Cove, at Walnut Canyon	12.4	15.8	18.6	--	--	--
At Paradise Cove, Approximately 2,000 feet Northeast of Walnut Canyon Creek Mouth along Beach	15.8	20.8	24.3	--	--	--
At Paradise Cove, at Ramirez Canyon Mouth	11.5	14.3	16.9	--	--	--

Table 11: Summary of Elevations for Wave Runup and Wave Setup (continued)

Flooding Source and Location	Wave Runup Elevation[1] (feet)			Wave Runup Elevation[1] (feet)		
	10-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance	10-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
At Escondido Beach, at Escondido Canyon Mouth	10.7	12.9	15.5	--	--	--
At Escondido Beach, Approximately 200 feet East of the Intersection of Latigo Shore Place and Latigo Shore Drive	11.5	14.3	16.9	--	--	--
Approximately 500 feet West of Solstice Canyon Creek Mouth along Beach	13.9	18.3	21.3	--	--	--
At Solstice Canyon Creek Mouth	12.1	15.3	18.1	--	--	--
At Corral Beach, at Corral Canyon Creek Mouth	11.3	13.9	16.4	--	--	--
At Corral Beach, Approximately 250 feet South of the Intersection of Malibu Road and Pacific Coast Highway	13	16.9	19.6	--	--	--
Approximately 1,500 feet East of Corral Canyon Creek Mouth along Beach	13	16.9	19.6	--	--	--
At Puerco Beach, Approximately 200 feet South of the Intersection of Puerco Canyon Road and Malibu Road	11.3	13.9	16.4	--	--	--
At Puerco Beach, at Puerco Canyon Creek Mouth	13	16.9	19.6	--	--	--
At Amarillo Beach, Approximately 2,200 feet East of Marie Canyon Creek Mouth along Beach	11.3	13.9	16.4	--	--	--
At Amarillo Beach, Approximately 3,000 feet East of Marie Canyon Creek Mouth Along Beach	13	16.9	19.6	--	--	--

Table 11: Summary of Elevations for Wave Runup and Wave Setup (continued)

Flooding Source and Location	Wave Runup Elevation[1] (feet)			Wave Runup Elevation[1] (feet)		
	10-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance	10-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance
At Malibu Beach, Approximately 850 feet Southwest of Intersection of Malibu Road and Malibu Colony Drive	11.3	13.9	16.4	--	--	--
At Malibu Creek Mouth	10.6	12.8	15.2	7.7	8.9	11.1
At Las Flores Canyon Mouth	11.3	13.9	16.4	--	--	--
Approximately 2,500 feet East of Las Flores Canyon Mouth along Beach	11.6	14.5	17.1	--	--	--
Approximately 1,500 feet West of Piedra Gorda Canyon Creek Mouth along Beach	11.4	14.2	16.8	--	--	--
Approximately 100 feet South of the Intersection of Budwood Motorway and Pacific Coast Highway	11.9	14.9	17.6	--	--	--
At Topanga Canyon Mouth	11.4	14.1	16.7	--	--	--
At Marina Del Ray	--	--	--	7.7	8.9	11.1

[1] Average elevations given; elevation may vary within the area cited
-- Data not computed

Tsunamis

Tsunamis were computed using numerical models of the long wave equations describing tsunami behavior. The results were taken from the USACE Study which details the method used to compute tsunami behavior.

Tropical Cyclone Swells

Waves generated by a t ropical cyclone were determined using the JONSWAP spectrum with empirically derived shape and intensity parameters, which were correlated to radial position and wind speed. A cosine function centered about the local wind direction was used for the directional distribution function of the spectrum. The size of the tropical cyclone was defined by the radius at which the wind speed drops below 35 knots. Details of the node are discussed in "Methodology for Coastal Flooding in Southern California".

Flood elevations in areas studied by approximate methods were based on engineering judgment used in conjunction with topographic maps.

Levee Hazard Analysis

Some flood hazard information presented in prior FIRMs and in prior FIS reports for Los Angeles County and its incorporated communities was based on flood protection provided by levees. Based on the information available and the mapping standards of the National Flood Insurance Program at the time that the prior FISs and FIRMs were prepared, FEMA accredited the levees as providing protection from the flood that has a 1-percent-chance of being equaled or exceeded in any given year. For FEMA to continue to accredit the identified levees with providing protection from the base flood, the levees must meet the criteria of the Code of Federal Regulations, Title 44, Section 65.10 (44 CFR 65.10), titled "Mapping of Areas Protected by Levee Systems."

On August 22, 2005, FEMA issued Procedure Memorandum No. 34 - Interim Guidance for Studies Including Levees. The purpose of the memorandum was to help clarify the responsibility of community officials or other parties seeking recognition of a levee by providing information identified during a study/mapping project. Often, documentation regarding levee design, accreditation, and the impacts on flood hazard mapping is outdated or missing altogether. To remedy this, Procedure Memorandum No. 34 provides interim guidance on procedures to minimize delays in near-term studies/mapping projects, to help our mapping partners properly assess how to handle levee mapping issues.

While 44 CFR Section 65.10 documentation is being compiled, the release of more up-to-date FIRM panels for other parts of a community or county may be delayed. To minimize the impact of the levee recognition and certification process, FEMA issued Procedure Memorandum No. 43 - Guidelines for Identifying Provisionally Accredited Levees on March 16, 2007. These guidelines will allow issuance of preliminary and effective versions of FIRMs while the levee owners or communities are compiling the full documentation required to show compliance with 44 CFR Section 65.10. The guidelines also explain that preliminary FIRMs can be issued while providing the communities and levee owners with a specified timeframe to correct any maintenance deficiencies associated with a levee and to show compliance with 44 CFR Section 65.10.

FEMA contacted the communities within Los Angeles County to obtain data required under 44 CFR 65.10 to continue to show the levees as providing protection from the flood that has a 1-percent-chance of being equaled or exceeded in any given year.

FEMA understood that it may take time to acquire and/or assemble the documentation necessary to fully comply with 44 CFR 65.10. Therefore, FEMA put forth a process to provide the communities with additional time to submit all the necessary documentation. For a community to avail itself of the additional time, it had to sign an agreement with FEMA. Levees for which such agreements were signed are shown on the final effective FIRM as providing protection from the flood that has a 1-percent-chance of being equaled or exceeded in any given year and labeled as a Provisionally Accredited Levee (PAL). Communities have two years from the date of FEMA's initial coordination to submit to FEMA final accreditation data for all PALs. Following receipt of final accreditation data, FEMA will revise the FIS and FIRM as warranted.

FEMA coordinated with the USACE, the local communities, and other organizations to compile a list of levees that exist within Los Angeles County. Table 12, "List of Levees Requiring Flood Hazard Revisions" lists all levees shown on t he FIRM, to include PALs, for which corresponding flood hazard revisions were made.

Approximate analyses of "behind levee" flooding were conducted for all the levees in **Error! Reference source not found.**12 to indicate the extent of the "behind levee" floodplains. The methodology used in these analyses is discussed below.

The approximate levee analysis was conducted using information from existing hydraulic models (where applicable) and USGS topographic maps.

The extent of the 1-percent-annual-chance flood in the event of levee failure was determined. Base flood elevations and topographic information (where available) were used to estimate an approximate 1-percent-annual-chance floodplain and traced along the contour line representing the base flood elevation. If base flood elevations were not available they were estimated from effective FIRM maps and available information. Topographic features such as highways, railroads, and high ground were used to refine approximate floodplain boundary limits.

Several levees within Los Angeles County and its incorporated communities meet the criteria of the Code of Federal Regulations, Title 44, Section 65.10 (44 CFR 65.10), titled "Mapping of Areas Protected by Levee Systems." Table 133, "List of Certified and Accredited Levees" lists all levees shown on the FIRM that meet the requirements of 44 CFR 65.10 and have been determined to provide protection from the flood that has a 1-percent-chance of being equaled or exceeded in any given year.

