EXISTING CONDITIONS REPORT

RESTUDY OF
ALBUQUERQUE MASTER DRAINAGE STUDY
VOLUME II

K-1 81 D2

IV

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MARCH, 1987

PREPARED FOR

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ALBUQUERQUE, NEW MEXICO

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INTRODUCTION

This report contains the Phase I (Existing Conditions) results of a revision and update of the Albuquerque Master Drainage Study Volume II. It was prepared for the City of Albuquerque under the authority of the Architect/Engineer's Agreement No. 86-24.

The Albuquerque Master Drainage Study (AMDS), Volume II was completed in January, 1981. Since its release, questions have arisen about the appropriateness of the modeling approach used. The technique most criticized is the use of analysis "lines" rather than analysis "points" to locate and characterize flows. Flows of interest were assumed to cross somewhere between the endpoints of the lines, a length of anywhere from a few hundred feet to a mile. This approach leaves in question the flow depth and other flow characteristics at any given point of interest.

Since January of 1981, several major storm drainage improvement projects have been completed. In order to update the 1981 study with these improvements and, at the same time, reanalyze flows using a "point" approach rather than "lines", the City of Albuquerque authorized Bohannan-Huston Inc. to prepare a restudy of AMDS. This revised AMDS Volume II is slated for completion in 1987. It consists of three phases.

The purposes of Phase I of the study were to:

1. Update the Inventory of Existing Storm Drainage Systems.

2. Prepare an updated rainfall-runoff computer model using existing conditions.

3. Determine the limits of the 10 and 100-year storm flooding and show them on 1:500 mapping.

Phases II (Draft Developed Conditions Report) and III (Final Developed Conditions Report) will build upon the Phase I findings to:

1. Propose drainage facilities to eliminate flooding.

2. Revise the computer model to reflect ultimate development conditions and the proposed drainage facilities.

3. Estimate costs to construct the proposed facilities.

4. Prepare an analysis to evaluate the extra cost of storm drain facilities required to meet the Development Process Manual criteria calling for one dry lane during the 10-year storm.

SCOPE

In order to accomplish the Phase I purposes listed above, the scope of the study encompassed the following:

- Collect data needed to prepare mapping and to analyze the existing storm drainage systems.

- Determine the hydrological parameters for the Volume II area to use in the modeling systems simulating the design storm and the resulting runoff patterns.

- Identify and study appropriate drainage basins to determine the basins' ability to capture surface runoff and detain it or transport it safely away from developed areas.

- Prepare 1:500 mapping on which the flood limits will be shown.

DESCRIPTION OF STUDY AREA

The AMAFCA North and South Diversion channels isolate Albuquerque's "Heights" from the Rio Grande Valley during all but extremely rare runoff events. Hydrology in the Heights has been, and continues to be, greatly influenced by the mountains.

The Heights area south of Interstate 40 and north of Tijeras Arroyo is commonly known as the Southeast Heights. The area lies on an alluvial fan formed by the Tijeras Arroyo. The study area, shown on the Existing Storm Drainage Facilities and Site Map located in the back pocket of this report, is bounded by I-25 to the west, City limits to the east, I-40 to the north, and the Tijeras Arroyo to the south.

The old Campus Draw, now a series of paved, residential streets, runs from the northern edge of Kirtland Air Force Base at Louisiana Boulevard to about Lomas Boulevard and University Boulevard (see Facilities and Site Map), and is the dominant physical and hydrologic feature in the area and the location of the most severe drainage and flooding problems in the Southeast Heights. Numerous smaller and less conspicuous arroyos cross the area north of the Campus Draw and influence both the topography and the drainage in their immediate vicinities. The developed section of the Southeast Heights south of Coal Avenue to Gibson Boulevard and the undeveloped South Industrial Area south of Gibson Boulevard are crossed by numerous small but well defined arroyos which generally govern the drainage patterns in those areas. The entire area slopes from east to west at 1% to 3%.

The soils are gravely fine sandy loams and loamy sands. The soils all exhibit fair to good permeability and are erosive. Prior to development, the uplands supported only native grasses and sage brush due to the scarcity of dependable rainfall during the growing season and the inability of the soil to retain moisture.

Development of the Heights began in the Southeast Heights around the University of New Mexico in the early part of this century. That area is characterized by large mature trees and stately older homes. The area just north of Kirtland Air Force Base and Albuquerque International Airport developed in response to the expansion of the U.S. Army contingent during World War II. The late 1950's and early 1960's saw the beginning of the push eastward and northward into what is now the Northeast Heights.

HISTORY OF FLOODING

Precipitation in the Albuquerque area is generally of two distinct types: winter frontal storms and summer convective thunderstorms. The winter precipitation may be in the form of rapidly melting snow accumulations of less than six inches or long duration light rainfall with an occasional squall line. The summer thunderstorm activities involve high intensity, short duration bursts of up to 4 inches in total rainfall volume but covering areas of two square miles or less. Of the two precipitation types, the summer thunderstorms cause the highest peak discharges and govern for design considerations. Severe flooding in the Albuquerque area is usually the result of the summer convective storms.

The noted historical floods in this area occurred in 1961, 1963, and 1980. The U.S. Soil Conservation Service conducted flood profile studies prior to 1974, and the U.S. Army Corps of Engineers has since done a more thorough investigation. Table 1 contains a partial list of some pertinent historical flood events with their locations and associated damage. The information on storm magnitude, location of flooding, and flood water depth associated with each location were obtained from newspaper files. The estimated figures of flood damage were from the files of the Albuquerque District Office of the Corps of Engineers. The damage figures reflect the dollar values in the year of the flood occurrence.

AGENCIES/JURISDICTION

Drainage regulation in Albuquerque is the responsibility of several governmental organizations with overlapping jurisdictions. The City of Albuquerque and the Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA), have the primary responsibilities for drainage within the city limits of Albuquerque. The City has jurisdiction everywhere within the municipal boundaries. The Middle Rio Grande Conservancy District (MRGCD) has significant control in valley areas where MRGCD drains can provide the only viable outfall for storm waters. The U.S. Army Corps of Engineers was responsible for the design and construction of the North and South Diversion Channels. Their Albuquerque District Engineer has the responsibility and authority to insure that the channels continue to provide their full design protection to the community. AMAFCA has, through a contractual

TABLE 1

Selected Historical Flood Events In Albuquerque

Date and Estimated Damage	Maximum Rainfall Depth (in)	Gage Location	Flooding Location	Approx. Max Depth (ft.)
August 11, 1961 $200,000	4.0	1206 Field N.E.	2nd and Candelaria	2.0
			2nd to the RR, Griegos to Hilton Ave.	1.5-2.5
			Commercial and McKnight	2.0
August 10, 1963 $1,875,000	3.3 inches in less than one hour.	Northeast Heights	San Mateo and Comanche	4.0
			Central and Hermosa	3.0
			Campus Blvd.	1.0
			Winrock Shopping Center	1.0
			Broadway and Mountain	3.0
			2nd St., Menaul to Candelaria	2.0-3.0
			Edith and Commercial	3.0
			Central at RR underpass	4.0
June 17, 1967 $442,000	1.4	3500 Campus NE	Central at RR underpass	4.0
			Both sides of RR, Lomas to Mountain	1.5-2.5
			Broadway and Lomas	1.5
			Menaul and University	1.0
			Commercial to RR, Menaul to Candelaria	1.5
September 7, 1969 $328,000	1.4	1417 Lead SE	Williams to RR, Woodward to Trumbull	1.0-3.0
			Williams and Abeline	3.0
			2nd and Bridge	1.5
			Central and RR underpass	4.0
			Lomas and Broadway	2.0
			Lomas and Edith	2.0
			Candelaria and 2nd	2.0
July 15, 1970 $200,000	1.3 (unofficial)	500 Juan Tabo SE	Williams to RR, Anderson to San Jose	1.0-3.0
			Williams and Commercial	2.5
			Central at RR underpass	4.0
			Silver and Broadway	1.5
			Broadway and Grand	2.0
			Broadway, Mountain to Marquette	1.5
			Lomas, 1st to Edith	1.5
			Lomas at RR	1.0
			Lomas and 4th	1.5
			Louisiana and Constitution	1.5
July 31, 1974 $123,000	1.2	Coors and Barcelona	Arenal and Coors	4.0
July 26, 1977 $20,000	1.7	1800 Carol NE	500 Hillview NE	2.5
			Hopewood and Freeway NE	2.5
August 14, 1980 $800,000	4.4 in 4 hours	Cherry Hills	General	1.0-3.0

agreement with the Corps of Engineers, the responsibility and obligation to provide for the continual operation and maintenance of the North and South Diversion Channels and their appurtenant inlet and outlet structures.

The Federal Emergency Management Agency, FEMA, has produced Flood Insurance Rate Maps, FIRM, and Floodway Maps of the entire area. The FIRM maps, which are currently enforced, set flood insurance rates for buildings that are within the 100-year limits of flooding. These limits of flooding are based on the AMDS Study completed in January, 1981.

SOURCES OF INFORMATION

Surface drainage patterns were determined from contours on City of Albuquerque topographic orthophoto mapping. Information on existing storm drainage facilities was obtained from various sources including:

- o City of Albuquerque Drainage Facilities Maps.
- o Albuquerque Master Drainage Study, January, 1981.
- o Design Analysis and Drawings for the San Mateo Storm Drainage System, Highland Detention Basin & System, Fairgrounds Detention Basin & System and Southeast Heights Storm System.
- o Field Reconnaissance by Bohannan-Huston Inc. personnel.

HYDROLOGY

Precipitation. In the project contract, Bohannan-Huston Inc. is directed to determine a representative rainfall hydrograph for the Albuquerque area. Since the City of Albuquerque has obtained the

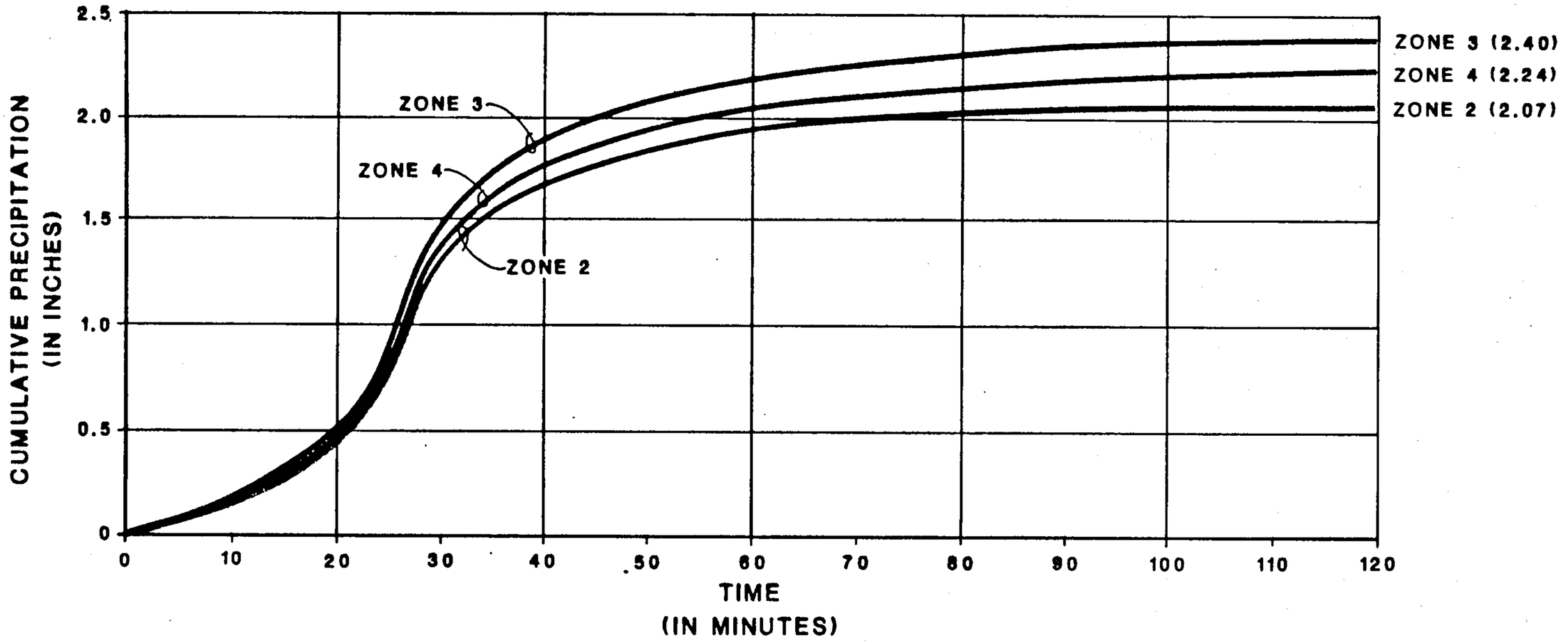
services of Dr. Richard Heggen of the University of New Mexico Civil Engineering Department to update the City Development Process Manual (DPM) section on hydrology; it was decided (with City approval) to use Dr. Heggen's recommended rainfall hydrograph for this study. A description follows.

Dr. Heggen divided the Albuquerque area into four rainfall zones:

Zone 1. West of the Rio Grande.
Zone 2. East of the Rio Grande below Elevation 5200'.
Zone 3. North of Central Avenue above Elevation 5200'.
Zone 4. South of Central Avenue above Elevation 5200'.

From the National Oceanic and Atmospheric Administration (NOAA) Atlas, Volume IV, Heggen obtained 6-hour rainfall depths for the 5, 10, 15, and 30 minute storms and the 1, 2, 3 and 6 hour storms for each zone. Depth-duration curves were plotted and the five-minute intensities calculated. The intensities were rearranged to produce a smooth ascending and descending hydrograph with the peak at 30 minutes. The new depth-duration curve was then calculated from the rearranged intensities. Dr. Heggen's 100-year, six-hour storms for Zones 2, 3 and 4 are shown on Figure 1. (The study area includes no Zone 1 land).

Bohannan-Huston, Inc. used the same procedure as outlined in the preceding paragraph to generate the 10-year, six-hour storms, again peaking the hydrograph at 30 minutes. The Zones 2, 3 and 4 storms are shown on Figure 2. The rainfall distribution shown on Figures 1 and 2 were used for the runoff model.



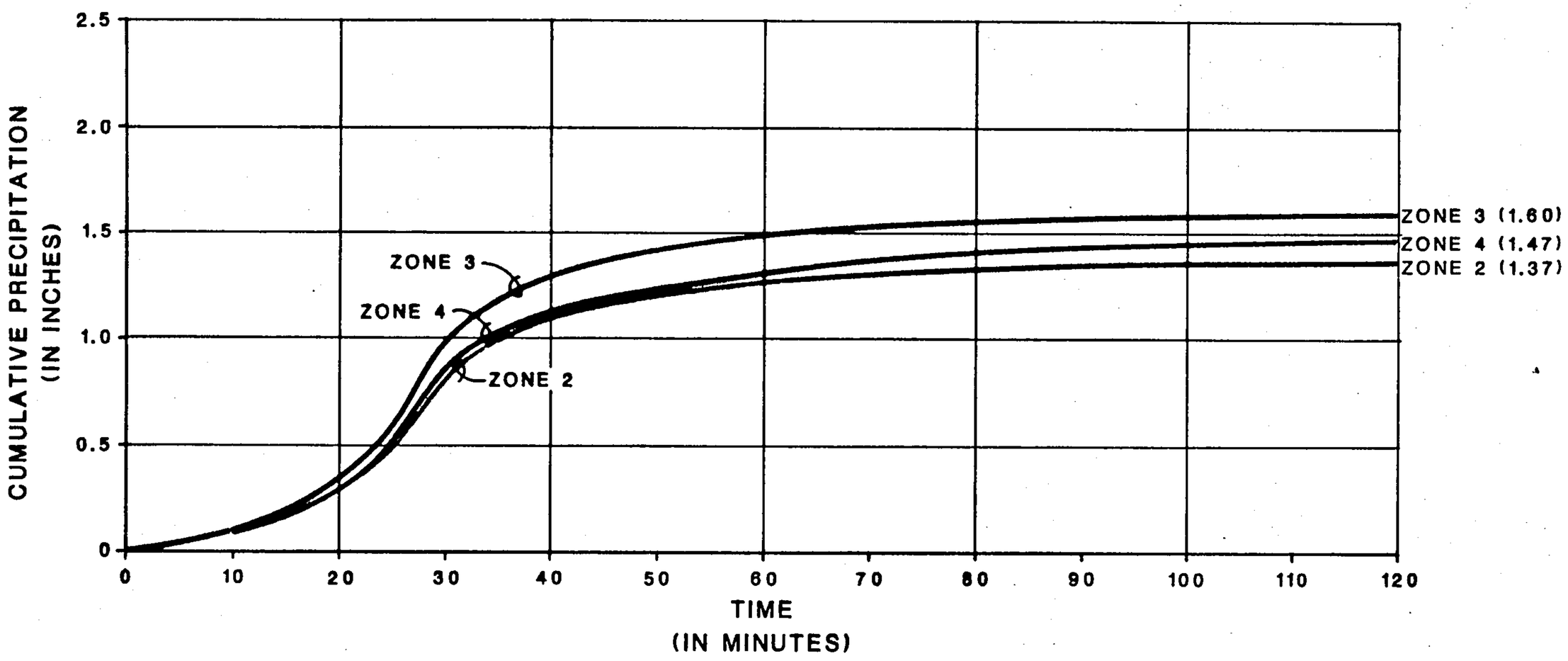
100-YEAR, 6-HOUR RAINFALL ALBUQUERQUE AREA

ZONE 2 EAST OF RIO GRANDE, BELOW 5200' ELEV.

ZONE 3 NORTH OF CENTRAL AVE., ABOVE 5200' ELEV.

ZONE 4 SOUTH OF CENTRAL AVE., ABOVE 5200' ELEV.

FIGURE 1



10-YEAR, 6-HOUR RAINFALL ALBUQUERQUE AREA

ZONE 2 EAST OF RIO GRANDE, BELOW 5200' ELEV.

ZONE 3 NORTH OF CENTRAL AVE., ABOVE 5200' ELEV.

ZONE 4 SOUTH OF CENTRAL AVE., ABOVE 5200' ELEV.

FIGURE 2

Rainfall-Runoff Model. HYMO (Hydrologic Modeling), a computer program developed by the Agricultural Research Service of the U. S. Department of Agriculture, was used to model the drainage systems of the study area.

A unit hydrograph approach is used by HYMO for generating runoff hydrographs for each subbasin. Three equations (found in the HYMO Users' Manual) define the rising limb and the recession of the hydrograph. The shape of the unit hydrograph is defined by two variables, the time to peak, T_p, and the recession constant, K. The HYMO program will compute the values of T_p and K or allow the user to specify them. The following equations were used to define these variables:

$$T_c = \frac{.0078 \times L^{.77}}{S^{.385}} \qquad \text{(Kirpich Equation)}$$

T_c = Time of concentration in minutes
L^c = Length of longest water pathway in feet
S = Slope of longest water pathway

Time to Peak $T_p = 2/3\ T_c$ in minutes

Recession Constant $K = 1/2\ T_p$ in minutes

Next, the depth of runoff in inches is found by applying the Soil Conservation Service (SCS) rainfall-runoff relationship:

$$Q = (P-(200/CN)+2)^2/(P+(800/CN)-8)$$

Where: Q = runoff in inches depth over the drainage area.

P = accumulated rainfall in inches.

CN = curve number.

Storm outflow hydrographs are then computed by merging the accumulated depth of runoff with the unit hydrograph.

Subbasin Flow Considerations. A subbasin can be defined as an area whose drainage routes all have a common collection point which is to be evaluated. Most of the subbasins in the study area have both underground storm sewer systems and overland flow conveying rainfall from the area.

In this study, the subbasins were modeled around the surface flow rather than the underground storm drain system. Flow carried by the storm sewers is separated from surface flows using the Divide Hydrograph Command. The amount separated out for storm sewer flow is the capacity of the drain pipe immediately downstream of the storm sewer analysis point.

In most cases, the overland and storm drain collection analysis points are at the same location, for example, a street intersection. Occasionally, storm sewer systems leave a subbasin upstream of the overland collection/analysis point.

In these cases, flow to the storm sewer was apportioned based on area. If the apportioned flow exceeded sewer capacity, the maximum capacity of the storm sewer was used. When the apportioned flow was less than the storm sewer capacity, the apportioned flow was used.

Routing. HYMO routes water through open channels and streams using the variable storage coefficient (VSC) flood routing method. Reservoir routing applies the storage-indication method of the Soil Conservation Service. A storm water pipe routing algorithm was added by Bohannan-Huston, Inc. using the VSC method.

Curve Numbers. The SCS method requires the determination of the curve number for the analysis area. Runoff curve numbers represent the combined effect of soil type, vegetative cover, land use and antecedent moisture condition.

In this analysis, pervious and impervious areas within a subbasin were considered separately. Under City of Albuquerque direction, impervious areas were further divided into two categories:

1. Direct runoff impervious areas such as streets, sidewalks, drive pads and pitched roofs.

2. Detained runoff impervious areas such as flat roofs and parking areas acting as intentional or unintentional detention areas.

The direct runoff impervious areas were assigned a curve number of 95 and their runoff calculated separately from the pervious areas. Runoff for the combined pervious and detained impervious areas was then generated using a composite curve number. This technique reduces the impact of the detained impervious areas and, in effect, simulates detention. The composite curve number was calculated as follows:

$$\text{Composite CN} = \frac{\text{Perv. CN} \times \%\ \text{Perv. Area} + \text{Imperv. CN} \times \%\ \text{Detained Area}}{100}$$

An impervious CN of 95 was used for the detained impervious areas.

Percentages of impervious areas were determined through windshield surveys and aerial photographs. In general, residential areas were found to be 45% impervious with 18% directly contributing runoff and 27% detained runoff. Fifty-five percent was found to be pervious.

Soil types and their respective hydrologic soil groupings were obtained from the Bernalillo County Soil Survey. "B" soil groups predominated, followed by a significant portion of "A" soils and a small area of "D" soils.

Two categories of land use were designated for the pervious areas. "Pervious Urban Areas" are defined as those lands used for lawns, landscaping, gardens and irrigated parks. All other lands were considered to be "Range in Fair to Poor Condition".

Free discharge from commercial and residential developments is assumed for the AMDS Volume II Restudy.

Antecedent Moisture Condition II, used by the SCS for the entire State of New Mexico, was assumed for this study, also.

Following the above listed guidelines, the curve numbers used in this study are:

Land Use	Soil Group A	Soil Group B	Soil Group C	Soil Group D
Pervious Urban Areas	44	55	65	72
Range-Fair to Poor Condition	55	63	72	80

Composite CN's ranged from 62 to 74, with most areas having composite curve numbers of 68 to 72.

Modeling Map. The modeling map for the study area can be found in the pocket at the end of this report. Subbasin boundaries, existing drainage facilities and analysis points are shown.

HYDRAULIC STRUCTURES

Storm Sewers. The capacity of underground storm drain pipes is determined in the HYMO program using Manning's equation. Since almost all storm sewer pipes in the City of Albuquerque system are reinforced concrete, a roughness value "n" of 0.015 was used in the Manning's equation. In most cases, pipes were analyzed at full depth flow (not pressure flow). However, if isolated pipes within a system were found to restrict flow, the hydraulic slope was increased by adding two feet of head to the upstream end of the restrictive pipe. It is assumed that these isolated reaches will pressurize without discharging back into the streets. Storm sewers are assumed to be clean and in good repair.

The Existing Storm Drainage Facilities map for the study area can be found in the pocket at the end of this report.

Catch Basins. Throughout the study area, catch basins were assumed to be properly located and sized in conjunction with their associated trunk lines. If the trunk capacity is 80 cfs, it is assumed that the catch basins can capture 80 cfs. The exception to this assumption is Subbasin W1. While the 96" pipe leaving the subbasin has a capacity of 600 cfs, the catch basins in Subbasin W1 were found to have a capacity of only 300 cfs. The system was modeled to allow collection of only 300 cfs by this sewer line in Subbasin W1.

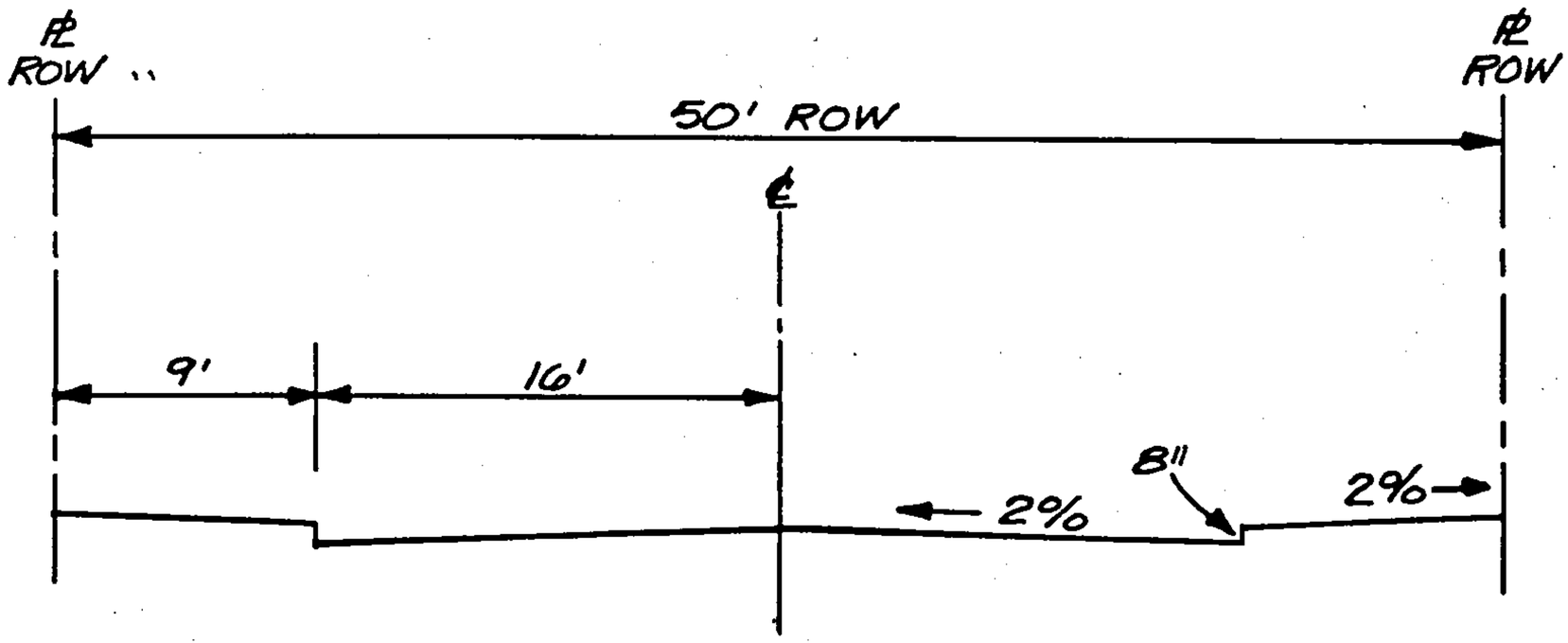
Street Flow. Seven typical Albuquerque street templates were used to model roadway flow. These are listed below and a schematic presented on Figures 3, 4, 5 & 6.

1. Normal local.
2. Major local.
3. Collector.
4. Minor arterial with elevated median.
5. Minor arterial with painted median.
6. Principal arterial with elevated median.
7. Principal arterial with painted median.

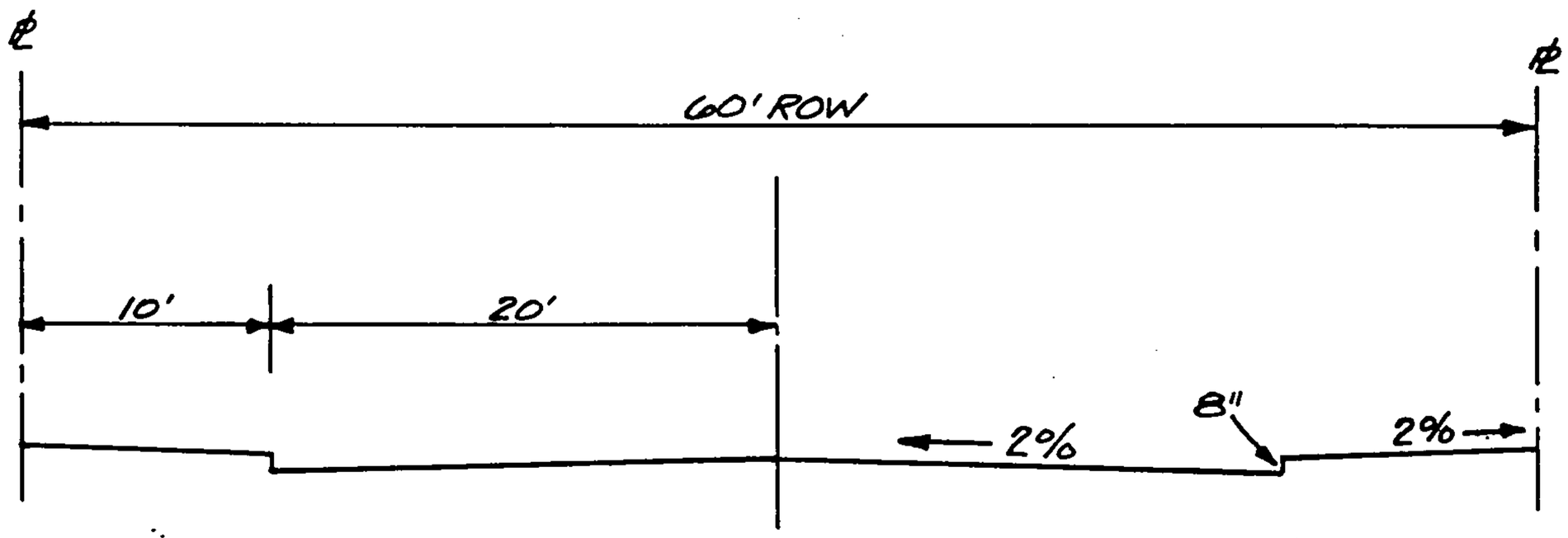
A roughness value of 0.017 was used to analyze street flow, as opposed to the value of 0.020 used in the original AMDS Volume II Study.

Street flooding for the 100-year flow is defined as any flow with a depth of more than 0.2 feet above the top of the curb. The 10-year street flooding is defined as any flow with a depth above the top of the curb.

Open Channels. Open channels are analyzed using Manning's equation. For hard-lined channels, flooding is indicated if out of bank flow occurs. For earthen channels, the flow boundaries are also the flood boundaries, whether the flow is within or outside the banks.