Table 12: List of Levees Requiring Flood Hazard Revisions

Community	Flood Source	Levee Inventory ID	FIRM Panel	USACE Level
City of Santa Clarita	South Fork Santa Clara River Bouquet Canyon Creek Santa Clara River	2, 4,7,10, 13, 15, 26	06037C0820F	No
City of Compton City of Long Beach	Compton Creek	20b	06037C1955F	No
City of Cerritos City of Lakewood City of Hawaiian Gardens City of Long Beach	Coyote Creek	21	06037C1990F	No
City of Carson City of Los Angeles	Dominguez Channel	22a	06037C1935	No
City of Carson City of Los Angeles	Dominguez Channel	22b	06037C1965	No
City of Bell City of Cudahy City of Southgate City of Vernono	Los Angeles River	25a	06037C0100F	Yes
Los Angeles County	Undetermined	28a	06037C0100F	No
Los Angeles County	Undetermined	28c	06037C07 15F	No
Los Angeles County	Undetermined	28d	06037C0975F	No
City of Los Angeles	Undetermined	29	06037C1780F	No

Table 12: List of Levees Requiring Flood Hazard Revisions (continued)

Community	Flood Source	Levee Inventory ID	FIRM Panel	USACE Level
City of Bellflower City of Cerritos City of Downey City of Lakewood City of Long Beach City of Norwalk City of Pico Rivera	San Gabriel River	33	06037C1664F 06037C1668F 06037C1829F 06037C1830F 06037C1840F 06037C1841F 06037C1980F 06037C1988F 06037C1990F 06037C2076F	No

Table 13: List of Certified and Accredited Levees

Community	Flood Source	Levee Inventory ID	FIRM Panel	USACE Level
City of Santa Clarita	Santa Clara River Bouquet Canyon Creek South Fork Santa Clara River	5, 6, 14, 23	06037C0840F	No
City of Long Beach City of Southgate City of Paramount	Los Angeles River	25b	06037C1668F 06037C1664F 06037C1830F 06037C1820F 06037C1840F 06037C1980F 06037C1990F 06037C1988F 06037C2076F	No
City of Bell Gardens City of Commerce City of Downey City of Montebello City of Pico Rivera City of Southgate	Rio Hondo River	31	06037C1663F 06037C1664F 06037C1810F 06037C1820F 06037C1830F	No

3.3 Vertical Datum

All FIS reports and FIRMs are referenced to a specific vertical datum. The vertical datum provides a starting point against which flood, ground, and structure elevations can be referenced and compared. Until recently, the standard vertical datum used for newly created or revised FIS reports and FIRMs was the National Geodetic Vertical Datum of 1929 (NGVD). With the completion of the North American Vertical Datum of 1988 (NAVD), many FIS reports and FIRMs are now prepared using NAVD as the referenced vertical datum.

All flood elevations shown in this FIS report and on the FIRM are referenced to the NAVD 88. These flood elevations must be compared to structure and ground elevations referenced to the same vertical datum. It is important to note that adjacent counties may be referenced to NGVD, which may result in differences in base flood elevations across the corporate limits between the communities.

For information regarding conversion between the NGVD and NAVD, see the FEMA publication entitled *Converting the National Flood Insurance Program to the North American Vertical Datum of 1988* (Reference 6), visit the National Geodetic Survey website at www.ngs.noaa.gov, or contact the National Geodetic Survey at the following address:

NGS Information Services
NOAA, N/NGS12
National Geodetic Survey
SSMC-3, #9202
1315 East-West Highway
Silver Spring, Maryland 20910-3282
(301) 713-3242

Temporary vertical monuments are often established during the preparation of a flood hazard analysis for the purpose of establishing local vertical control. Although these monuments are not shown on the FIRM, they may be found in the Technical Support Data Notebook associated with the FIS report and FIRM for this community. Interested individuals may contact FEMA to access these data.

The conversion factor for each stream studied by detailed methods is shown below in **Error! Reference source not found.**14, “Stream Conversion Factor.”

Table 14: Stream Conversion Factors

Stream Name	Elevation (feet NAVD above NGVD)
Amargosa Creek	+2.8
Anaverde Creek	+2.8
Avalon Canyon	+2.8

Stream Name	Elevation (feet NAVD above NGVD)
Big Rock Wash	+2.8
Cheseboro Creek	+2.9
Cold Creek	+2.9
Dark Canyon	+2.9
Dry Canyon	+2.9
Escondido Canyon	+2.9
Flow Along Empire Avenue	+2.8
Flowline No. 1	+2.8
Garapito Creek	+2.9
Hacienda Creek	+2.8
Kagel Canyon	+2.8
La Mirada Creek	+2.8
Lake Street Overflow	+2.8
Las Flores Canyon	+2.9
Las Virgenes Creek	+2.9
Liberty Canyon	+2.9
Lindero Canyon above confluence with Medea Creek	+2.9
Lindero Canyon above Lake Lindero	+2.9
Little Rock Wash - Profile A	+2.8
Little Rock Wash - Profile B	+2.8
Little Rock Wash - Profile C	+2.8
Lobo Canyon	+2.9
Lockheed Drain Channel	+2.8
Lopez Canyon Channel	+2.8
Los Angeles River left overbank path 2	+2.8
Los Angeles River right overbank path 1	+2.8
Los Angeles River right overbank path 2	+2.8
Malibu Creek	+2.9
Medea Creek	+2.9

Stream Name	Elevation (feet NAVD above NGVD)
Medea Creek (above Ventura Freeway)	+2.9
Mill Creek	+2.8
North Overflow	+2.8
Old Topanga Canyon	+2.9
Overflow Area of Lockheed Drain Channel	+2.8
Overflow Area of Lockheed Storm Drain	+2.8
Palo Comando Creek	+2.9
Ramirez Canyon	+2.9
Rio Hondo River left overbank path 3	+2.8
Rio Hondo River left overbank path 5	+2.8
Rio Hondo River left overbank path 6	+2.8
Rustic Canyon	+2.8
Santa Maria Canyon	+2.9
Stokes Canyon	+2.9
Topanga Canyon	+2.9
Trancas Creek	+2.9
Triunfo Creek	+2.9
Unnamed Canyon (Serra Retreat Area)	+2.9
Upper Los Angeles River left overbank	+2.8
Weldon Canyon	+2.9
Zuma Canyon	+2.9

4.0 FLOODPLAIN MANAGEMENT APPLICATIONS

The NFIP encourages State and local governments to adopt sound floodplain management programs. To assist in this endeavor, each FIS report provides 1-percent-annual-chance floodplain data, which may include a combination of the following: 10-, 2-, 1-, and 0.2-percent-annual-chance flood elevations; delineations of the 1- and 0.2-percent-annual-chance floodplains; and a 1-percent-annual-chance floodway. This information is presented on the FIRM and in many components of the FIS report, including Flood Profiles, Floodway Data tables, and Summary of Stillwater Elevation tables. Users should reference the data presented in the FIS report as well as additional information that may be available at the local community map repository before making flood elevation and/or floodplain boundary determinations.

4.1 Floodplain Boundaries

To provide a national standard without regional discrimination, the 1-percent-annual-chance flood has been adopted by FEMA as the base flood for floodplain management purposes. The 0.2-percent-annual-chance flood is employed to indicate additional areas of flood risk in the community. For each stream studied by detailed or limited detailed methods, the 1- and 0.2-percent-annual-chance floodplain boundaries have been delineated using the flood elevations determined at each cross section.

The 1- and 0.2-percent-annual-chance floodplain boundaries for streams studied by detailed methods are shown on t he FIRM. On this map, the 1-percent-annual-chance floodplain boundary corresponds to the boundary of the areas of special flood hazards (Zones A, AE, V and VE), and the 0.2-percent-annual-chance floodplain boundary corresponds to the boundary of areas of moderate flood hazards. In cases where the 1- and 0.2-percent-annual-chance floodplain boundaries are close together, only the 1-percent-annual-chance floodplain boundary has been shown. Small areas within the floodplain boundaries may lie above the flood elevations, but cannot be shown due to limitations of the map scale and/or lack of detailed topographic data.

For streams studied by approximate methods, only the 1-percent-annual-chance floodplain boundary is shown on the FIRM (Exhibit 2).

4.2 Floodways

Encroachment on floodplains, such as structures and fill, reduces flood-carrying capacity, increases flood heights and velocities, and increases flood hazards in areas beyond the encroachment itself. One aspect of floodplain management involves balancing the economic gain from floodplain development against the resulting increase in flood hazard. For purposes of the NFIP, a floodway is used as a tool to assist local communities in this aspect of floodplain management. Under this concept, the area of the 1-percent-annual-chance floodplain is divided into a floodway and a floodway fringe. The floodway is the channel of a stream, plus any adjacent floodplain areas, that must be kept free of encroachment so that the base flood can be carried without substantial increases in flood heights. Minimum Federal standards limit such increases to 1 f oot, provided that hazardous velocities are not produced. The floodways in this study are presented to local agencies as minimum standards that can be adopted directly or that can be used as a basis for additional floodway studies.

The floodways presented in this study were computed for certain stream segments on the basis of equal-conveyance reduction from each side of the floodplain. Floodway widths were computed at cross sections. Between cross sections, the floodway boundaries were interpolated. The results of the floodway computations are tabulated for selected cross sections and provided in Table 15: , “Floodway Data.” The computed floodway is shown on the FIRM (Exhibit 2). In cases where the floodway and 1-percent-annual-chance floodplain boundaries are either close together or collinear, only the floodway boundary is shown on the FIRM.

Near the mouth of streams studied in detail, floodway computations were made without regard to flood elevations on the receiving water body. Therefore, “Without Floodway” elevations presented in Table 15: for certain downstream cross sections are lower than the regulatory flood elevations in that area, which must take into account the 1-percent-annual-chance flooding due to backwater from other sources.

Encroachment into areas subject to inundation by floodwaters having hazardous velocities aggravates the risk of flood damage and heightens potential flood hazards by further increasing velocities. A listing of stream velocities at selected cross sections is provided in the Floodway Data Table. In order to reduce the risk of property damage in areas where the stream velocities are high, the community may wish to restrict development in areas outside the floodway.

Los Angeles County

In this study, Trancas, Malibu, Garapito, Cold, Cheseboro, Palo Comado, Medea, Lindero, Triunfo, Mill, and Hacienda Creeks; Zuma, Ramirez, Escondido, Unnamed (Serra Retreat Area), Las Flores, Topanga, Santa Maria, Old Topanga, Dark, Logo, Stokes, Dry, and Liberty Canyons; and Lopez Canyon and Kagel Canyon Channels have relatively high velocity discharges which have historically eroded the main channel. This results in unpredictable meandering of flood flows and presents a severe hazard to structures located within the floodplain. In addition, flooding depths often preclude practical floodproofing of structures.

City of Agoura Hills

In Agoura Hills, Cheseboro, Palo Comado, Medea, and Lindero Canyon channels have relatively high-velocity discharges which have historically eroded the main channel. This results in unpredictable meandering of flood flows and presents a severe hazard to structures located within the floodplain. In addition, flooding depths often preclude practical floodproofing of structures. For these reasons the 1-percent annual chance floodplain is designated as the floodway.

No floodways were computed for Medea Creek as part of the 1998 restudy due to the high degree of development in this area. However, the 1-percent annual chance floodplain is designated as the floodway along Medea Creek due to the relatively high velocity discharges.

City of Avalon

In Avalon, this concept of encroachment is not appropriate. In the densely developed area, the 1-foot rise in flood height that would result from allowing encroachment in the floodway fringe would increase the flood hazard to many existing properties. However, development of the few vacant lots between Tremont and Beacon Streets would not increase the base flood elevations because those lots were assumed to be developed for this study. In the open area upstream of Tremont Street, new development would greatly increase the flood hazard to the developed area downstream of Tremont Street, unless a channel was built that would adequately collect and convey the base flood through the city to the ocean. In the reach downstream of Beacon Street, development of the plaza area would increase the base flood and, consequently, the flood hazard to existing properties. For these reasons, it is recommended that the entire Avalon flood plain be designated as the floodway, thus prohibiting development that would cause any increase in water-surface elevation.

Cities of Bellflower, Carson, Compton, Downey, Gardena, Lakewood, Long Beach, Lynwood, Montebello, Paramount, Pico Rivera, South Gate, Whittier

In this study the Los Angeles River channel and the Rio Hondo channel carry generally high velocities. The density of development within overbank areas in these communities affected by potential overflow of the Los Angeles River or Rio Hondo will limit overbank flow to relatively low velocities, due to relatively flat gradients and large open space available within the floodplain encroachments. For these reasons, floodways were not computed for this study.

City of Burbank

A regulatory floodway was not computed because the flooded area is fully developed and the degree of flooding meets the Zones AO and AH shallow flooding criteria.

Floodways for the Lockheed Drain Channel were not determined as part of this restudy. Due to the lack of capacity of the storm-drain channel, floodway limits cannot be defined in the study area because any increase in water-surface elevation will result in increased overflows and flooding in other areas.

City of Culver City

The special flood hazard areas in Culver City are areas of shallow flooding; therefore, the concept of a floodway was not applied to this community.

City of La Mirada

The floodway concept was explained to the City Planning Director, at a meeting held on September 11, 1978. The city recognizes this flood hazard area and has already adopted regulatory zoning and building restrictions on a portion of the flooded area. At the intermediate coordination meeting held on October 3, 1978, the City Planning Director indicated that the city is prepared to adopt ordinances to restrict development in the remainder of the flooded area; therefore, the floodway concept was not applied to the City of La Mirada. This has been approved by the FIA.

City of Los Angeles

The regulatory floodway concept was explained to representatives of the City Engineer. It was emphasized that in natural watercourses in the city, high-velocity flows have historically eroded the main channel and resulted in unpredictable meandering of flood flows. The city recognizes the highly erosive nature of these streams and agrees with the conclusion that, in the case of Weldon, Kagel, and Rustic Canyons, the entire 1-percent annual chance flood plain should be delineated as a floodway. The results of these computations are tabulated at selected cross sections for each stream segment for which a floodway was computed.

The floodway concept was not applied to Big Tujunga, Little Tujunga, or Pacoima Washes where alluvial fan zones are designated. Also, floodways were not computed in areas where flooding is caused by ponding water.

City of Lancaster

For this study, floodways have not been determined because the special flood-hazard areas in Lancaster are areas of alluvial fan shallow flooding, or have poorly defined channels.

City of Palmdale

In areas of high velocities and potential subcritical flow conditions, encroachment analyses were performed to determine floodway boundaries and to limit both the increase in water-surface elevation and energy grade lines to maximum of 1 foot.

The floodplain and floodway boundaries, as determined by hydrologic and hydraulic analyses, have been delineated on the California DWR horizontal-scale orthophoto topographic mapping at a scale of 1" = 400', with a 5-foot contour interval (State of California, Department of Water Resources, April 9, 1990).

In this restudy, the floodway for Anaverde Creek was computed on the basis of equal-conveyance reduction from each side of the floodplain. Floodway widths were computed at cross sections. Between cross sections, the floodway boundaries were interpolated.

Floodplain boundaries were defined based on BFEs as determined by subcritical flow analyses. In channel reaches were subcritical flow conditions could occur, the BFEs were based on critical depth.