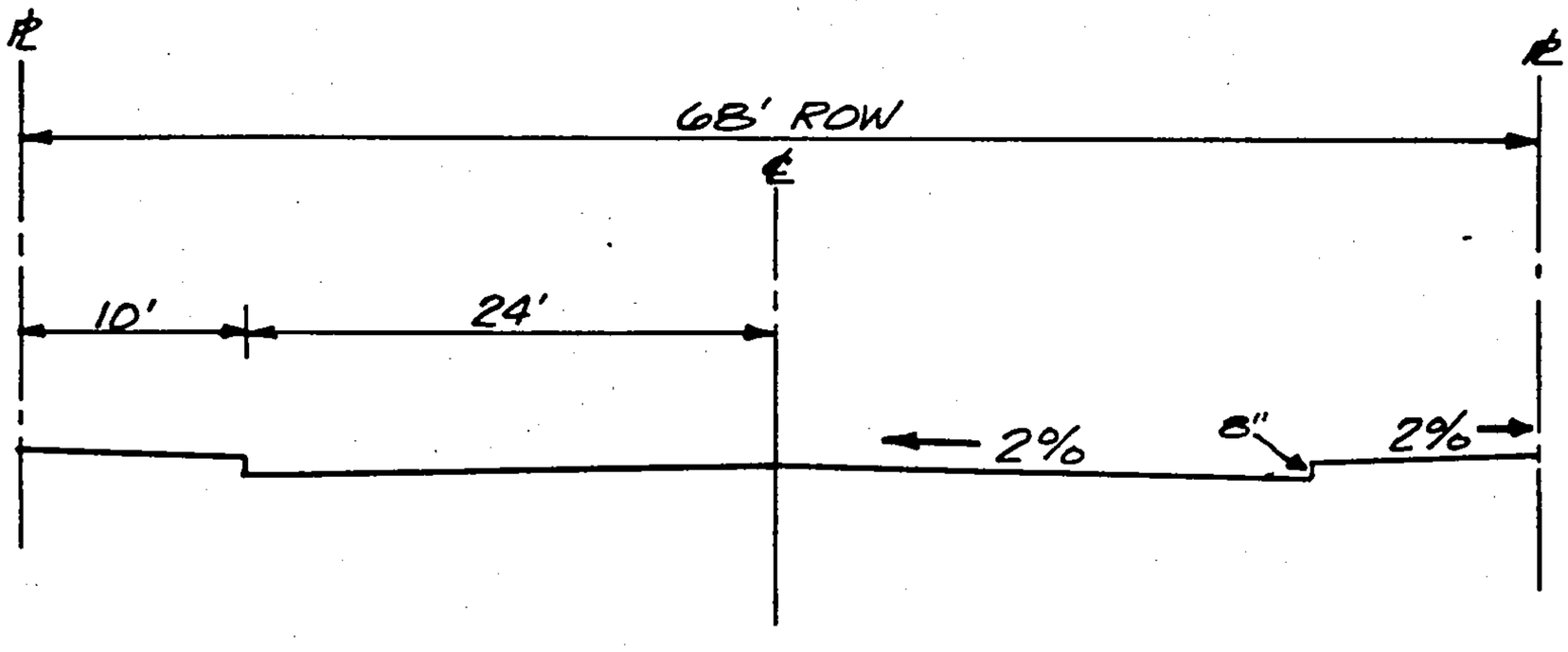


NORMAL LOCAL X-SECTION
(TYPICAL RESIDENTIAL)
32' WIDTH

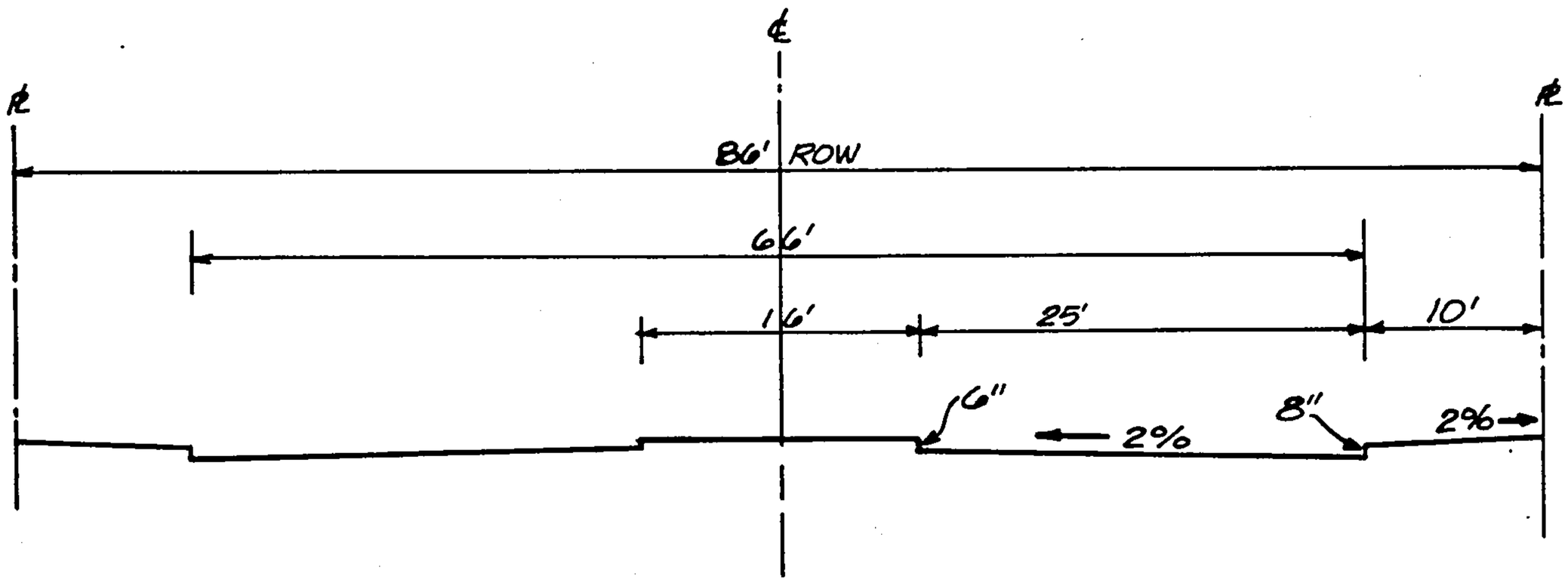


MAJOR LOCAL X-SECTION
40' WIDTH

FIGURE 3

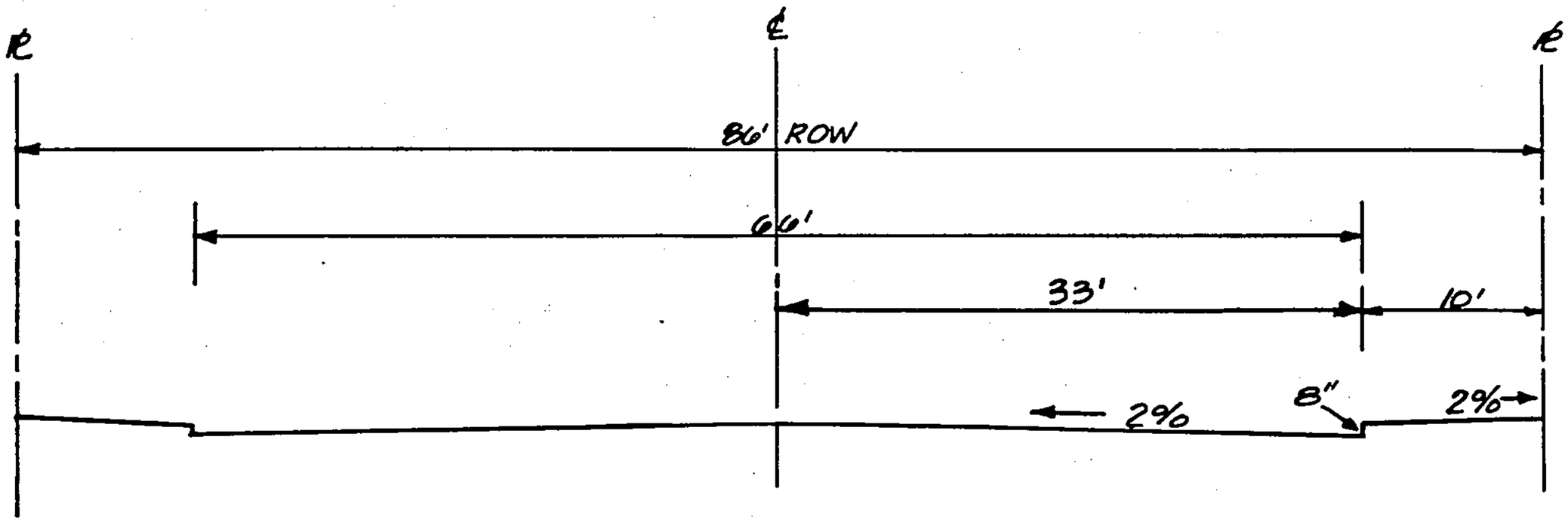


COLLECTOR X-SECTION
48' WIDTH

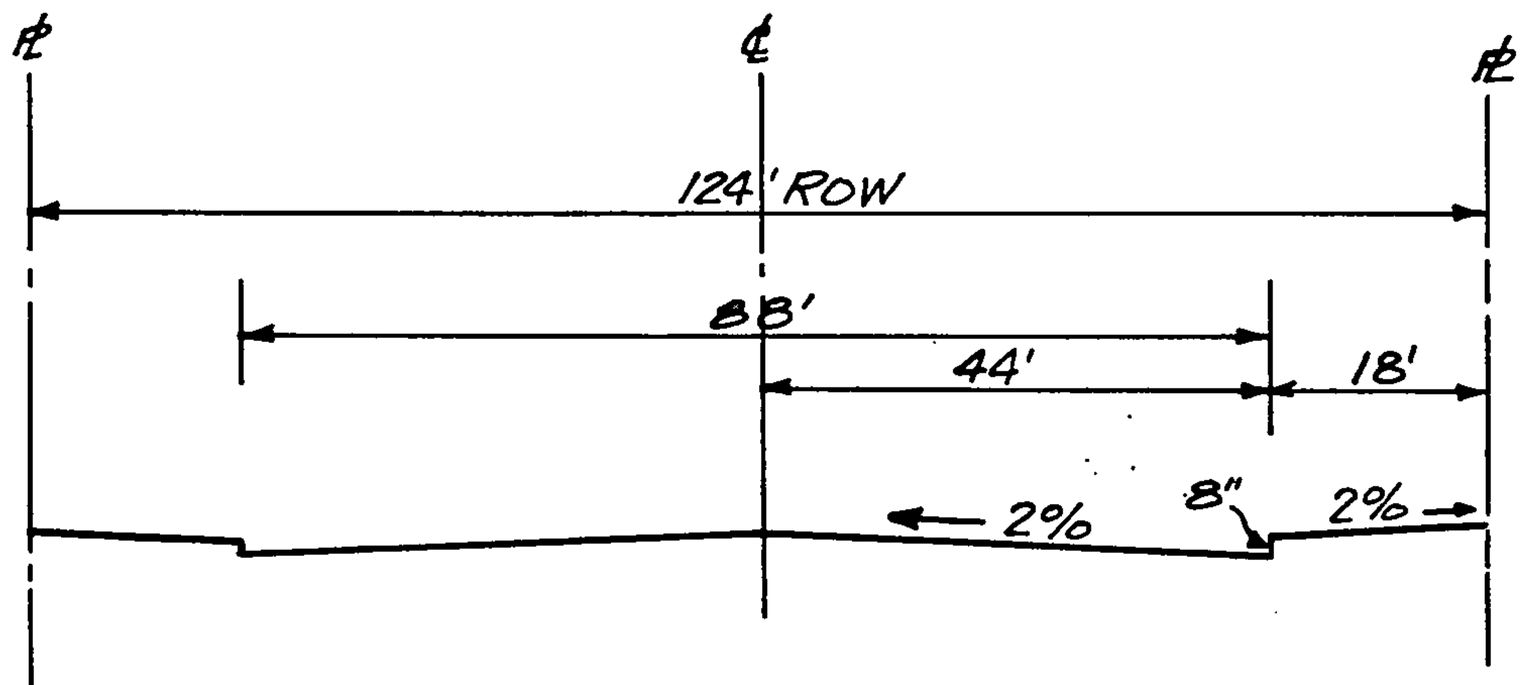


MINOR ARTERIAL X-SECTION
66' WIDTH, INCLUDING MEDIAN

FIGURE 4

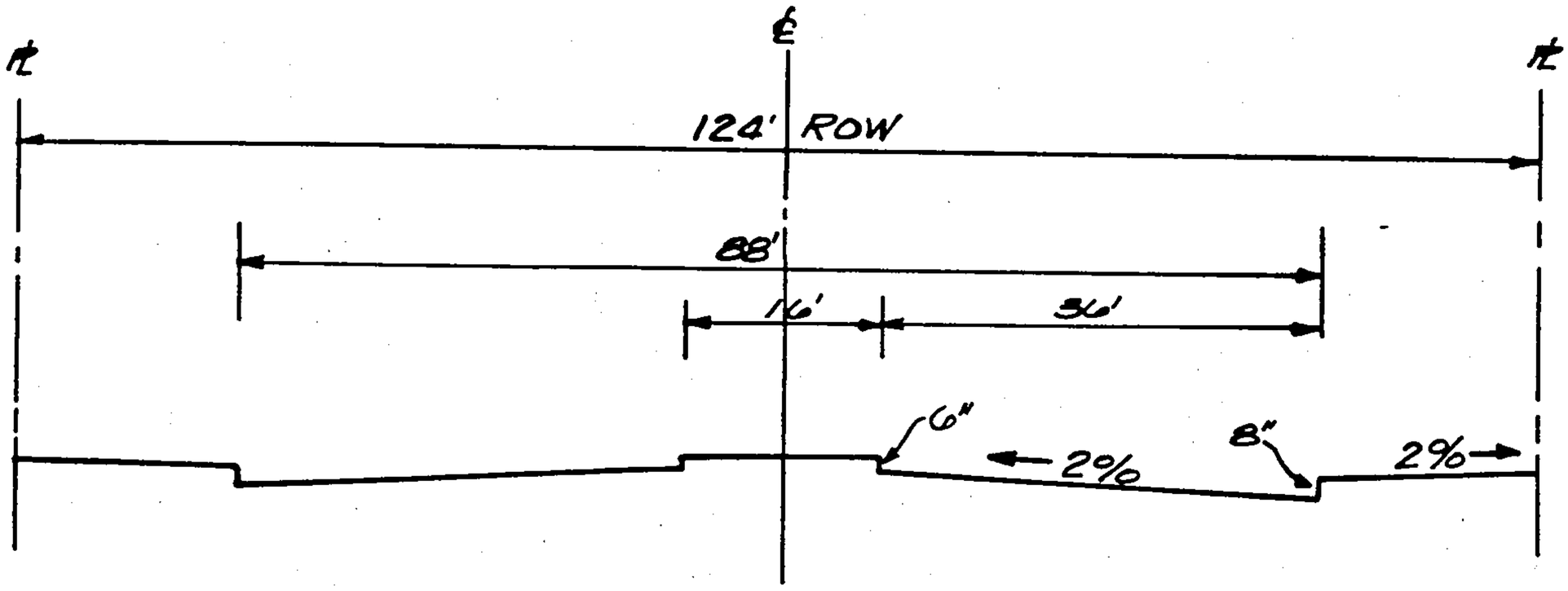


MINOR ARTERIAL X-SECTION
66' WIDTH, INCLUDING PAINTED MEDIAN



PRINCIPAL ARTERIAL X-SECTION
88' WIDTH, INCLUDING PAINTED MEDIAN

FIGURE 5



PRINCIPAL ARTERIAL X-SECTION
88' WIDTH, INCLUDING MEDIAN

FIGURE 6

Detention Reservoirs. Three existing detention reservoirs were modeled, the Highlands, Fairgrounds and Kirtland North Detention Reservoirs. The HYMO program includes a stage-storage-discharge routine for this analysis.

COMPARISON OF THE ORIGINAL AMDS VOLUME II CRITERIA AND THE RESTUDY CRITERIA.

Several criteria changes have been made in the Volume II Restudy from the original AMDS Volume II criteria. These changes will affect the floodplain boundaries to varying degrees. The differing criteria are presented below and discussed in the subsequent paragraphs.

ORIGINAL AMDS VOLUME II CRITERIA	RESTUDY AMDS VOLUME II CRITERIA
No increase in runoff rates due to development	Free Discharge allowed
Friction factor 'N'=0.02 for streets	'N'=0.017 for streets
100-yr storm flood above top of curb.	100-yr storm flood 0.2' above top of curb.

From the early 1960's to the early 1980's, the City of Albuquerque criteria allowed no increase in runoff due to development. Excess runoff had to be detained on site. Since enforcement of this policy was difficult and maintenance of detention facilities erratic, the City changed its position in the early 1980's and allowed free discharge. While the resulting increase in watershed discharge is felt primarily in the developing areas of the City, the mostly developed Southeast Heights area will also experience some increase in runoff from new development and from old retention facilities allowed to discharge freely.

The street friction factor of 0.017 was used in the Restudy in order to conform to the City's Development Process Manual. Where above-curb flow is expected, the roughness coefficient was adjusted accordingly. A decrease in roughness from 0.02 to 0.017 will increase the flow velocity and, thus, decrease the depth of flow in the street. For a typical 40'-wide street with a 2% slope running curb full, the roughness of 0.017 will produce a flow depth of approximately 0.1' less than the depth for a street roughness of 0.02.

The criteria change for the flow depth defining street flooding has obvious consequences. The Restudy criteria of .2' over the curb for the 100-year storm is less conservative than the original AMDS study. A larger flow would be required to produce flood boundaries under the Restudy criteria than under the old criteria.

For the 10-year storm, the flood depth criteria for the Restudy and the original study are identical--the top of curb.

The criteria changes discussed above are, to some extent, off-setting. While the free discharge would tend to increase flooding, the smaller roughness value and the raising of flood depth criteria will result in less flooding being shown on the flood maps. Since the increase in discharge is difficult to determine, it is not possible to quantify the overall effect on the flood boundaries.

The Restudy also includes a change in analysis approach. Rather than using "lines" to analyze flow crossing the lines, the Restudy uses analysis points to locate and characterize flows.

This approach tends to show a concentration of flows, while the "line" analysis assumes the flow may cross anywhere or everywhere along the line. While the "point" approach may result in changes to the original AMDS flood boundaries, the new boundaries should be more accurate.

BASIN, ANALYSIS POINT, AND MANHOLE IDENTIFICATION NUMBERS

A variety of identification labels are required to present findings and systematize modeling. The labels used in this study are explained in this section.

Basin Identification Numbers. A basin can be loosely defined as that land drained by a waterway having a unique outfall from the study area. 37 such basins were identified and a HYMO model prepared for each basin. These basins were labeled A through Z and AA through NN. To avoid confusion, labels I, O, II and OO were not used.

The subbasins within the basins carry the basin letter prefix followed by an Arabic numeral. Thus, Basin U, the largest basin with 54 subbasins, contains Subbasins U1, U2,...U54. Similarly, Basin CC is composed of Subbasins CC1, CC2,...CC12. Two subbasins, U5A and U17A carry an additional suffix, since close scrutiny revealed the need to further divide subbasins U5 and U17.

Analysis Point Identification Numbers. All analysis point labels begin with the prefix AP (Analysis Point). Next follows the identification of the subbasin within which the analysis point lies: APDD1. Some analysis points have both storm sewers and overland flow passing the analysis point. To distinguish these separate flows, the analysis point label may add the suffixes SS (storm sewer) or OL (overland flow), resulting in a label such as: APX1-SS and APX1-OL.

Most subbasins have only one analysis point, but a significant number have multiple analysis points. These points have an additional clarifying suffix: APV1A to distinguish it from APV1 or APW1SS-2 to distinguish it from APW1SS-1 and APW1SS-3. Subbasins and analysis points are shown on the modeling map.

Manhole Identification Numbers. The inventory of existing storm drainage facilities is shown on the Modeling Map. The manholes on each end of a pipe section are used to identify that section. This report uses the City of Albuquerque numbering system for the manholes. This system begins the label with an "S" (for storm sewer) followed by a 1, 2 or 3 digit Arabic numeral: S13 or S468. These numbers may be repeated, but not within the same Zone Atlas sheet.

Several storm system improvements are included in this analysis which have been designed but not yet installed. These improvements are:

Highland Detention Basin System, Phase II and III
South San Mateo Storm Drain Extension, Phase I and II

Since the Design Division of the City Engineering Group only assigns numbers after installation, the manholes for these systems have been labeled with letters in this report. Manholes labeled G, BF, K, etc., are in this category. As with the City system, these lettered labels may be repeated, but not within the same Zone Atlas sheet.

PUBLIC INFRASTRUCTURE

In order to show potential construction conflicts between the public utility infrastructure and proposed storm sewer facilities, the utility infrastructure is presented on the flood hazard maps. Underground utilities are shown on collector and arterial streets and on local streets carrying major flows.

Line locations and sizes were obtained from the Gas Company of New Mexico, Public Service Company of New Mexico and Mountain Bell. Sanitary sewer and water lines and sizes were taken from the City facilities maps. Sanitary sewer lines 10" or larger, gas lines 6" or larger, water lines 8" or larger, primary electric lines and all underground telephone lines are shown.

To avoid clutter, a single dashed line represents all utilities. If more than one utility exists in a given location, one line is shown but each utility denoted by a letter, G for gas, W for water, S for sanitary sewer, E for underground electrical and T for underground telephone. Where clarification is needed to show where lines end, a circle is placed on the dashed line.

SUMMARY OF EXISTING FACILITIES

Tab I contains the 1:500 scale site maps of the study area. Shown on these maps are the 10-year and 100-year flood boundaries found in this study. The major existing utility infrastructure is presented along with the analysis points which reference the flowrates used to determine the floodplains.

Tab II presents the hydraulic information describing flows at the analysis points. The discharge, velocity and flow depth for storm sewer pipes and surface flow are tabulated for the 10- year and 100-year storms. This table also provides the location of the analysis points as well as the Zone Atlas maps on which the points are located.

Tab III contains a summary of the subbasin hydrology. The 10-year and 100-year discharges of each subbasin are presented as well as the HYMO model input parameters--area, curve number, percent impervious, unit hydrograph time to peak and recession coefficient K.

When comparing the Restudy flood limits with the original AMDS Volume II, the similarities and differences in the hydrologic models, the hydraulic analyses and runoff criteria must be considered:

1. Both studies used HYMO for modeling. Following City of Albuquerque guidance, the restudy duplicated the original study approach of separating the subbasins into pervious, direct runoff impervious and detained runoff impervious areas (see page 13). The restudy percents impervious and curve numbers are similar to those used in the original study.

2. The original study made extensive use of analysis lines, while the Restudy uses analysis points. The "point" approach tends to concentrate flows and should yield deeper but less widespread flooding.

3. The Criteria changes listed on page 21 are, to some extent, offsetting. While the assumption of free discharge used in the Restudy should produce more flooding, the new street friction factor and the less stringent depth to flood criteria should result in a decrease of flooding.

The above differences between the two studies result in various areas being susceptible to flooding in the Restudy that previously were not susceptible under the original study. Likewise, some areas shown as susceptible to flooding in the original study are shown as flood-free in the Restudy.

The study revealed various locations susceptible to 100-year and 10-year flooding within the study area. The following discussions describe the major existing drainage systems, give a brief synopsis of their function and effectiveness and summarize expected flooding. The Existing Storm Drainage Facilities and Site Map located in the back pocket of this report shows these systems.

The **Central Avenue Storm Water Collector System** collects storm runoff in underground conduits along Central Avenue between Tramway Boulevard and Eubank Boulevard and conveys it to the intersection of Eubank and Copper Avenue. Here, runoff from the residential development bounded by Juan Tabo, Chico Road, Eubank and Central is collected. These flows are then conveyed along Eubank to the Interstate 40 channel.

The portion of this system which collects flows on Central is adequately sized for the 100-year storm, as is the pipeline carrying flow from the intersection of Chico and Morris west on Chico to

Eubank. However, the line underneath Eubank is undersized for the 100-year storm from Chico north to Interstate 40.

The subbasins served by the Central Avenue Storm Water Collectors System include A-1 through A-5, U9 through U13, M15 and B1.

The **Wyoming Storm Water Collector System** drains the residential development bounded by Eubank, Interstate 40, Wyoming Boulevard, and Central and conveys it to the Interstate 40 channel.

This system is adequate to convey both the 100-year and 10-year storm without flooding.

The subbasins served by the Wyoming Storm Water Collector System include M14, M15, F4 through F7, C1 and D1.

The **Pennsylvania Storm Water Collector System** collects storm water from the intensely developed light industrial area east of Moon Street and south of Central Avenue as well as the developed areas along Central east of Wyoming along Zuni Road from Central to Pennsylvania. These flows are conveyed northward along Pennsylvania and Dallas Street to Chico where runoff from the residential area between Wyoming and Pennsylvania and north of Central are intercepted. Flows are then conveyed to the Interstate 40 channel along Dallas Street.

Although a pocket of flooding has been reported near Mesilla Street and Constitution, the remainder of the area is flood free during the 10-year and 100-year storms.

The subbasins served by the Pennsylvania Storm Water Collector System include U7 through U9, U21 through U23, M11 through M14, F2, F3 and F6.