High-channel velocities and localized high-overbank velocities should be considered significant floodplain management factors. Channel velocities exceeded potential erosive magnitudes up to a maximum of over 13 feet per second (fps). Overbank velocities reached up to 7 fps.

City of Palos Verdes Estates

The floodways for Unnamed Streams were computed on the basis of equal-conveyance reduction from each side of the floodplain. Floodway widths were computed at cross sections. Between cross sections, the floodway boundaries were interpolated.

City of Redondo Beach

The floodway is the channel of a stream plus any adjacent flood plain areas that must be kept free of encroachment in order that the 1-percent annual chance flood may be carried without substantial increases in flood heights. A floodway generally is not applicable in areas where the dominant source of flooding is from coastal waters; thus, no floodway was computed for this study.

City of Santa Clarita

In the Santa Clarita Valley, flood flows sometimes unpredictably meander, presenting a severe hazard to structures located within the floodplains. Therefore, no floodways were computed for this study.

City of Santa Fe Springs

The special flood hazard areas shown with constant elevations on the map are caused by ponding water; therefore, the concept of a floodway was not applicable. The flooding northeast of the intersection of Pioneer Boulevard (Flowline No. 1) is caused by flowing water. The floodway concept was explained to the City Director of Public Works (the City Engineer) at a meeting on April 25, 1978. The city recognizes this flood-hazard area and indicated that development of the property will not be permitted until the flood hazard is removed. Therefore, the floodway concept was not applied at this location.

City of Torrance

The special flood hazard areas in the city are caused by ponding and shallow flooding; therefore, the concept of a floodway was not applied to the community.

City of West Hollywood

For this study, floodways have not been determined because areas studied within the community exhibit shallow flooding.

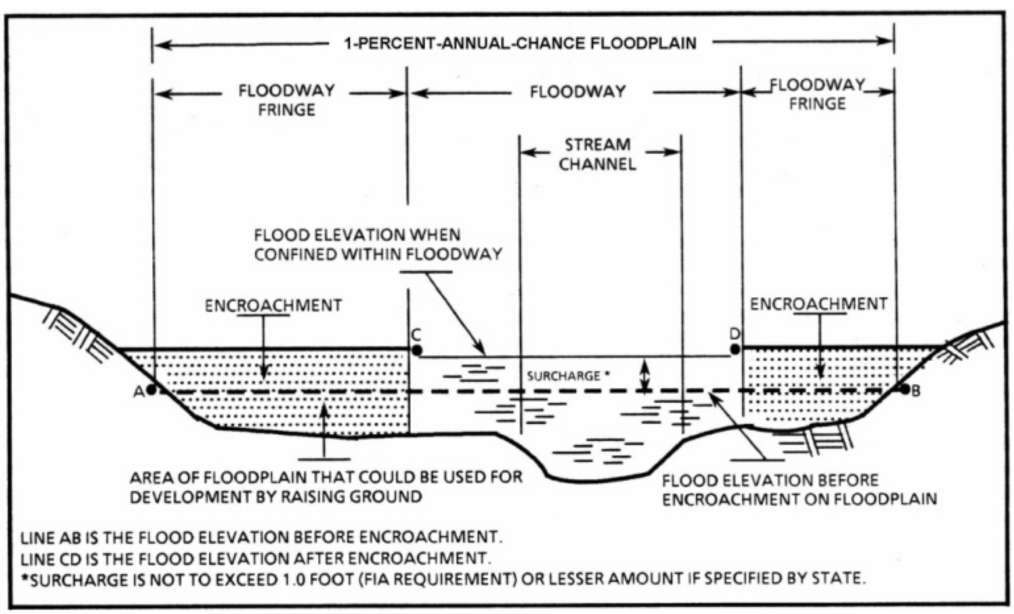

Figure 1. Floodway Schematic

The area between the floodway and 1-percent-annual-chance floodplain boundaries is termed the floodway fringe. The floodway fringe encompasses the portion of the floodplain that could be completely obstructed without increasing the water-surface elevation (WSEL) of the base flood more than 1 foot at any point. Typical relationships between the floodway and the floodway fringe and their significance to floodplain development are shown in Figure 1.

5.0 INSURANCE APPLICATIONS

For flood insurance rating purposes, flood insurance zone designations are assigned to a community based on the results of the engineering analyses. These zones are as follows:

Zone A

Zone A is the flood insurance rate zone that corresponds to the 1-percent-annual-chance floodplains that are determined in the FIS report by approximate methods. Because detailed hydraulic analyses are not performed for such areas, no base (1-percent-annual-chance) flood elevations (BFEs) or depths are shown within this zone.

Zone AE

Zone AE is the flood insurance rate zone that corresponds to the 1-percent-annual-chance floodplains that are determined in the FIS report by detailed methods. Whole-foot BFEs derived from the detailed hydraulic analyses are shown at selected intervals within this zone.

Zone AH

Zone AH is the flood insurance rate zone that corresponds to the areas of 1-percent-annual-chance shallow flooding (usually areas of ponding) where average depths are between 1 and 3 feet. Whole-foot BFEs derived from the detailed hydraulic analyses are shown at selected intervals within this zone.

Zone AO

Zone AO is the flood insurance rate zone that corresponds to the areas of 1-percent-annual-chance shallow flooding (usually sheet flow on sloping terrain) where average depths are between 1 and 3 feet. Average whole-foot base flood depths derived from the detailed hydraulic analyses are shown within this zone.

Zone V

Zone V is the flood insurance risk zone that corresponds to the 1-percent-annual-chance coastal floodplains that have additional hazards associated with storm waves. Because approximate hydraulic analyses are performed for such areas, no Base Flood Elevations are shown within this zone.

Zone VE

Zone VE is the flood insurance rate zone that corresponds to the 1-percent annual chance coastal floodplains that have additional hazards associated with storm waves. Whole-foot base flood elevations derived from the detailed hydraulic analyses are shown at selected intervals within this zone.

Zone X

Zone X is the flood insurance rate zone that corresponds to areas outside the 0.2-percent-annual-chance floodplain, areas within the 0.2-percent-annual-chance floodplain, areas of 1-percent-annual-chance flooding where average depths are less than 1 foot, areas of 1-percent-annual-chance flooding where the contributing drainage area is less than 1 square mile (sq. mi.), and areas protected from the base flood by levees. No BFEs or depths are shown within this zone.

Zone D

Zone D is the flood insurance rate zone that corresponds to unstudied areas where flood hazards are undetermined, but possible.

FLOODING SOURCE		FLOODWAY			1-PERCENT-ANNUAL-CHANCE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Anaverde Creek								
A	1,220	104	354	10.5	2,744.4	2,744.4	2,744.4	0.0
B	1,410	105	342	10.9	2,745.2	2,745.2	2,745.2	0.0
C	2,110	310	535	7.0	2,756.3	2,756.3	2,756.4	0.1
D	2,400	285	403	9.3	2,760.6	2,760.6	2,761.0	0.4
E	3,020	579[2]	596	6.3	2,768.9	2,768.9	2,768.9	0.0
F	4,090	257[2]	436	8.6	2,785.3	2,785.3	2,785.9	0.6
G	4,371	480	549	6.8	2,800.2	2,800.2	2,800.7	0.5
H	4,476	480	3,261	1.1	2,801.2	2,801.2	2,801.9	0.7
I	5,251	140	391	9.5	2,803.2	2,803.2	2,803.2	0.0
J	8,501	57[3]	292	12.4	2,859.5	2,859.5	2,859.5	0.0
K	8,871	53[3]	329	11.0	2,869.2	2,869.2	2,869.2	0.0
L	9,261	80[3]	372	9.8	2,875.4	2,875.4	2,875.4	0.0
M	9,711	105[3]	488	7.4	2,879.8	2,879.8	2,880.3	0.5
N	10,191	127[3]	342	9.4	2,886.7	2,886.7	2,886.7	0.0
O	12,251	139[3]	549	5.8	2,905.7	2,905.7	2,905.7	0.0
P	12,581	139[3]	432	7.4	2,907.6	2,907.6	2,907.6	0.0
Q	13,291	220	1,008	3.2	2,914.0	2,914.0	2,914.1	0.1
R	13,561	220	1,401	2.3	2,914.4	2,914.4	2,914.6	0.2
S	13,941	250	997	3.2	2,914.6	2,914.6	2,914.9	0.3
T	14,381	139	333	7.3	2,916.2	2,916.2	2,916.6	0.4
U	18,091	115	812	3.0	2,928.4	2,928.4	2,928.5	0.1
V	18,341	31	300	8.1	2,928.6	2,928.6	2,928.7	0.1
W	18,611	31	272	9.0	2,931.8	2,931.8	2,931.8	0.0

[1] Feet above Division Street
[2] Area of stilling basin -- no floodway determined between sections
[3] Lies entirely outside corporate limits of City of Palmdale

TABLE 15

FEDERAL EMERGENCY MANAGEMENT AGENCY
LOS ANGELES COUNTY, CALIFORNIA
AND INCORPORATED AREAS

FLOODWAY DATA

ANAVERDE CREEK

FLOODING SOURCE		FLOODWAY			1-PERCENT-ANNUAL-CHANCE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Kagel Canyon								
A	650[2]	100	149	7.23	1,150.8	1,150.8	1,150.8	0.0
Rustic Canyon								
A	4,164[3]	60	216	9.63	192.8	192.8	192.8	0.0
B	4,780[3]	120	243	8.29	204.8	204.8	204.8	0.0
C	5,400[3]	150	149	7.23	219.8	219.8	219.8	0.0
D	6,130[3]	65	230	7.97	235.6	235.6	235.6	0.0
E	7,350[3]	29	180	9.81	259.2	259.2	259.2	0.0
F	8220[3]	49	141	12.01	281.6	281.6	281.6	0.0
Weldon Canyon								
A	1,290[1]	70	210	5.40	1,377.9	1,377.9	1,377.9	0.0

[1] Feet Upstream of Golden State Freeway Bridge

[2] Feet Upstream from Northwest Edge of Osbourne Street

[3] Feet Upstream of Latimer Road

TABLE 15

FEDERAL EMERGENCY MANAGEMENT AGENCY

LOS ANGELES COUNTY, CALIFORNIA

AND INCORPORATED AREAS

FLOODWAY DATA

KAGEL CANYON - RUSTIC CANYON - WELDON CANYON

FLOODING SOURCE		FLOODWAY			1-PERCENT-ANNUAL-CHANCE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A	342	14	19	6.7	149.4	149.4	149.4	0.0
B	434	30	24	5.2	174.5	174.5	174.5	0.0
C	482	41	27	4.6	177.1	177.1	177.1	0.0
D	539	28	24	5.3	182.6	182.6	183.4	0.8
E	586	35	26	4.9	185.2	185.2	185.3	0.1
F	888	32	25	5.0	196.3	196.3	196.3	0.0
G	934	39	27	4.7	199.2	199.2	199.2	0.0
H	960	37	26	4.8	203.2	203.2	203.2	0.0
I	1,040	27	24	5.3	207.8	207.8	208.1	0.3
J	1,256	58	30	4.2	213.4	213.4	213.6	0.2
K	1,582	60	70	1.8	216.2	216.2	216.2	0.0
L	1,722	26	9	3.4	233.7	233.7	233.7	0.0
M	1,823	35	10	3.1	240.4	240.4	240.4	0.0
N	2,054	29	40	0.8	246.7	246.7	247.3	0.6
O	2,373	11	7	4.6	257.9	257.9	257.9	0.0
P	2,485	32	10	3.2	268.7	268.7	268.7	0.0
Q	2,506	19[2]	2	1.8	272.1	272.1	272.1	0.0
R	2,700	9[2]	2	1.3	277.8	277.8	277.8	0.0
S	2,858	34	90	9.2	283.9	283.9	283.9	0.0
T	3,031	75	122	6.8	293.3	293.3	293.3	0.0
U	3,246	24	63	9.2	300.6	300.6	300.6	0.0
V	3,699	21	60	9.6	326.3	326.3	326.3	0.0
W	3,774	33	70	8.3	336.2	336.2	336.2	0.0
X	3,946	22	61	9.5	338.6	338.6	338.6	0.0
Y	4,068	27	65	8.9	350.7	350.7	350.7	0.0
Z	4,261	36	72	8.0	355.6	355.6	355.6	0.0

[1] Feet above Pacific Ocean

[2] 1% annual chance flood discharge contained in structure

TABLE 15

FEDERAL EMERGENCY MANAGEMENT AGENCY
LOS ANGELES COUNTY, CALIFORNIA
AND INCORPORATED AREAS

FLOODWAY DATA

UNNAMED STREAM MAIN REACH

FLOODING SOURCE		FLOODWAY			1-PERCENT-ANNUAL-CHANCE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
AA	4380	55	83	7.0	369.4	369.4	369.5	0.1
AB	4,434	35	72	8.1	372.0	372.0	372.0	0.0
AC	4,490	33	156	3.7	373.1	373.1	373.1	0.0
AD	4,565	8	1	2.3	379.3	379.3	379.3	0.0
AE	5,024	16	4	0.8	410.4	410.4	410.4	0.0
AF	5,087	37	18	4.0	416.7	416.7	416.7	0.0
AG	5,136	24	15	4.6	422.9	422.9	422.9	0.0
AH	5,153	39	18	3.9	428.5	428.5	428.5	0.1
AI	5,177	48	19	3.6	429.3	429.3	429.3	0.0
AJ	5,520	18	2	1.7	472.0	472.0	472.1	0.1
AK	5,533	7	2	1.3	472.4	472.4	472.4	0.0
AL	5,626	9	1	2.2	488.1	488.1	488.1	0.0
AM	5,648	44	18	3.7	497.9	497.9	497.9	0.0
AN	5,730	54	35	4.7	521.6	521.6	521.6	0.0
AO	5,753	33	30	5.5	523.5	523.5	523.5	0.0
AP	5,792	30	29	5.6	523.8	523.8	523.9	0.1
AQ	5,934	30	12	1.8	526.9	526.9	526.9	0.0

[1] Feet above Pacific Ocean

TABLE 15

FEDERAL EMERGENCY MANAGEMENT AGENCY
LOS ANGELES COUNTY, CALIFORNIA
AND INCORPORATED AREAS

FLOODWAY DATA

UNNAMED STREAM MAIN REACH

FLOODING SOURCE		FLOODWAY			1-PERCENT-ANNUAL-CHANCE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A	57	30	23	5.0	380.3	380.3	380.3	0.0
B	239	23	46	2.6	388.1	388.1	388.4	0.3
C	314	27	22	5.3	399.3	399.3	399.3	0.0
D	366	25	22	5.4	410.1	410.1	410.3	0.2
E	546	18	20	6.0	421.7	421.7	421.8	0.1
F	799	33	24	4.9	441.6	441.6	441.6	0.0
G	935	29	23	5.1	457.0	457.0	457.0	0.0
H	1,009	18	6	3.3	458.9	458.9	458.9	0.0
I	1,051	29	25	5.3	463.7	463.7	463.7	0.0
J	1,145	25	24	5.6	493.2	493.2	493.2	0.0
K	1,227	22	23	5.8	508.2	508.2	508.2	0.0
L	1,343	15	21	6.6	514.4	514.4	514.4	0.0
M	1,374	26	24	5.6	525.7	525.7	525.7	0.0
N	1,400	23	57	2.4	526.3	526.3	526.3	0.0