The **San Pedro Storm Water Collector System** collects runoff from the area directly east of the New Mexico State Fair Grounds along Louisiana Boulevard and conveys it across the Fair Grounds to San Pedro, Boulevard. Storm runoff generated on Lomas east of San Pedro, as well as from the residential area between Louisiana and San Pedro north of Lomas, is also collected. These flows are then conveyed to the Interstate 40 channel. In 1983, The **Fairgrounds Detention Basin and Storm Water Collection Extension** were added to the San Pedro system, extending it east of the Fairgrounds and south down Alcazar Street to Trumbull Avenue.

This study found that, with the installation of the 1983 improvement, the combined systems now are adequate to prevent 10-year and 100-year flooding in the area they serve. A small pocket of 100-year flooding is reported at the intersection of Zimmerman Avenue and San Pedro.

The subbasins served by the combined San Pedro Storm Water Collector System and Fairgrounds Detention Basin and Storm Water Collection Extension include H1, G1, K4, K6 through K9, M10 through M12, M16, M18, U7, U19, and U20.

The **San Mateo Storm Water Collector System**, after completion of Phases I, II and III, will collect runoff along the Campus Draw from the intersection of Gibson and Louisiana to the corner of Southern and San Mateo. The trunk line proceeds northward to Constitution, collecting runoff in route at every street in between. From Constitution, it continues northwesterly under Sunningdale Street to the Interstate 40 channel.

The recently completed Fairgrounds Detention Basin and Storm Water Collection Extension intercepts much of the flow upstream of the San Mateo system. This project and the completion of the above-mentioned Phases I, II and III will substantially reduce flooding in the area served. The only remaining 100-year flooding is an area bounded by San Mateo, Constitution, Madeira Drive and Summer Avenue (see Zone Atlas sheet J-18).

The subbasins served by the San Mateo Storm Water Collector System includes K1 through K3, K5, M9, M17, M21, U53, U6, U18, U19, U28, U29, U31, U32, U34, U35, U37 through U39 and U41.

The **Campus Draw Storm Water Collector System** was improved in 1985 with the addition of the **Highland Detention Basin** and associated extensions. The completed system collects storm runoff along the Campus Draw from Gibson near San Pedro to the North Diversion Channel near University Boulevard and Lomas.

Since installation of the detention basin, the system has adequate capacity along its entire route for the 10-year storm, and protection through most of the area from the 100-year storm. However, the low-lying streets along the old Campus Draw from the intersection of Anderson Avenue and Ortiz Drive downstream to Southern Avenue and Monroe Avenue are susceptible to flooding during the 100-year flood (see Zone Atlas pages L-17 and L-18).

The subbasins served by the Campus Draw Storm Water Collector System combined with the Highland Detention Basin and Storm Water Collector System include U2 through U5, U15 through U17A, U27, U31 through U34, U36 and U40

The **Rita Storm Water Collector System** collects runoff from the area bounded by Jefferson Street, San Mateo, Constitution and Carlisle Boulevard and conveys it along Morningside Drive, La Hacienda Drive and Rita Drive to the Interstate 40 channel, collecting flows at each intersection along the route.

Although adequately sized for the 10-year storm, flooding from the 100-year storm can be expected along the trunk route from Hacienda Place to the Interstate 40 channel (see Zone atlas pages J-16 and H-16).

The subbasins served by the Rita Storm Water Collector System include L1 through L6, M7 and M8.

The **Princeton Pump Station Storm Water Collector System** collects runoff from the area which drains to Lomas at Tulane Drive plus the Princeton Draw culminating at the Princeton Pump Station and detention facility.

During the 100-year storm, flooding can be expected along the trunk line between the Loma Vista/Mackland intersection and the Girard/Wilson Street intersection. A small pocket of flooding is reported at the intersection of Mountain Road and Calle del Sol. No 10-year storm flooding is expected.

In its Final Report - Volume Two for Albuquerque Storm Water Pumping Stations (1983), Bovay Engineers Inc. states that the pumping station at the North Diversion Channel utilizes four pumps with a rated capacity of 29 cfs each. The report continues, "Detention pond (at this station) is of sufficient size to contain major storms."

The subbasins served by the Princeton Pump Station Storm Water Collector System are M1 through M7.

The **UNM-Ridgecrest Storm Water Collector System** drains the area bounded by Ridgecrest Drive, Central (Lomas west of University), Interstate 25 and Gibson Boulevard. The five subsystems which compose this system are interconnected at several locations. The system drainage area is covered largely by residential development. A majority of the storm sewer within this system was built under design criteria less stringent than the 100-year storm, however, the streets are steep, resulting in advantageous hydraulic capacities.

The only major flooding (100-year) expected in the area begins at Yale and St. Cyr Avenue, runs south on Yale to Bell Avenue, and west on Bell to Buena Vista Drive. From this point, 100-year flooding continues south to Stadium Boulevard (see Zone Atlas page L15). Periodic flooding is reported at the intersection of Lead Avenue and Yale.

The subbasins served by the UNM-Ridgecrest Storm Water Collector System include X1, Y1, Y2, W1 through W5, U4, Z1, AA1, BB1 through BB12, CC2, CC4 through CC11, EE2 and EE4.

CONCLUSION

While much of the study area is protected from the 10-year and 100-year storm, several areas can be expected to experience major flooding during storms of these magnitudes. The most severe impact will be along the Campus Draw. 100-year flooding can also be expected in the vicinity of the Yale Boulevard/Bell Avenue intersection, in the streets occupying the old Princeton draw, along Rita Drive and at the intersection of San Mateo and Constitution. These flood limits and those of other flooding in the study area are presented on 1:500 scale mapping found in Tab 1.

While the flood limits presented in this report were generated using conventional, accepted hydrologic and hydraulic techniques, it is possible that localized, very intense rainfall could produce flooding in other areas or of greater magnitude than presented here. Such isolated and rare events are not considered in conventional urban drainage studies.

This report will be followed by the Phase II Improved Conditions Report determining technologically sound and cost efficient improvements to protect against 10-year and 100-year flooding. These solutions will be presented in the improved conditions report.

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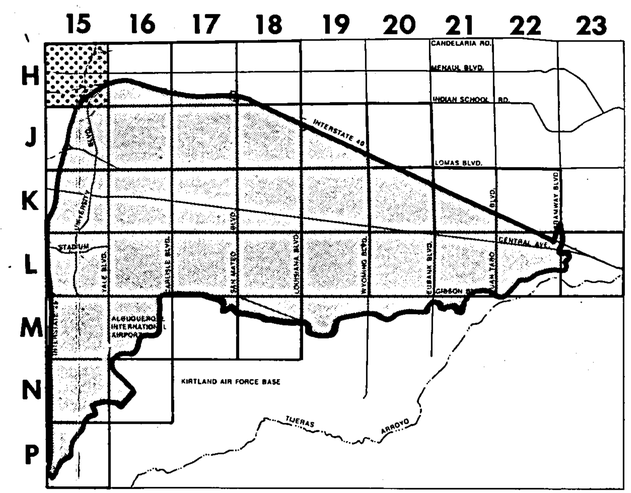

MAP KEY

LEGEND

- STUDY AREA BOUNDARY
- APC8 BASIN ANALYSIS POINT
- 10 YEAR FLOOD HAZARD AREA
- 100 YEAR FLOOD HAZARD AREA
- —36— PROPOSED STORM SEWER

EXISTING UTILITIES

- ----S---- SANITARY SEWER, SIZE 10" OR LARGER
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NOTE: Existing utilities are shown on arterial and collector streets and where drainage facilities are proposed

0 500 1000 1500 2000

SCALE IN FEET
1"=500'

Orthophotography and topography by Bohannan Huston, Inc. from aerial photograph dated 10-8-80.

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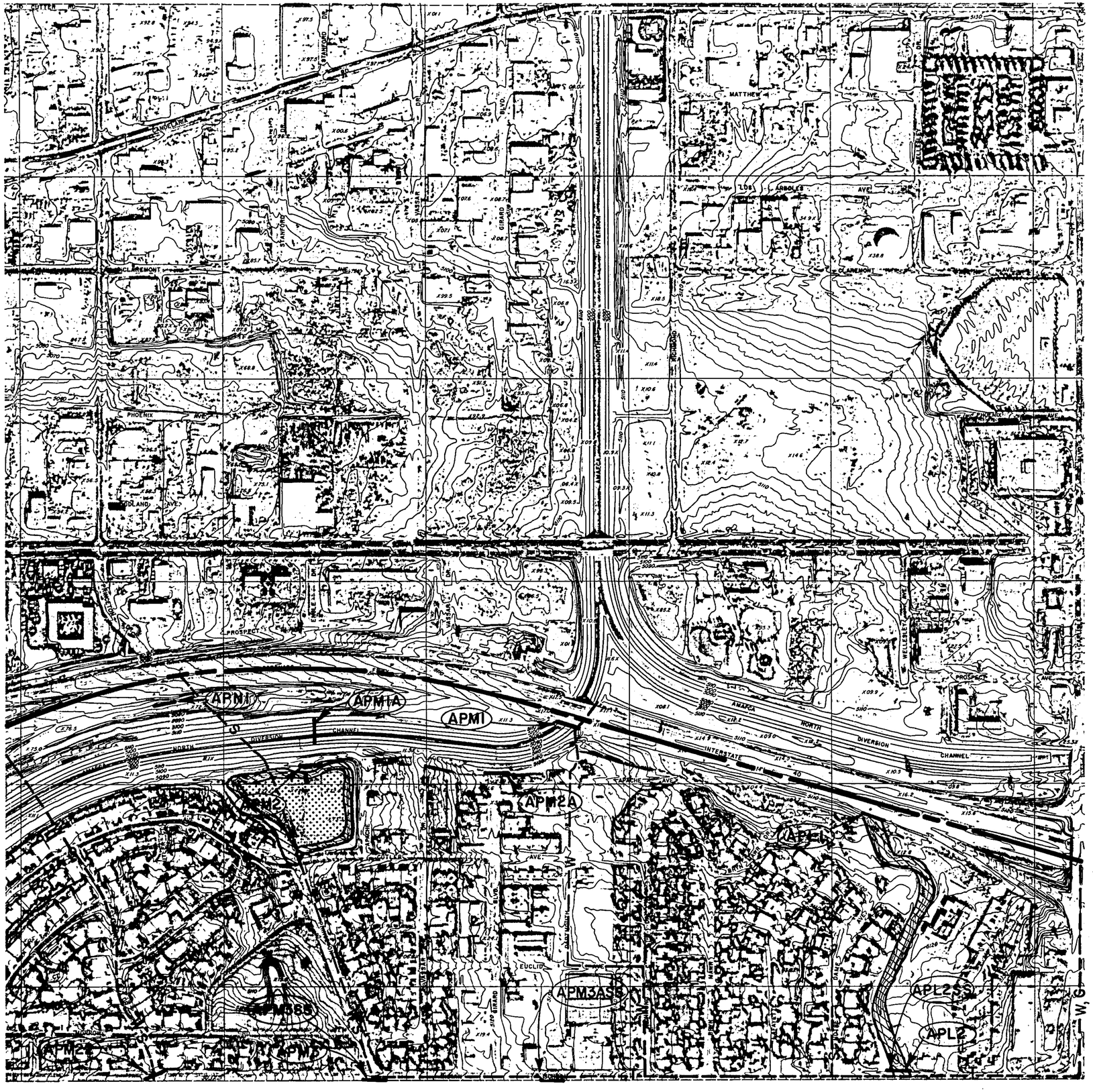
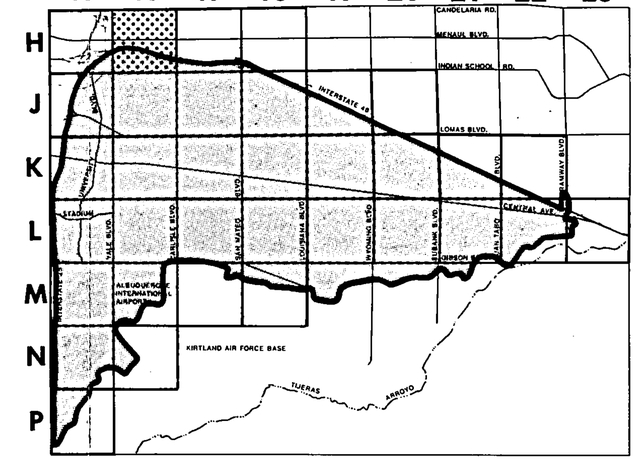
H-15





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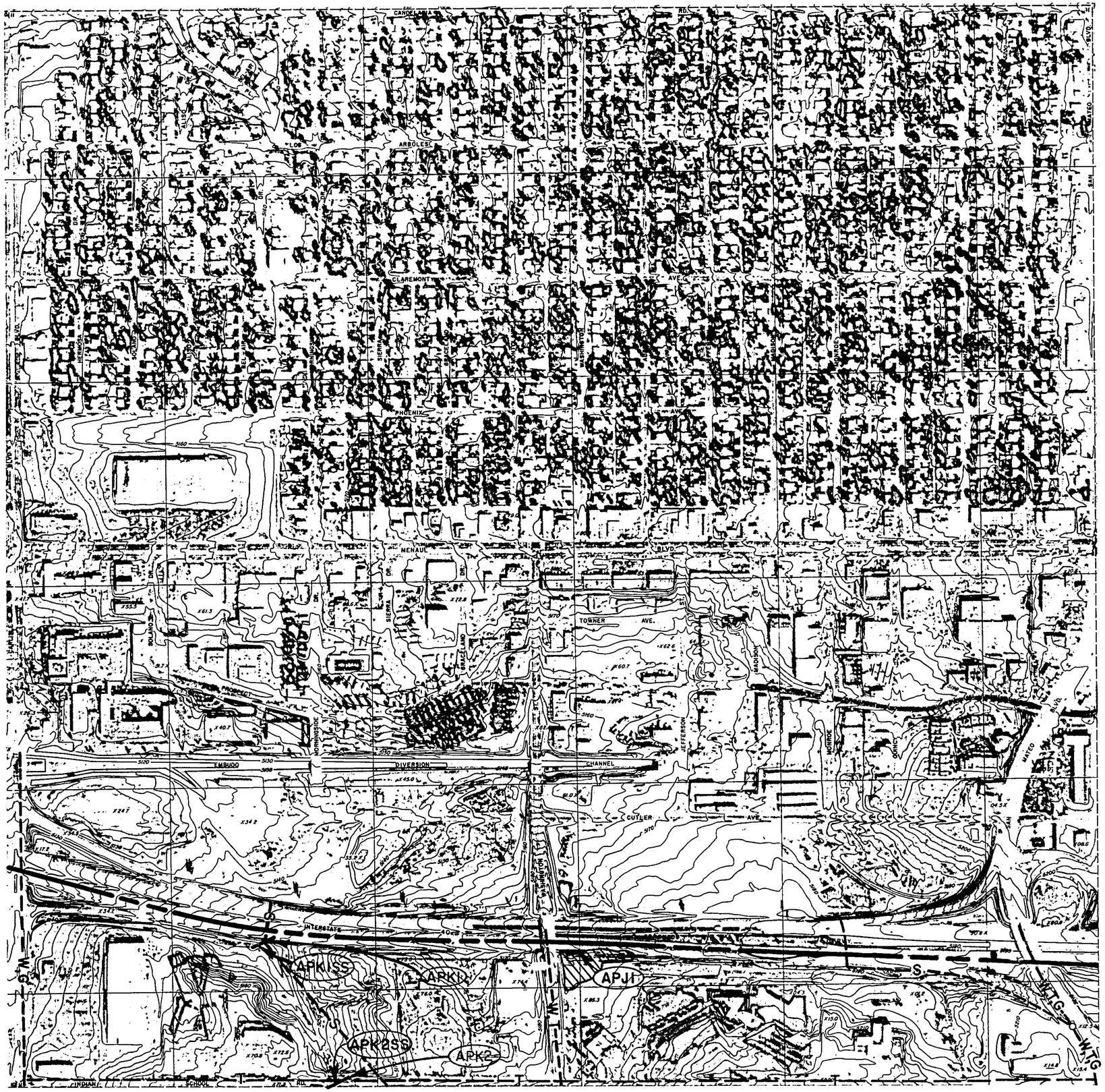
H-16



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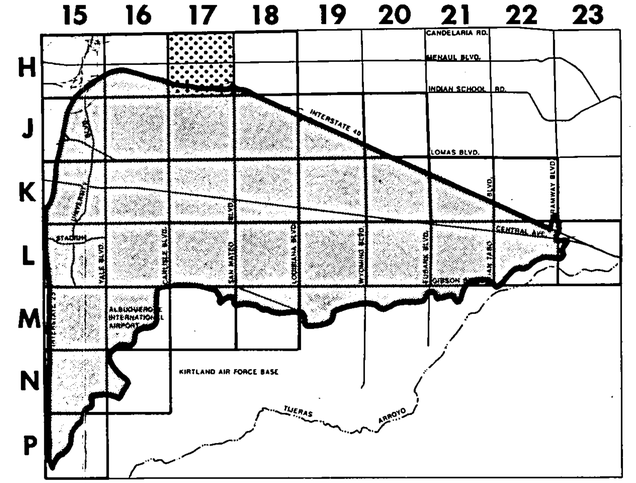