[1] Feet above confluence with Unnamed Stream Main Reach

TABLE 15

FEDERAL EMERGENCY MANAGEMENT AGENCY
LOS ANGELES COUNTY, CALIFORNIA
AND INCORPORATED AREAS

FLOODWAY DATA

UNNAMED STREAM TRIBUTARY 1

FLOODING SOURCE		FLOODWAY			1-PERCENT-ANNUAL-CHANCE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A	207	23	26	6.1	284.8	284.8	284.8	0.0
B	623	31	29	5.5	322.6	322.6	322.7	0.1
C	744	39	31	5.1	334.4	334.4	334.4	0.0
D	803	44	46	3.5	335.6	335.6	335.7	0.1
E	913	24	26	6.0	344.2	344.2	344.2	0.0
F	1,699	27	28	5.8	395.9	395.9	395.9	0.0
G	2,039	33	29	5.4	431.2	431.2	431.2	0.0
H	2,405	26	49	7.8	455.1	455.1	455.2	0.1
I	2,523	24	54	7.1	470.0	470.0	470.1	0.1
J	2,569	29	91	4.2	470.9	470.9	471.5	0.6
K	2,674	35	53	7.1	482.7	482.7	482.7	0.0
L	2,692	30	51	7.4	487.8	487.8	487.8	0.0
M	2,822	52	90	3.6	498.0	498.0	498.4	0.4
N	2,943	35	130	2.8	498.3	498.3	498.5	0.2

[1] Feet above confluence with Unnamed Stream Main Reach

TABLE 15

FEDERAL EMERGENCY MANAGEMENT AGENCY
LOS ANGELES COUNTY, CALIFORNIA
AND INCORPORATED AREAS

FLOODWAY DATA

UNNAMED STREAM TRIBUTARY 2

6.0 FLOOD INSURANCE RATE MAP

The FIRM is designed for flood insurance and floodplain management applications.

For flood insurance applications, the map designates flood insurance rate zones as described in Section 5.0 and, in the 1-percent-annual-chance floodplains that were studied by detailed methods, shows selected whole-foot BFEs or average depths. Insurance agents use zones and BFEs in conjunction with information on structures and their contents to assign premium rates for flood insurance policies.

For floodplain management applications, the map shows by tints, screens, and symbols, the 1- and 0.2-percent-annual-chance floodplains, floodways, and the locations of selected cross sections used in the hydraulic analyses and floodway computations.

The countywide FIRM presents flooding information for the entire geographic area of Los Angeles County. Previously, FIRMs were prepared for each incorporated community and the unincorporated areas of the County identified as flood-prone. This countywide FIRM also includes flood-hazard information that was presented separately on F lood Boundary and Floodway Maps (FBFMs), where applicable. Historical data relating to the maps prepared for each community are presented in Table 16, Community Map History.

COMMUNITY NAME	INITIAL IDENTIFICATION	FLOOD HAZARD BOUNDARY MAP REVISIONS DATE(S)	FIRM EFFECTIVE DATE	FIRM REVISIONS DATE(S)
Agoura Hills, City of	March 4, 1986	None	March 4, 1986	December 18, 1986 August 3, 1998
Alhambra, City of[1,2]	None	None	None	None
Arcadia, City of[2]	None	None	None	None
Artesia, City of[1,2]	None	None	None	None
Avalon, City of	October 8, 1976	None	September 29, 1978	November 1, 1985
Azusa, City of[2]	None	None	None	None
Baldwin Park, City of[1,2]	None	None	None	None
Bell Gardens, City of[2]	None	None	None	None
Bell, City of[1,2]	None	None	None	None
Bellflower, City of	July 6, 1998	None	July 6, 1998	None
Beverly Hills, City of[1,2]	None	None	None	None
Bradbury, City of[2]	None	None	None	None
Burbank, City of	July 19, 1974	September 26, 1975	March 16, 1981	January 20, 1999

[1]Non-floodprone Community
[2]This community did not have a FIRM prior to the first countywide FIRM for Los Angeles County

TABLE 16

FEDERAL EMERGENCY MANAGEMENT AGENCY

LOS ANGELES, CA
AND INCORPORATED AREAS

COMMUNITY MAP HISTORY

COMMUNITY NAME	INITIAL IDENTIFICATION	FLOOD HAZARD BOUNDARY MAP REVISIONS DATE(S)	FIRM EFFECTIVE DATE	FIRM REVISIONS DATE(S)
Calabasas, City of	December 2, 1980 (Los Angeles County)	None	December 2, 1980 (Los Angeles County)	None
Carson, City of	July 6, 1998	None	July 6, 1998	None
Cerritos, City of[2]	None	None	None	None
Claremont, City of	November 20, 2000	None	November 20, 2000	July 2, 2004
Commerce, City of[2]	None	None	None	None
Compton, City of	July 6, 1998	None	July 6, 1998	None
Covina, City of[2]	None	None	None	None
Cudahy, City of[2]	None	None	None	None
Culver City, City of	June 28, 1974	October 31, 1975 September 3, 1976	February 1, 1980	None
Diamond Bar, City of	December2, 1980 (Los Angeles County)	None	December 2, 1980 (Los Angeles County)	None
Downey, City of	July 6, 1998	None	July 6, 1998	None
Duarte, City of[2]	None	None	None	None

[1]Non-floodprone Community

[2]This community did not have a FIRM prior to the first countywide FIRM for Los Angeles County

TABLE 16

FEDERAL EMERGENCY MANAGEMENT AGENCY

LOS ANGELES, CA
AND INCORPORATED AREAS

COMMUNITY MAP HISTORY

COMMUNITY NAME	INITIAL IDENTIFICATION	FLOOD HAZARD BOUNDARY MAP REVISIONS DATE(S)	FIRM EFFECTIVE DATE	FIRM REVISIONS DATE(S)
El Monte, City of[1,2]	None	None	None	None
El Segundo, City of[2]	None	None	None	None
Gardena, City of	July 6, 1998	None	July 6, 1998	None
Glendale, City of[2]	None	None	None	None
Glendora, City of[2]	None	None	None	None
Hawaiian Gardens, City of[1,2]	None	None	None	None
Hawthorne, City of[1]	December 4, 1979	None	December 4, 1979	None
Hermosa Beach, City of[2]	None	None	None	None
Hidden Hills, City of	September 7, 1984	None	September 7, 1984	November 21, 2001 January 19, 2006
Huntington Park, City of[1,2]	None	None	None	None
Industry, City of[2]	None	None	None	None
Inglewood, City of[1,2]	None	None	None	None
Irwindale, City of[1,2]	None	None	None	None

[1]Non-floodprone Community

[2]This community did not have a FIRM prior to the first countywide FIRM for Los Angeles County

TABLE 16

FEDERAL EMERGENCY MANAGEMENT AGENCY

LOS ANGELES, CA
AND INCORPORATED AREAS

COMMUNITY MAP HISTORY

COMMUNITY NAME	INITIAL IDENTIFICATION	FLOOD HAZARD BOUNDARY MAP REVISIONS DATE(S)	FIRM EFFECTIVE DATE	FIRM REVISIONS DATE(S)
La Canada Flintridge, City of[2]	None	None	None	None
La Habra Heights, City of[2]	None	None	None	None
La Mirada, City of	June 28, 1974	October 10, 1975 December 10, 1976	July 2, 1980	None
La Puente, City of[1,2]	None	None	None	None
La Verne, City of[2]	None	None	None	None
Lakewood, City of	July 6, 1998	None	July 6, 1998	None
Lancaster, City of	September 11, 1979	None	January 6, 1982	None
Lawndale, City of[1,2]	None	None	None	None
Lomita, City of[1,2]	None	None	None	None
Long Beach, City of	July 26, 1974	July 11, 1978	September 15, 1983	July 6, 1998
Los Angeles, City of	December 13, 1977	April 8, 1980	December 2, 1980	February 4, 1987 July 6, 1998 May 4, 1999