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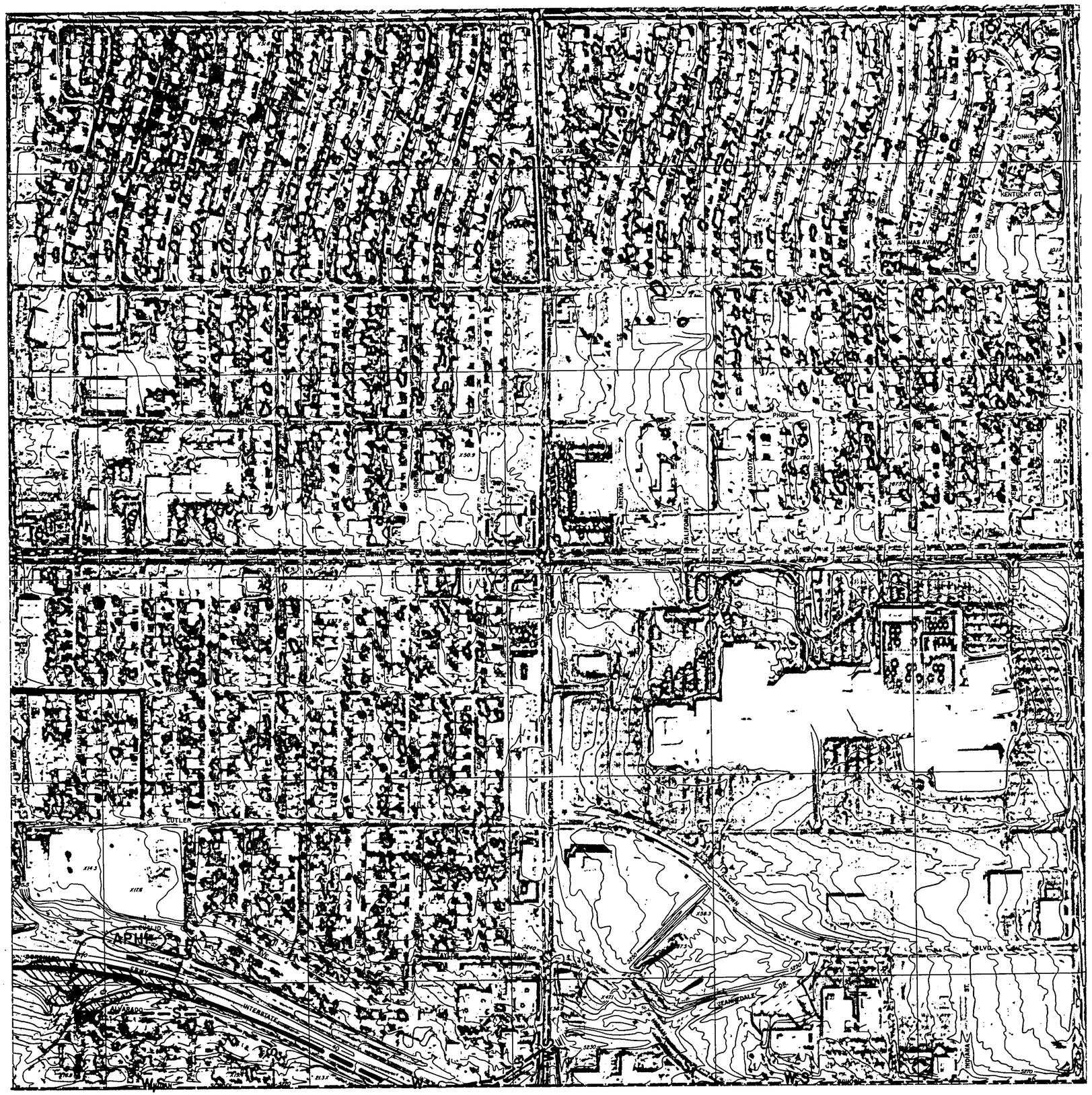

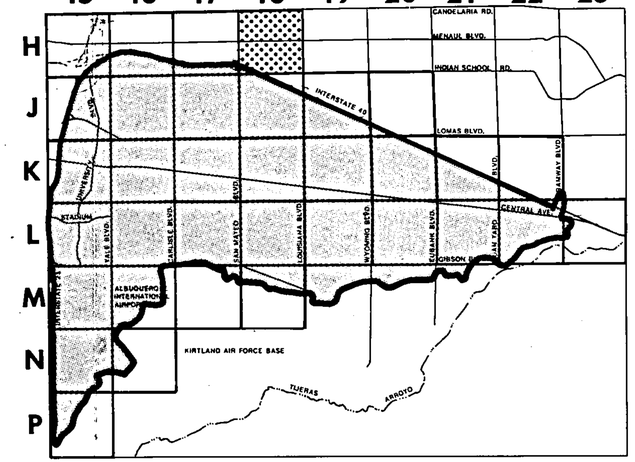

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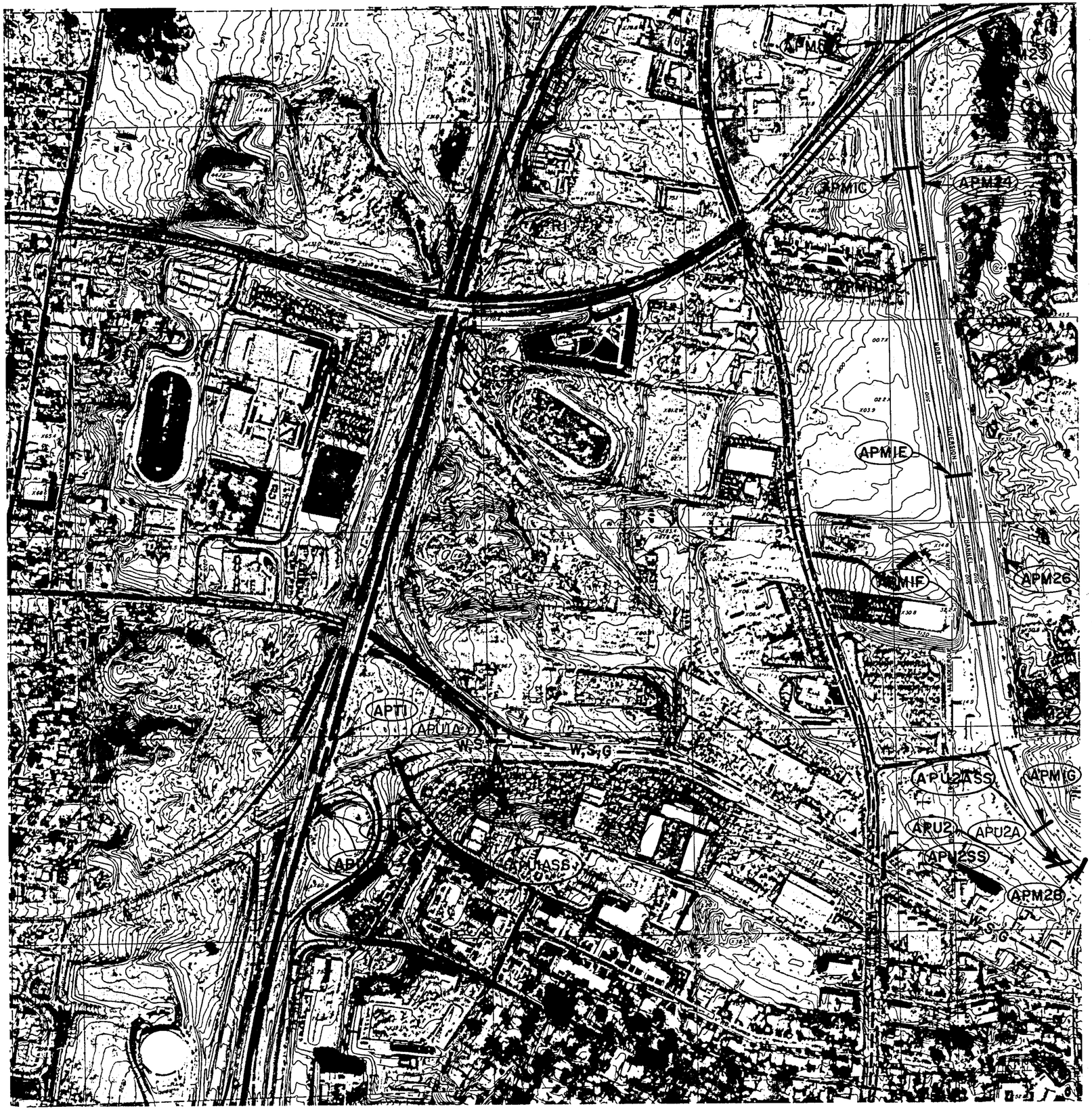

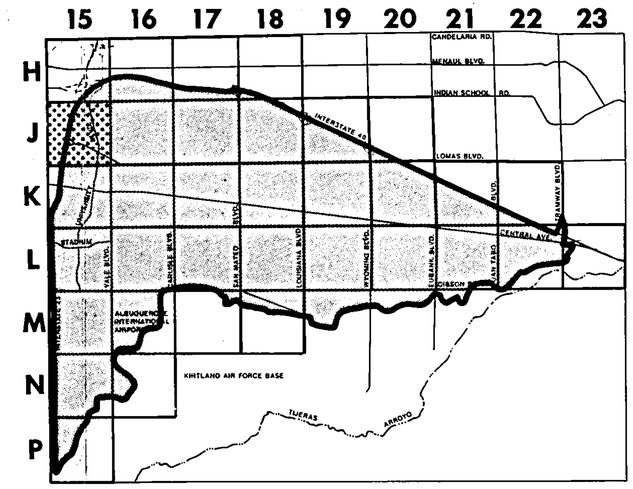

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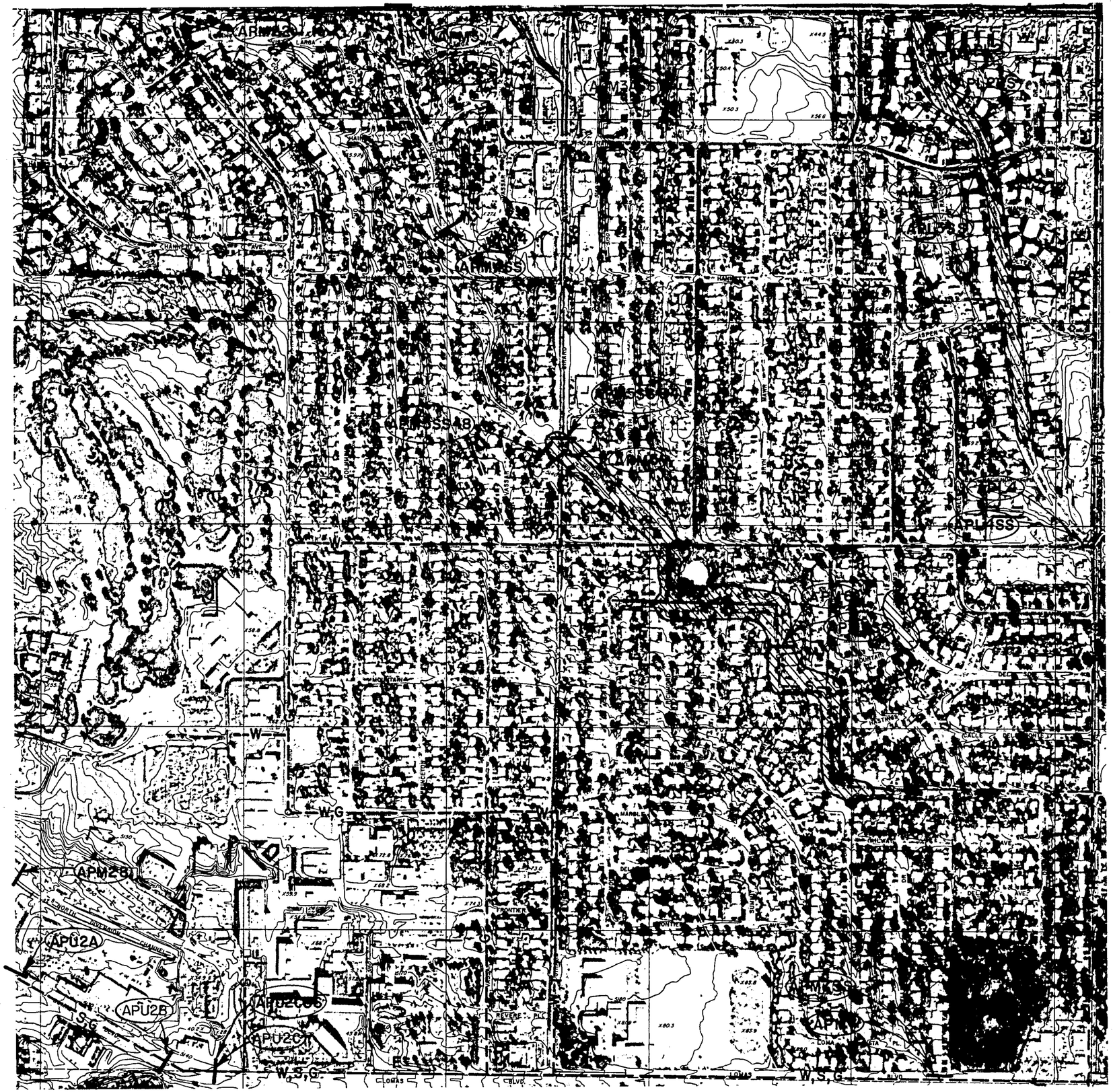
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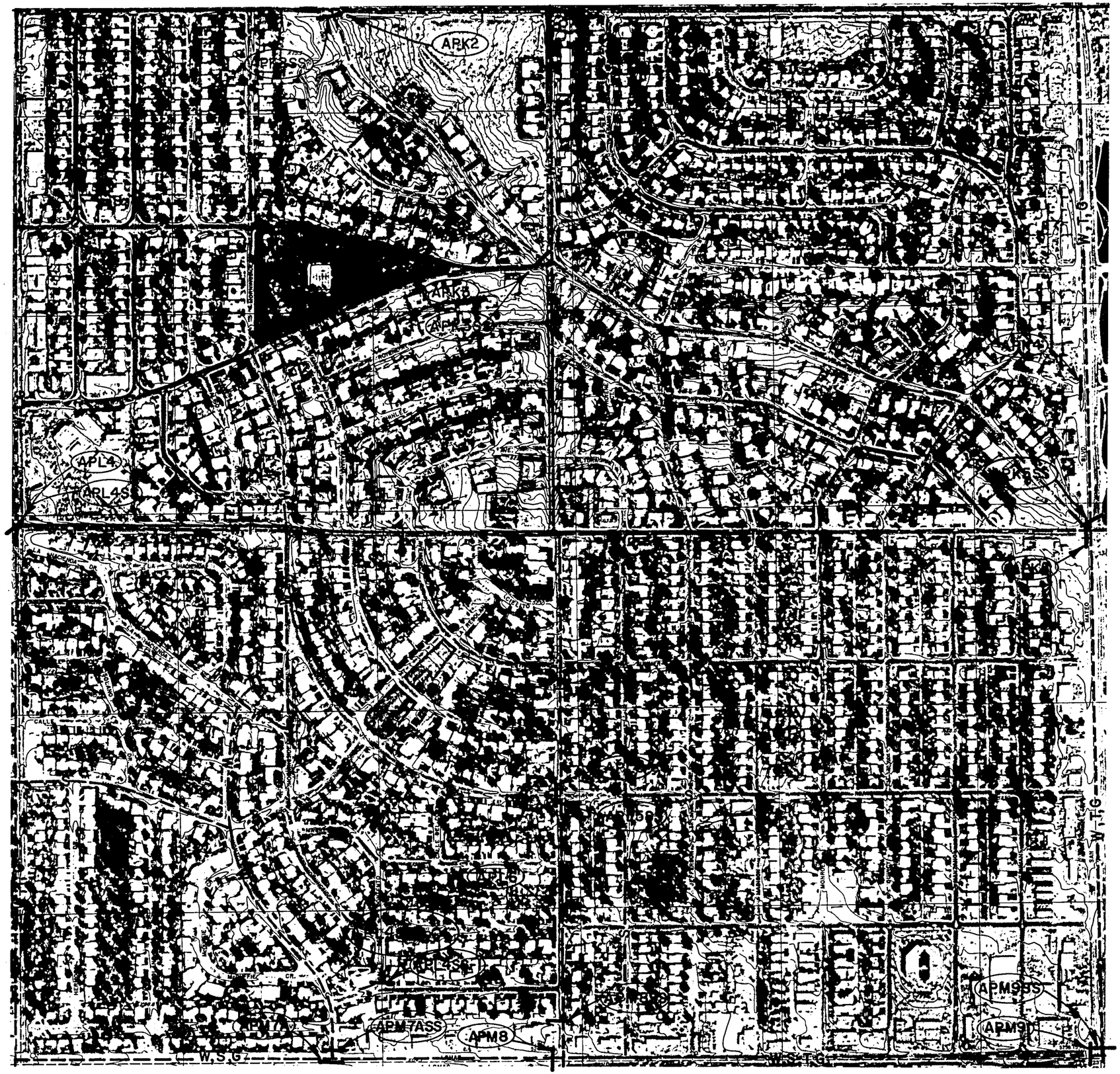

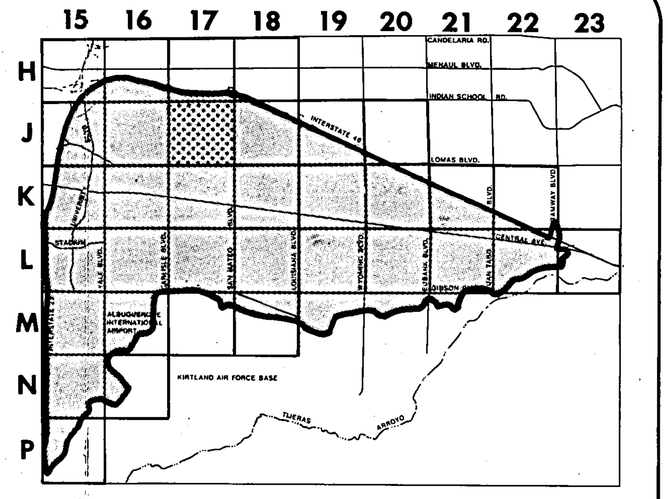

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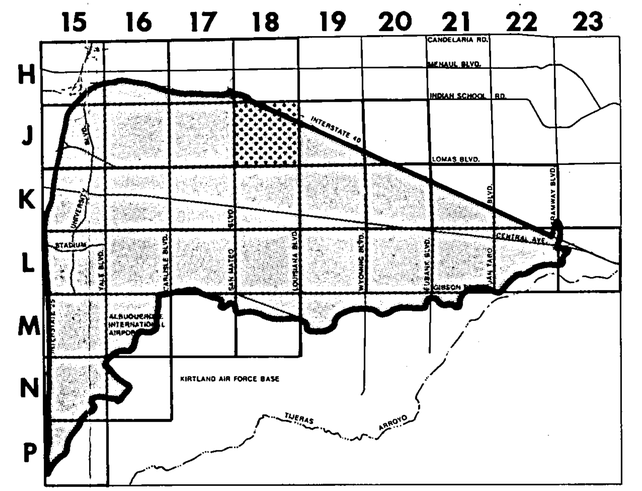

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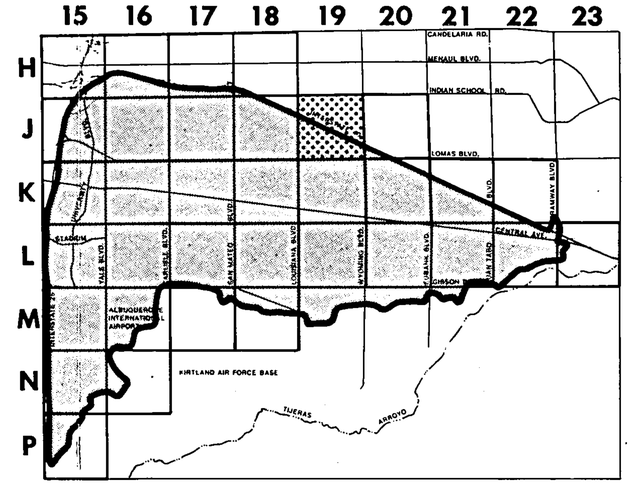

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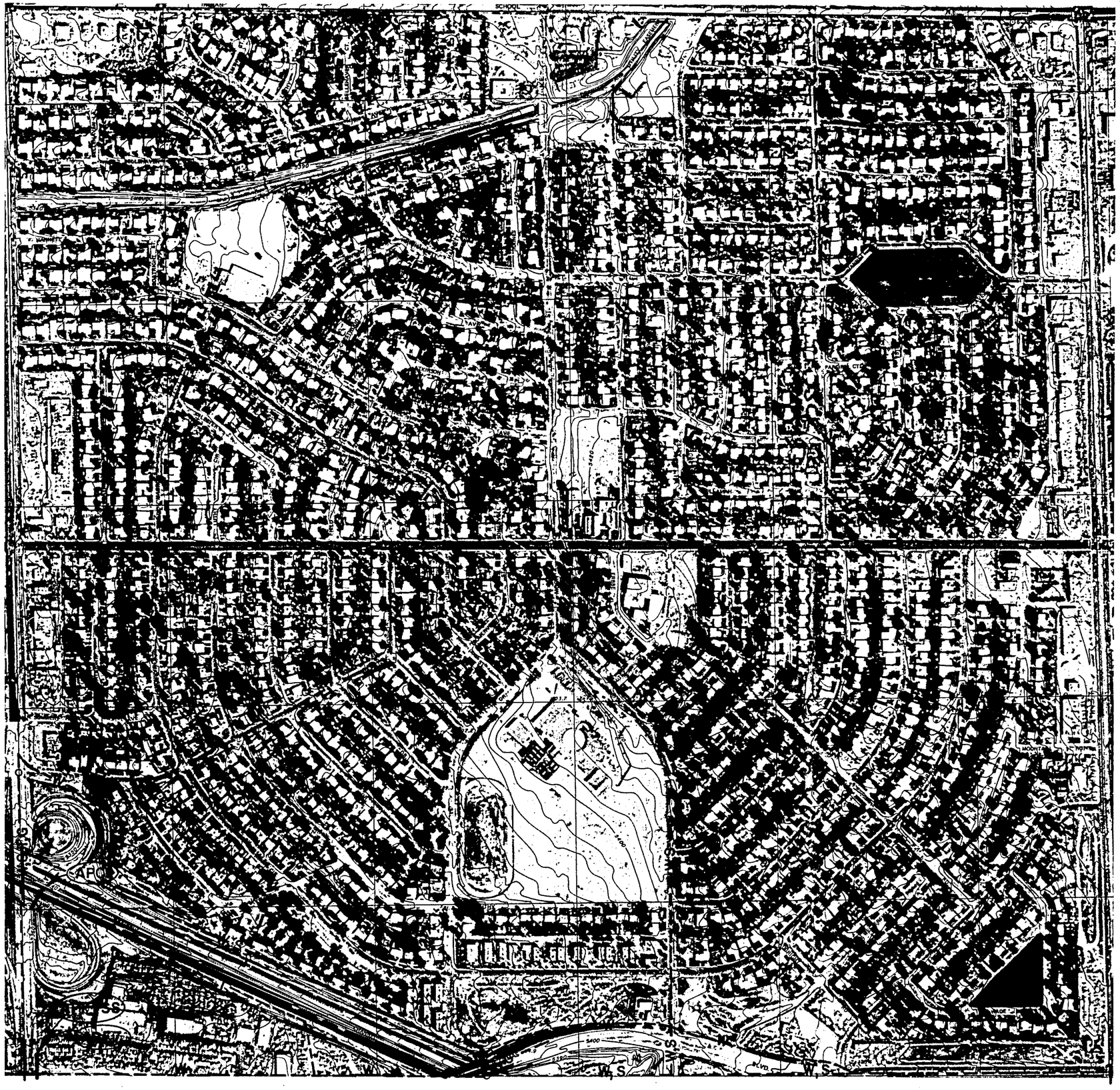
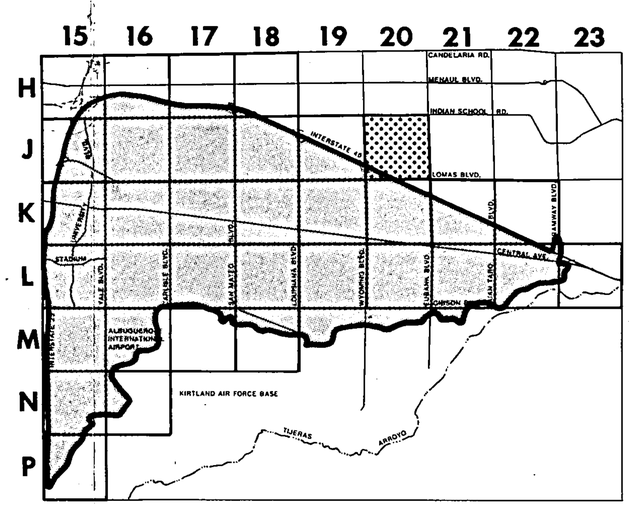
DECEMBER 1986

J-19





MAP KEY

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- STUDY AREA BOUNDARY
- APC8 — BASIN ANALYSIS POINT
- 10 YEAR FLOOD HAZARD AREA
- 100 YEAR FLOOD HAZARD AREA
- —36— PROPOSED STORM SEWER

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SCALE IN FEET

1"=500'

Orthophotography and topography by Bohannan Huston, Inc. from aerial photograph dated 10-8-80.

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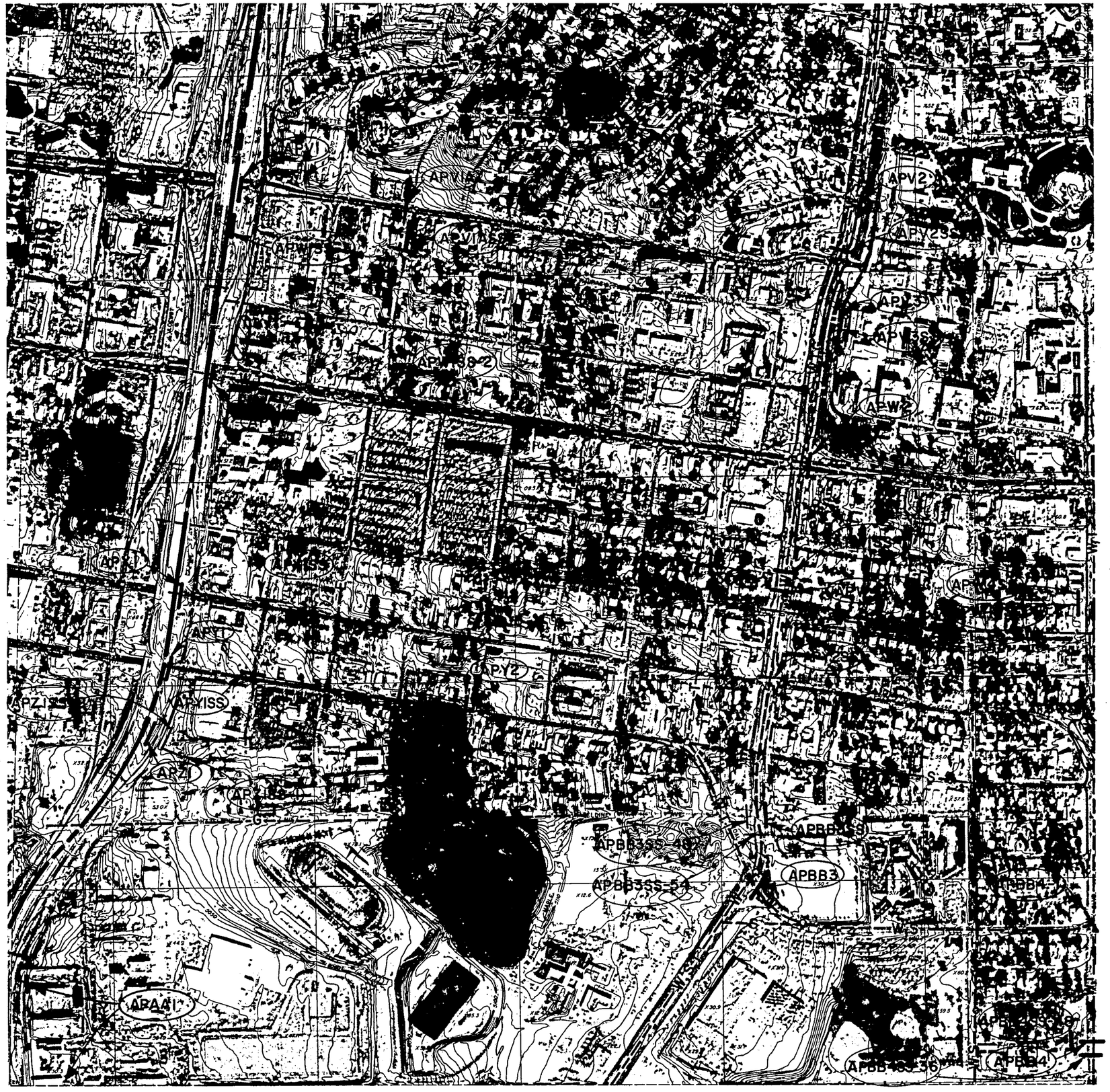
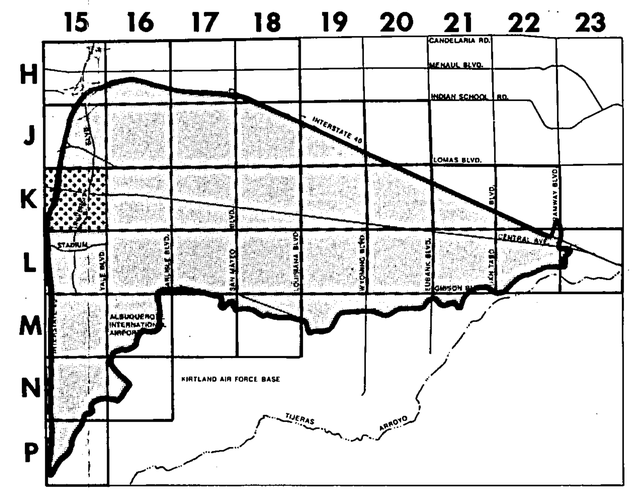
J-20





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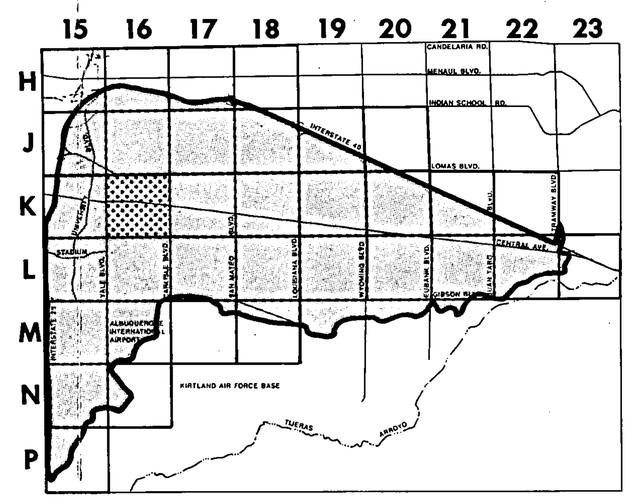

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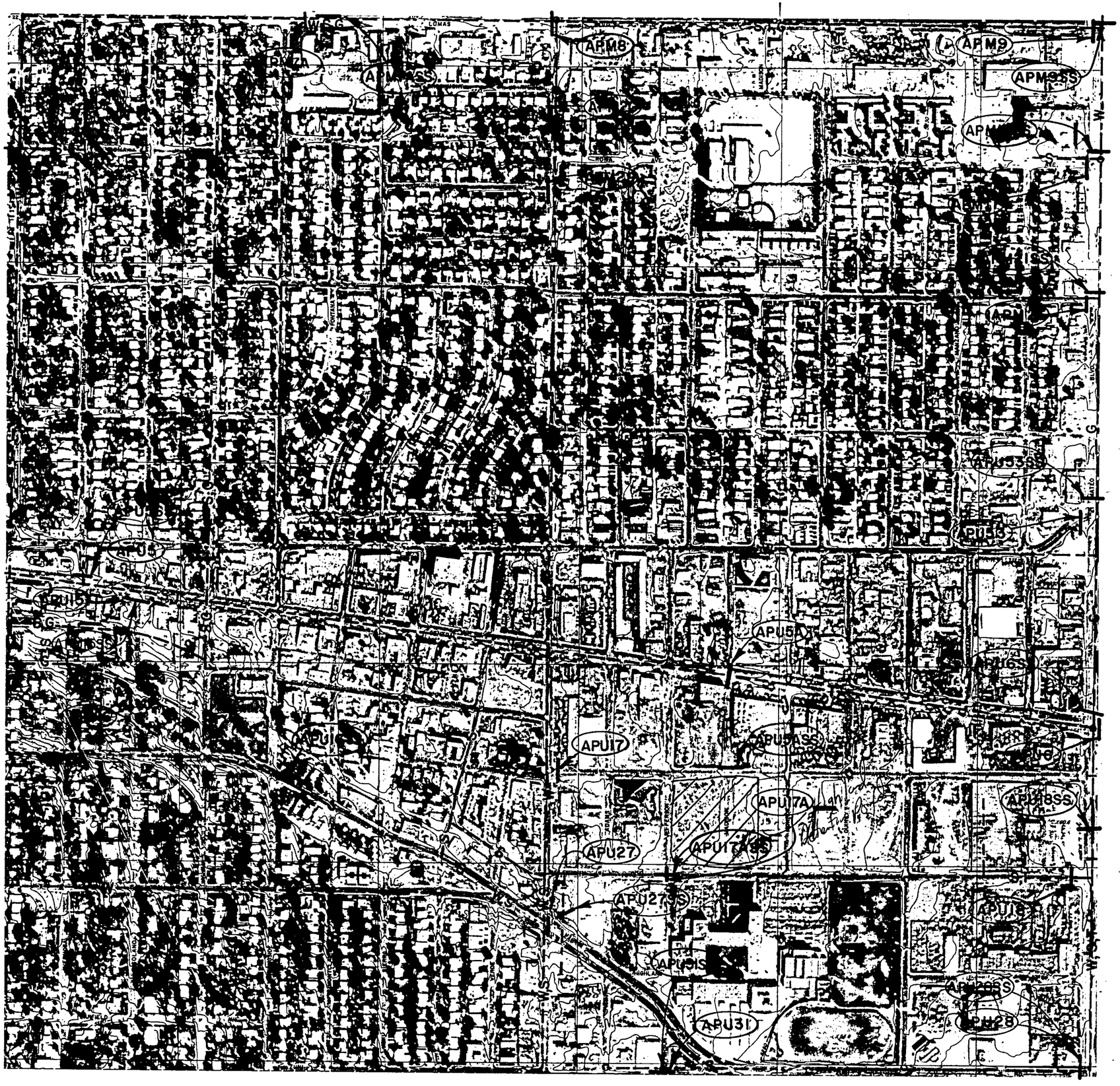

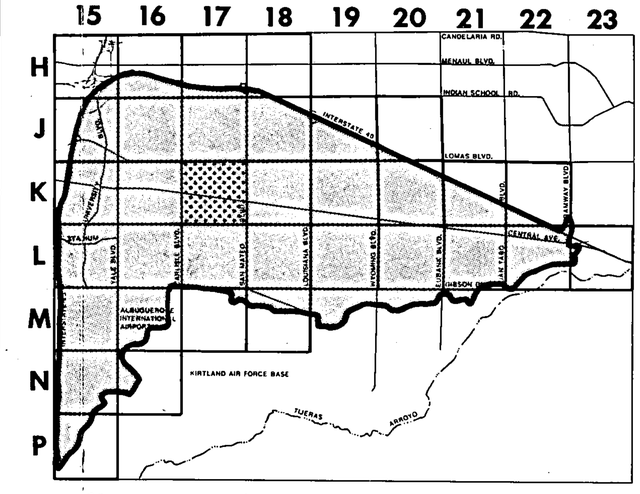

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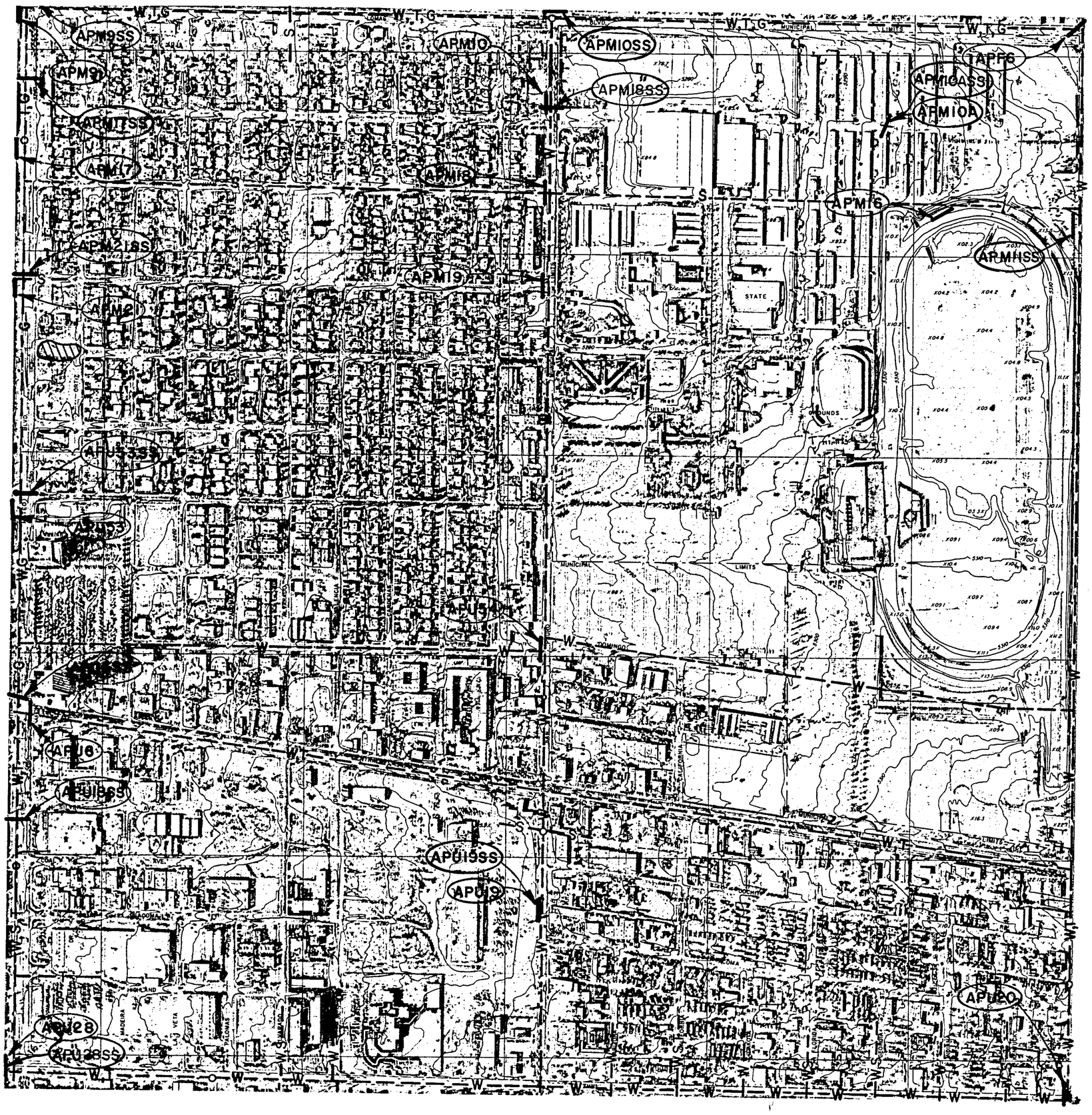
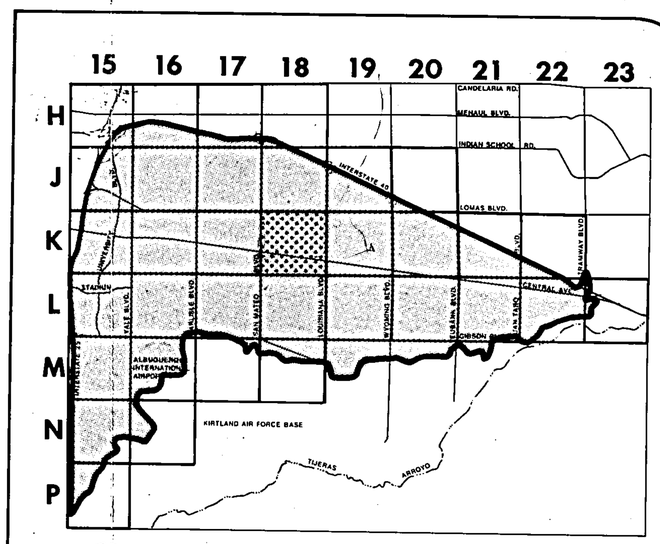
K-17