[1]Non-floodprone Community

[2]This community did not have a FIRM prior to the first countywide FIRM for Los Angeles County

TABLE 16

FEDERAL EMERGENCY MANAGEMENT AGENCY

LOS ANGELES, CA
AND INCORPORATED AREAS

COMMUNITY MAP HISTORY

COMMUNITY NAME	INITIAL IDENTIFICATION	FLOOD HAZARD BOUNDARY MAP REVISIONS DATE(S)	FIRM EFFECTIVE DATE	FIRM REVISIONS DATE(S)
Lynwood, City of	June 28, 1974	November 21, 1975	April 15, 1980	July 6, 1998
Malibu, City of[2]	None	None	None	None
Manhattan Beach, City of[2]	None	None	None	None
Maywood, City of[1,2]	None	None	None	None
Monrovia, City of[2]	None	None	None	None
Montebello, City of	June 28, 1974	December 19, 1975	March 18, 1980	July 6, 1998
Monterey Park, City of[1,2]	None	None	None	None
Norwalk, City of[2]	None	None	None	None
Palmdale, City of	October 18, 1974	December 24, 1976	January 6, 1982	June 18, 1987 March 30, 1998
Palos Verdes Estates, City of	September 7, 1984	None	September 7, 1984	November 21, 2001 July 2, 2004
Paramount, City of	July 6, 1998	None	July 6, 1998	None
Pasadena, City of[2]	None	None	None	None

[1]Non-floodprone Community

[2]This community did not have a FIRM prior to the first countywide FIRM for Los Angeles County

TABLE 16

FEDERAL EMERGENCY MANAGEMENT AGENCY

LOS ANGELES, CA
AND INCORPORATED AREAS

COMMUNITY MAP HISTORY

COMMUNITY NAME	INITIAL IDENTIFICATION	FLOOD HAZARD BOUNDARY MAP REVISIONS DATE(S)	FIRM EFFECTIVE DATE	FIRM REVISIONS DATE(S)
Pico Rivera, City of	July 6, 1998	None	July 6, 1998	None
Pomona, City of[2]	None	None	None	None
Rancho Palos Verdes, City of[2]	None	None	None	None
Redondo Beach, City of	June 28, 1974	May 21, 1976	September 15, 1983	None
Rolling Hills Estates, City of[1,2]	None	None	None	None
Rolling Hills, City of[2]	None	None	None	None
Rosemead, City of[2]	None	None	None	None
San Dimas, City of	June 28, 1974	None	April 1, 1977	June 2, 1978
San Fernando, City of[1,2]	None	None	None	None
San Gabriel, City of[1,2]	None	None	None	None
San Marino, City of[1,2]	None	None	None	None
Santa Clarita, City of	October 24, 1978	None	September 29, 1989	None
Santa Fe Springs, City of	June 28, 1974	October 3, 1975	April 15, 1980	None
Santa Monica, City of[2]	None	None	None	None

[1]Non-floodprone Community
[2]This community did not have a FIRM prior to the first countywide FIRM for Los Angeles County

TABLE 16

FEDERAL EMERGENCY MANAGEMENT AGENCY

LOS ANGELES, CA
AND INCORPORATED AREAS

COMMUNITY MAP HISTORY

COMMUNITY NAME	INITIAL IDENTIFICATION	FLOOD HAZARD BOUNDARY MAP REVISIONS DATE(S)	FIRM EFFECTIVE DATE	FIRM REVISIONS DATE(S)
Sierra Madre, City of[2]	None	None	None	None
Signal Hill, City of[1,2]	None	None	None	None
South El Monte, City of[1,2]	None	None	None	None
South Gate, City of	July 6, 1998	None	July 6, 1998	None
South Pasadena, City of[1,2]	None	None	None	None
Temple City , City of[2]	None	None	None	None
Torrance, City of	August 2, 1974	December 5, 1975	December 18, 1979	None
Vernon, City of[1,2]	None	None	None	None
Walnut, City of[2]	None	None	None	None
West Covina, City of	December 2, 2004	None	December 2, 2004	None
West Hollywood, City of	June 18, 1987	None	June 18, 1987	None
Westlake Village, City of[2]	None	None	None	None

[1]Non-floodprone Community
[2]This community did not have a FIRM prior to the first countywide FIRM for Los Angeles County

TABLE 16

FEDERAL EMERGENCY MANAGEMENT AGENCY

LOS ANGELES, CA
AND INCORPORATED AREAS

COMMUNITY MAP HISTORY

COMMUNITY NAME	INITIAL IDENTIFICATION	FLOOD HAZARD BOUNDARY MAP REVISIONS DATE(S)	FIRM EFFECTIVE DATE	FIRM REVISIONS DATE(S)
Whittier, City of	June 28, 1974	December 12, 1975	January 16, 1981	None
Los Angeles County (Unincorporated Areas)	July 7, 1970	October 24, 1978	December 2, 1980	November 15, 1985 July 6, 1998 March 30, 1998

[1]Non-floodprone Community

[2]This community did not have a FIRM prior to the first countywide FIRM for Los Angeles County

TABLE 16

FEDERAL EMERGENCY MANAGEMENT AGENCY

LOS ANGELES, CA
AND INCORPORATED AREAS

COMMUNITY MAP HISTORY

7.0 OTHER STUDIES

Los Angeles County

A Flood Hazard Boundary Map for Los Angeles County was published in 1978. In most cases, Special Flood Hazard Areas shown on the Flood Hazard Boundary Map are either located in flood control facilities, are included as Special Flood Hazard Areas on the maps, or were eliminated as a result of this study. Differences in flooding limits can be attributed to the more detailed methods of analysis used in this study. In some instances, Special Flood Hazard Areas shown on the Flood Hazard Boundary Map were found to be adequate to portray approximate flooding limits. In the Malibu area, approximate boundaries have been extended in a few cases. This study supersedes the Flood Hazard Boundary Map for Los Angeles County.

Drainage deficiencies and historical flooding information, on file at the Los Angeles County Flood Control District, were reviewed in the course of the study.

The Flood Insurance Study for Ventura County, California, is in agreement with this study.

This study is in general agreement with the Flood Insurance Studies for San Bernardino County, California, and Orange County, California, with the exception of small approximate areas. These areas were determined to be areas of low development potential and, therefore, were considered insignificant.

City of Agoura Hills

This study was prepared from data used in the preparation of the Flood Insurance Study for Los Angeles County, California, published in December 1980 (Federal Emergency Management Agency, 1980). Currently, areas of Los Angeles County are being revised by FEMA and this study is in agreement with those revisions.

City of Avalon

A Flood Hazard Boundary Map for the City of Avalon was published in 1976. This study supersedes the Flood Hazard Boundary Map.

This study supersedes the 1978 Flood Insurance Study for Avalon.

In 1973, a U.S. Geological Survey Map of Flood-Prone Areas for Santa Catalina Island East was compiled. The flooding shown on that map is approximate and is superseded by this study.

This study is authoritative for the purposes of the NFIP; data presented herein either supersede or are compatible with all previous determinations.

Cities of Bellflower, Carson, Compton, Downey, Gardena, Lakewood, Long Beach, Lynwood, Paramount, Pico Rivera, South Gate

The USACE developed overflow maps for this study area during their Los Angeles County Drainage Area study. Their maps indicate a large floodplain associated with the Los Angeles River and Rio Hondo of that time period. Both flood control channels have been significantly upgraded since the time of study, and the floodplain maps contained herein supersede that study.