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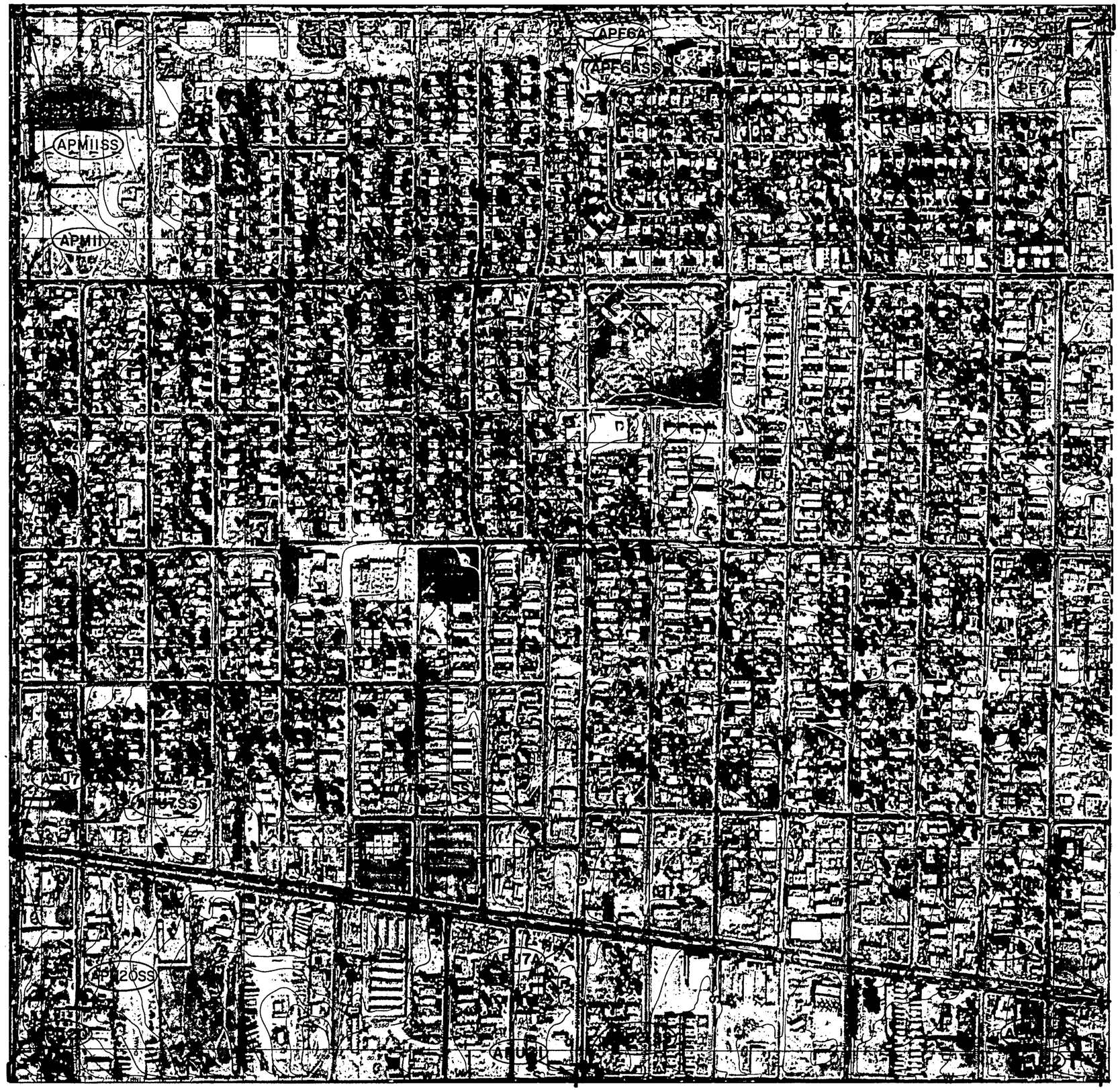

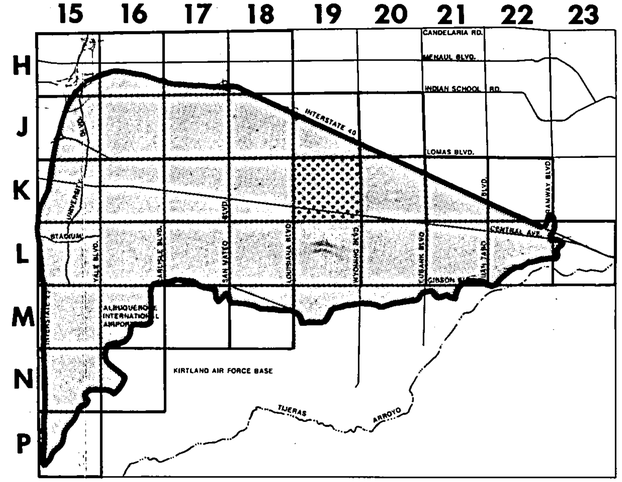

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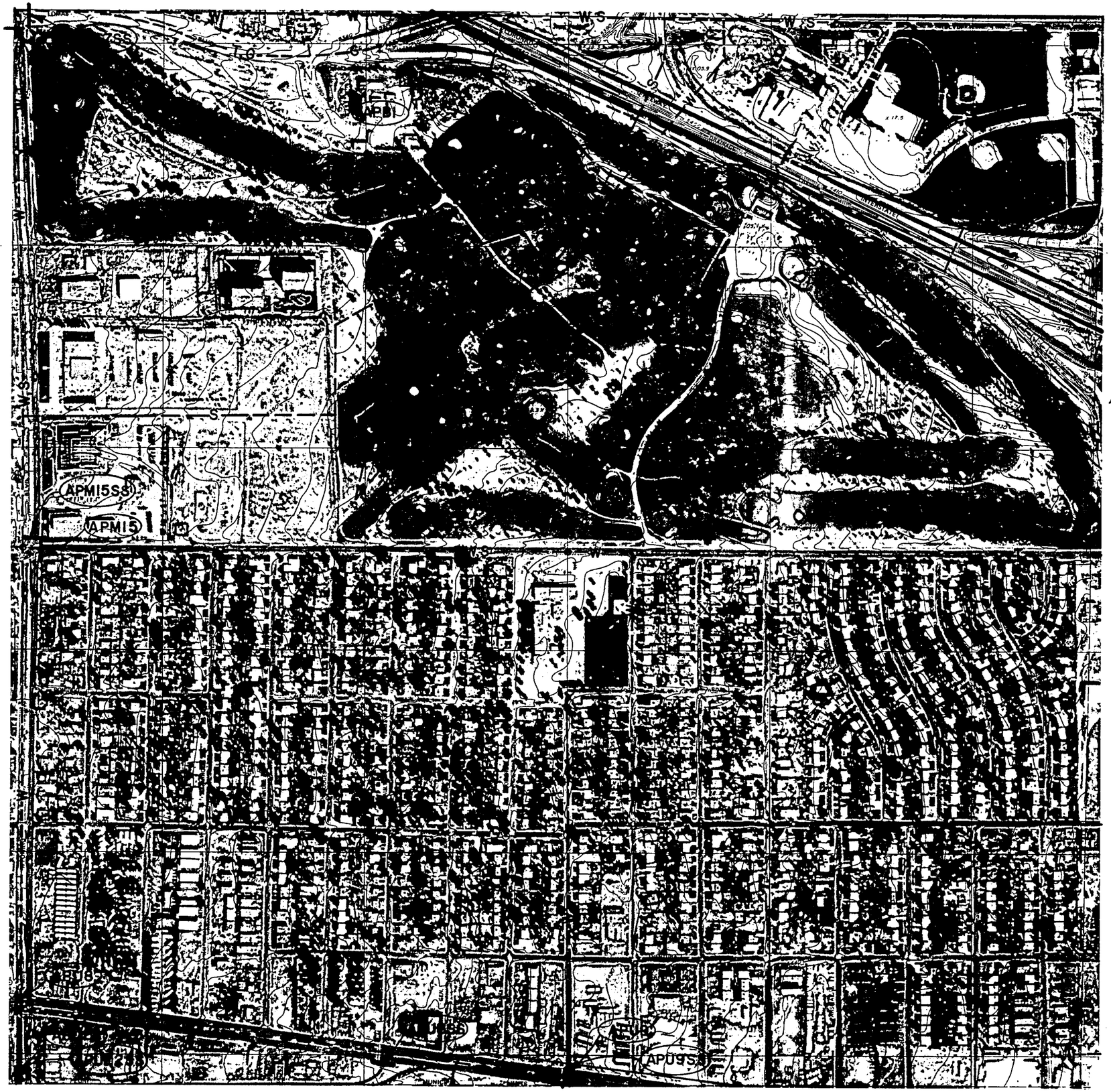

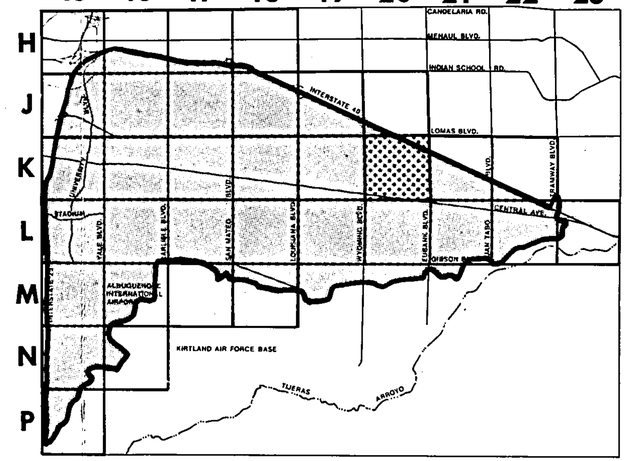

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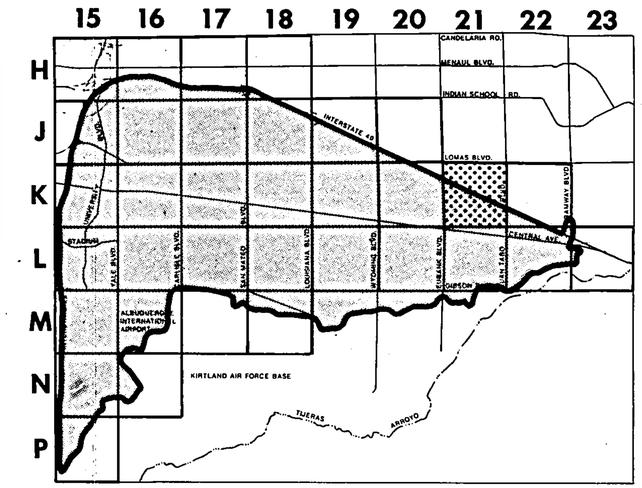

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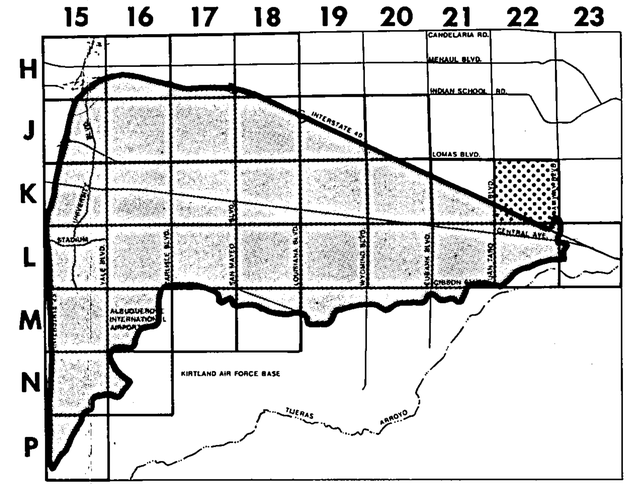
K-21



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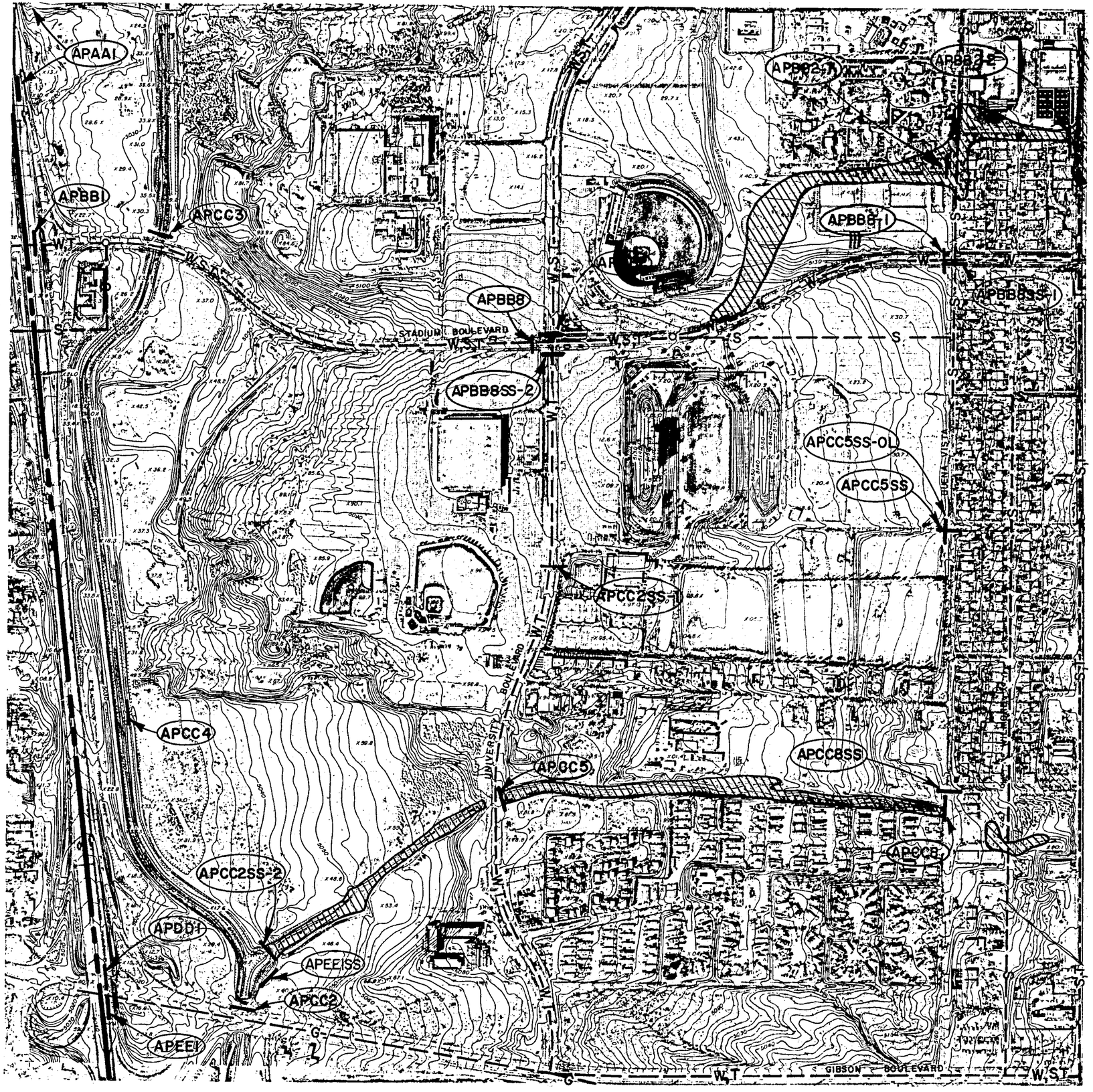