City of Burbank

The Los Angeles District of the USACE prepared a Flood Insurance Study for Burbank. Due to the use of completely different criteria, discharges arrived at in this Flood Insurance Study for flooding of the 1-percent annual chance flood event are significantly greater than those in the USACE study. In addition, Flood Insurance Studies for the unincorporated areas of Los Angeles County and the incorporated City of Los Angeles have been completed. These studies will be in complete agreement with this Flood Insurance Study. A Flood Hazard Boundary Map for the City of Burbank was published by the FIA on September 26, 1975. Flooding shown on this map conforms to flooding delineated in this study. Minor differences can be attributed to the more detailed methods of analysis used in this study.

City of Culver City

A Flood Hazard Boundary Map for Culver City was published by the FIA on September 3, 1976. Flooding shown on the Flood Hazard Boundary Map conforms to flooding delineated in this study. Minor differences can be attributed to the more detailed methods of analysis used in this study.

The USACE, Los Angeles District, has undertaken an analysis of the Ballona Creek Channel watershed. Their file data includes (1) discharge-frequency curves for the stream gage at Sawtelle Boulevard; (2) channel and bridge capacities; and (3) the magnitude of the 1-percent annual chance frequency flood for various locations along Ballona Creek Channel. The discharge-frequency curves for Ballona Creek Channel were used to evaluate Ballona Creek Channel. The Los Angeles County Flood Control District's findings concur with the USACE' results that Ballona Creek Channel has adequate capacity to convey the 1-percent annual chance frequency discharge.

City of La Mirada

A Flood Hazard Boundary Map for the City of La Mirada was published by the FIA on December 10, 1976. Flooding shown on the Flood Hazard Boundary Map conforms to flooding delineated in this study. Minor differences between the flooding shown on the previous map and the results of this study can be attributed to the more detailed methods of analysis used for this study.

Flood Insurance Studies were prepared for the contiguous Cities of Buena Park, Fullerton, La Habra, and Santa Fe Springs as well as for the unincorporated areas of Orange County, California. These studies are in general agreement with this study.

Drainage deficiencies and historical flooding information are on file at the Los Angeles County Flood Control District, and were reviewed in the course of the study.

Cities of Lancaster and Palmdale

A Flood Hazard Boundary Map for Palmdale was published by the FIA on' December 24, 1976. Flooding shown on the Flood Hazard Boundary Map conforms to flooding delineated in this study. Differences can be attributed to the more detailed topographic data and extensive field reviews used in this study. Therefore, the Flood Hazard Boundary Map for Lancaster and Palmdale is superseded by this Flood Insurance Study.

The USACE, Los Angeles District, has investigated the Antelope Valley watersheds. Their report includes discharge-frequency curves for the stream gages on Little Rock and Big Rock Washes and the magnitude of the 1-percent annual chance frequency flood for various locations throughout Antelope Valley. The discharge-frequency curves for Antelope Valley were used to evaluate the flood hazards in Palmdale. The report is in general agreement with this Flood Insurance Study.

City of Los Angeles

A Flood Hazard Boundary Map for the City of Los Angeles was published on December 13, 1977. The Special Flood Hazard Areas shown on the Flood Hazard Boundary Map are located in flood-control facilities, are included as Special Flood Hazard Areas, or were eliminated as a result of this study. Minor differences in flooding limits can be attributed to the more detailed methods of analysis used in this study. Therefore, this study supersedes the Flood Hazard Boundary Map. This study also supersedes two unpublished reports by the USACE dated May 1971 and June 1971.

The USACE developed overflow maps for this study area during their Los Angeles County Drainage Area study. Their maps indicate a large floodplain associated with the Los Angeles River and Rio Hondo of that time period. Both flood control channels have been significantly upgraded since the time of study, and the floodplain maps contained herein supersede that study.

City of Montebello

A Flood Hazard Boundary Map for the City of Montebello was published by the FIA on December 19, 1975. Flooding shown on the Flood Hazard Boundary Map conforms to flooding delineated in the original study. Minor differences between the flooding shown on the Flood Hazard Boundary Map and the results of the original study can be attributed to the more detailed methods used in the original study.

The USACE developed overflow maps for this study area during their Los Angeles County Drainage Area study. Their maps indicate a large floodplain associated with the Los Angeles River and Rio Hondo of that time period. Both flood control channels have been significantly upgraded since the time of study, and the floodplain maps contained herein supersede that study.

City of Redondo Beach

This study supersedes the existing Flood Hazard Boundary Map for the City of Redondo Beach, California.

City of Santa Fe Springs

A Flood Hazard Boundary Map for the City of Santa Fe Springs was published by the FIA on June 28, 1974. The special flood hazard areas shown on that map are either located in the flood control facilities or are identified on the map. Minor differences in flooding limits can be attributed to the more detailed methods of analysis used in this study.

The Los Angeles County Flood Control District has, on file, information relating to drainage deficiencies and historical flooding in Santa Fe Springs. This information was used in preparation of the present study and is, therefore, in agreement.

The Flood Insurance Studies for all communities bordering Santa Fe Springs were reviewed to ensure that this study is consistent with all other applicable studies.

City of Torrance

A Flood hazard Boundary Map for the City of Torrance was published by the FIA on December 5, 1975. Flooding shown on the Flood Hazard Boundary Map conforms to flooding delineated in this study. Minor differences can be attributed to the more detailed methods used in the current analysis.

Drainage deficiencies and historical flooding information on file at the Los Angeles County Flood Control District were reviewed during the course of the study.

City of West Hollywood

Since this Flood Insurance Study was prepared directly from the technical data presented in the Los Angeles County Flood Insurance Study and the Flood Insurance Study for the City of Los Angeles, all flood boundaries match.

City of Whittier

The FIA has previously, published a Flood Hazard Boundary Map for Whittier. However, the present study represents a more detailed analysis.

Flood Insurance Studies have been published for the adjacent Cities of La Habra and Santa Fe Springs. In southwest Whittier, at the corporate limits of Santa Fe Springs, 1-percent annual chance shallow flooding does not exceed the crown, or centerline, of Mulberry Drive. The results of this study are in agreement with the Flood Insurance Studies prepared for these communities.

Toups Corporation supplied hydrologic data and 1-percent annual chance flood boundaries for La Mirada Creek. This information was used in the analysis of La Mirada Creek as it passes through Whittier. The study contractor's findings of flooding of La Mirada Creek are in agreement with information furnished by Toups Corporation.

Information pertaining to revised and unrevised flood hazards for each jurisdiction within Los Angeles County has been compiled into this FIS report. Therefore, this FIS report supersedes or is compatible with all previous studies published on streams studied in this report and should be considered authoritative for the purposes of the NFIP.

8.0 LOCATION OF DATA

Information concerning the pertinent data used in the preparation of this study can be obtained by contacting Federal Emergency Management Agency, Region IX, 1111 Broadway, Suite 1200, Oakland, California 9460-4052.

9.0 BIBLIOGRAPHY AND REFERENCES

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10.0 REVISION DESCRIPTIONS

This section has been added to provide information regarding significant revisions made since the original FIS was printed. Future revisions may be made that do not result in the republishing of the FIS report. To assure that the user is aware of all revisions, it is advisable to contact the appropriate community repository of flood-hazard data listed on Sheet 3 of the Map Index for the FIRMs.

10.1 Revision (Revised January 6, 2016)

The purpose of the January 6, 2016, revision is to incorporate new detailed hydraulic models completed by HDR Engineering in 2008, for the following streams in the City of Palos Verdes Estates – Unnamed Stream Main Reach, Unnamed Stream Tributary 1, and Unnamed Stream Tributary 2. The new mapping revises effective panel 1920, which is now split into 4 panels (1916, 1917, 1918, 1919). This revision also includes new detailed analysis and floodplain mapping for Las Virgenes Creek in the City of Calabasas. Las Virgenes Creek falls on Panels 1264, 1526, & 1527.