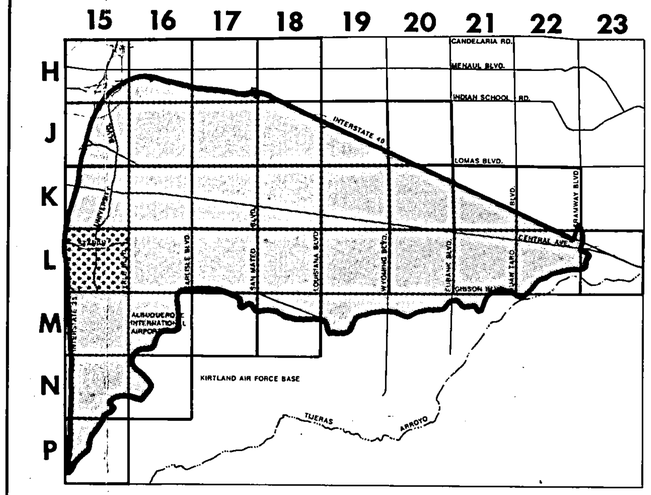

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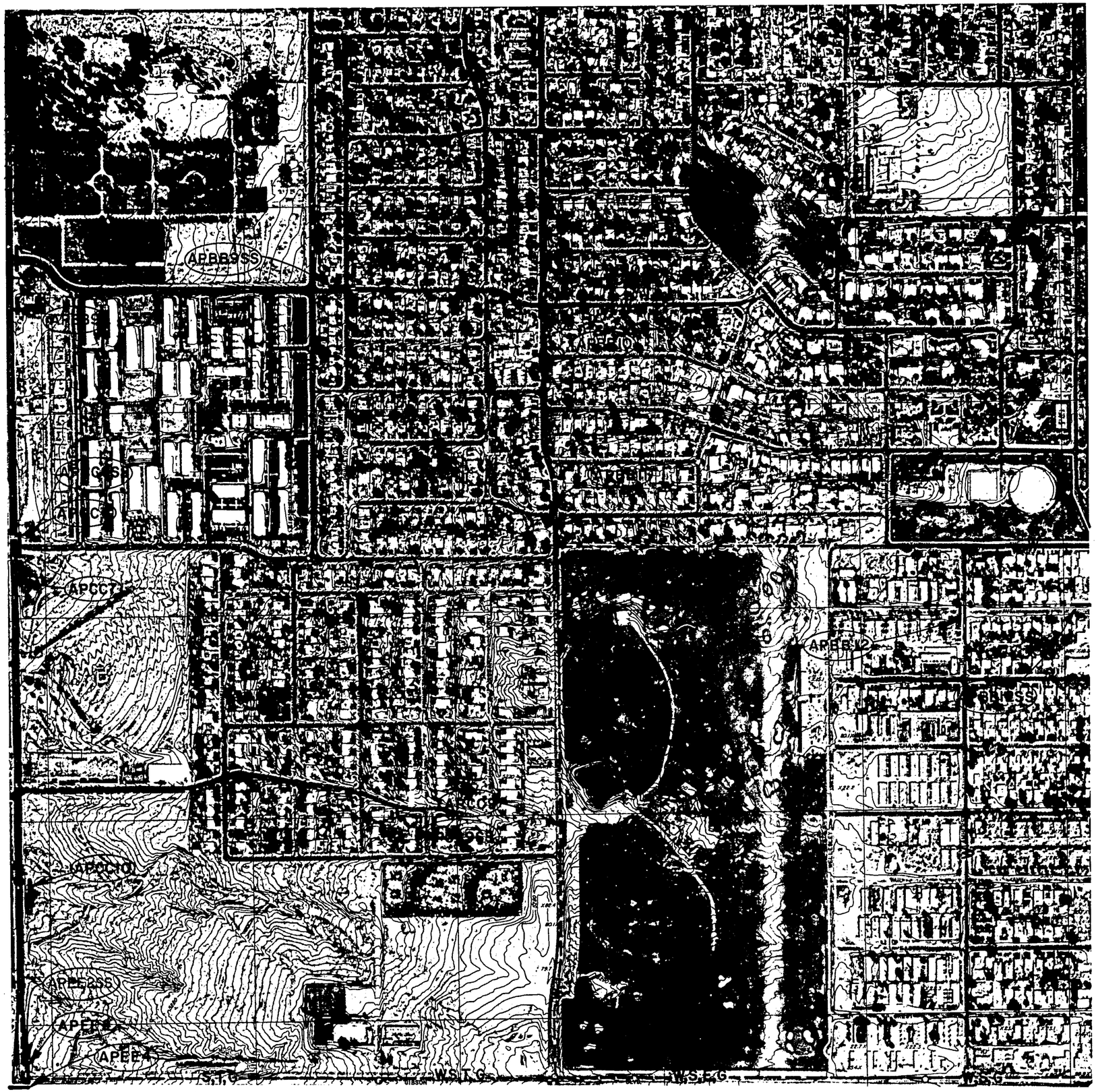
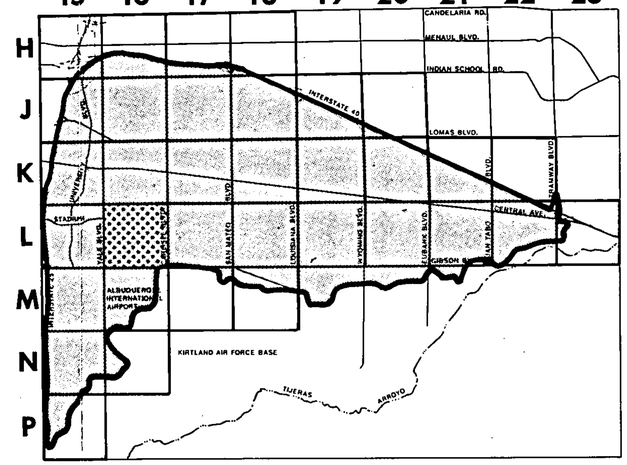
L-15





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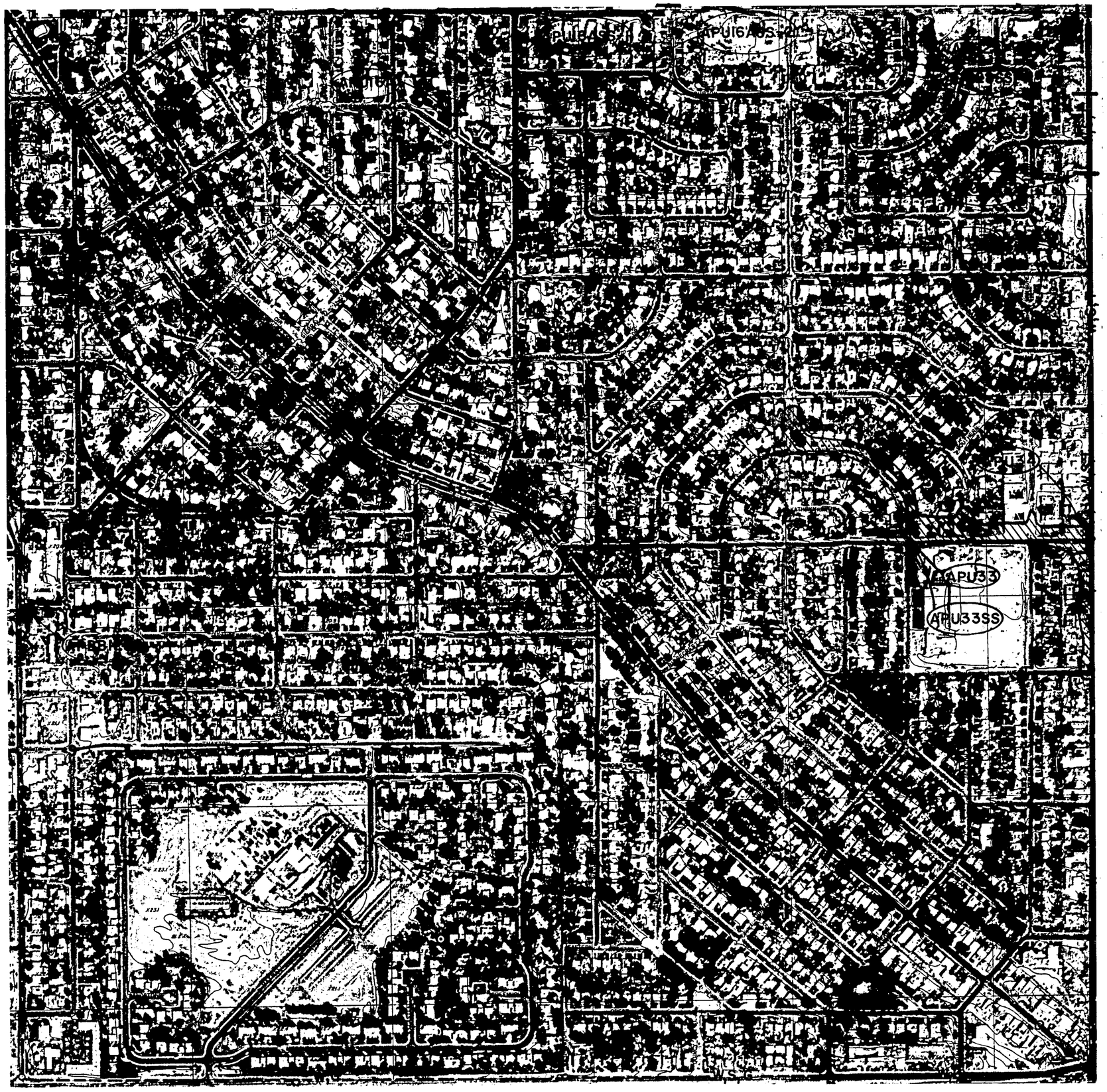

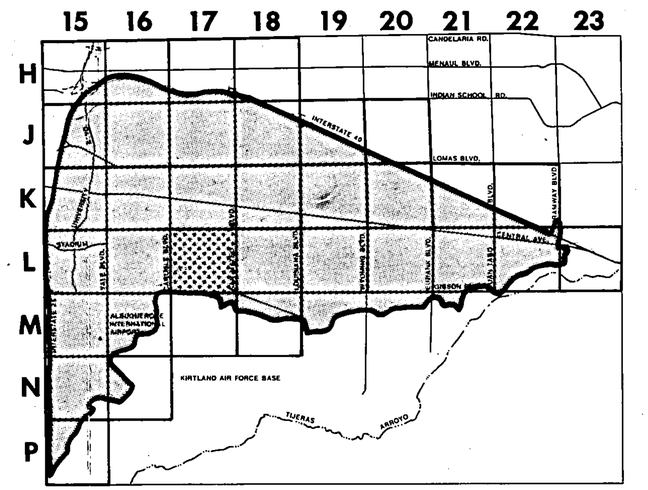

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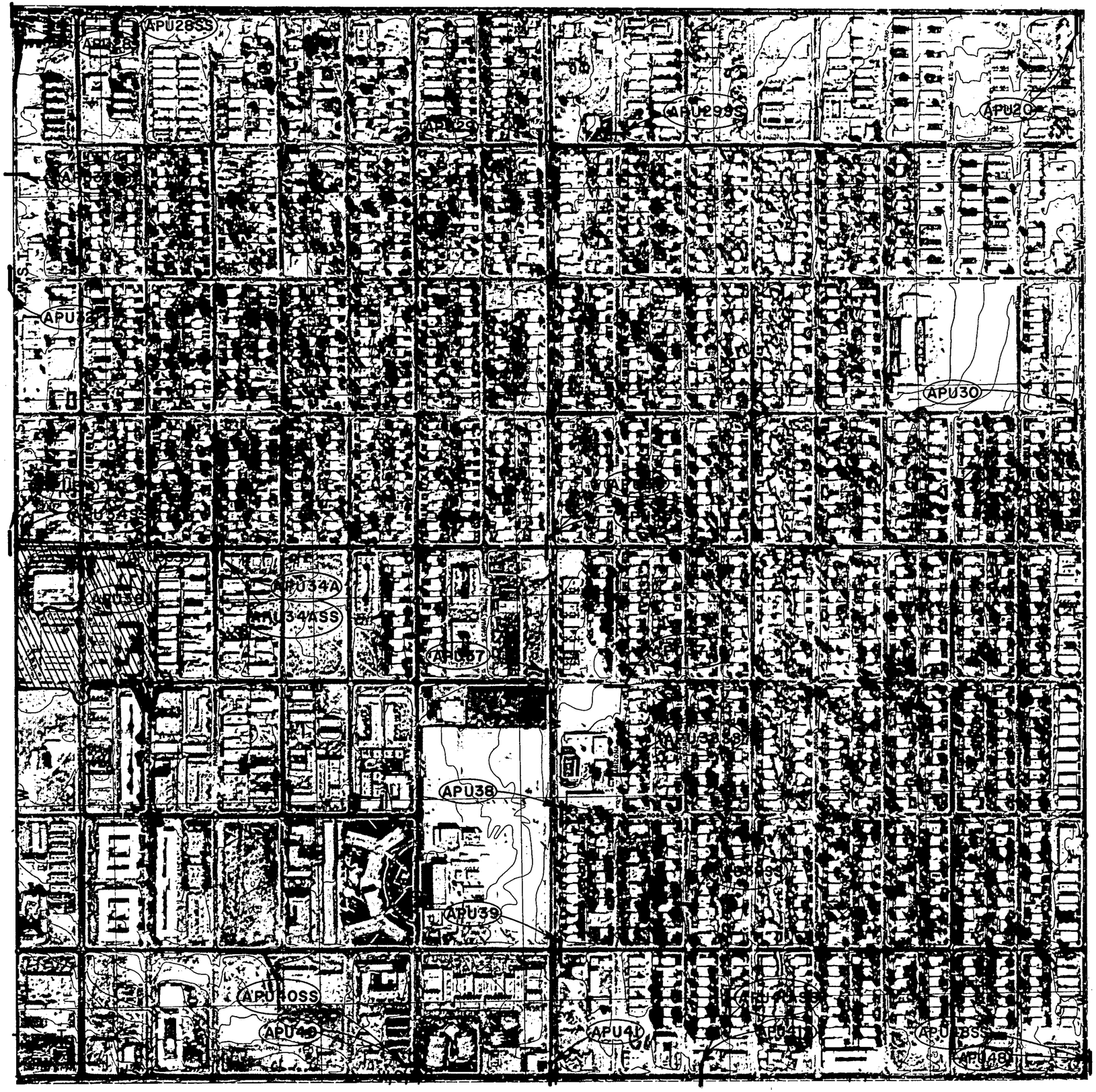

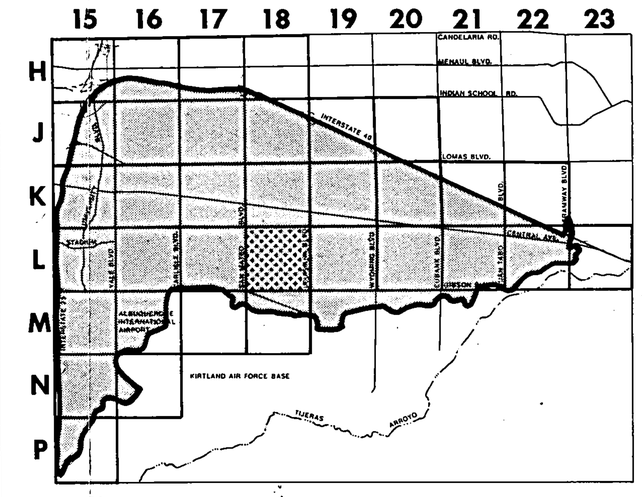

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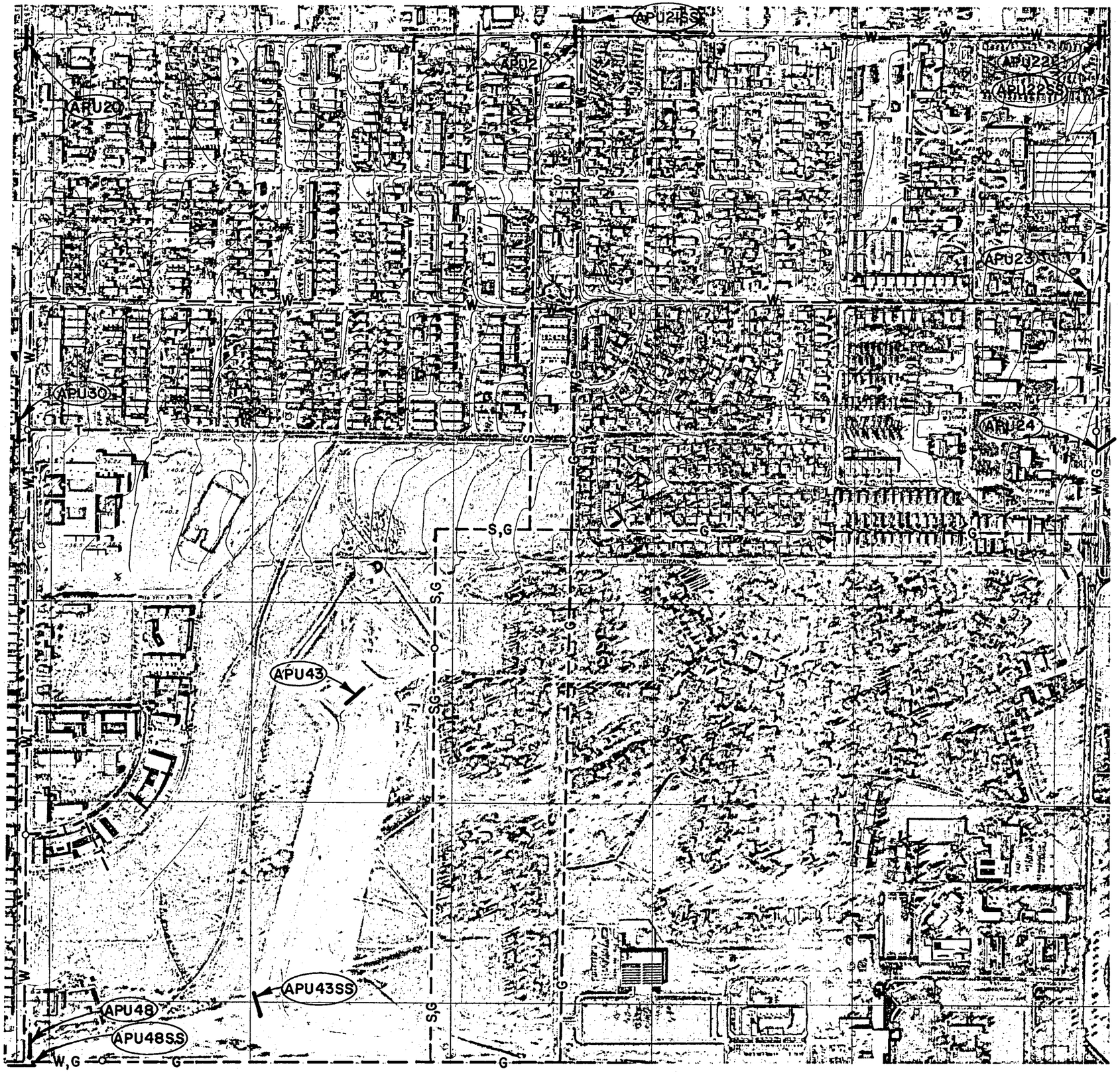

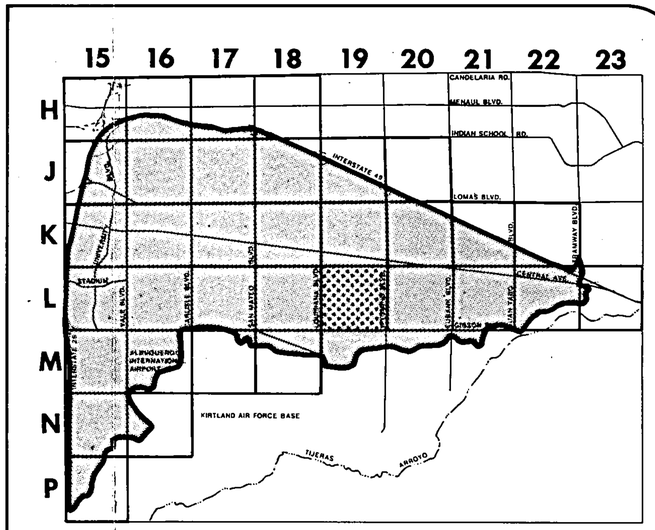

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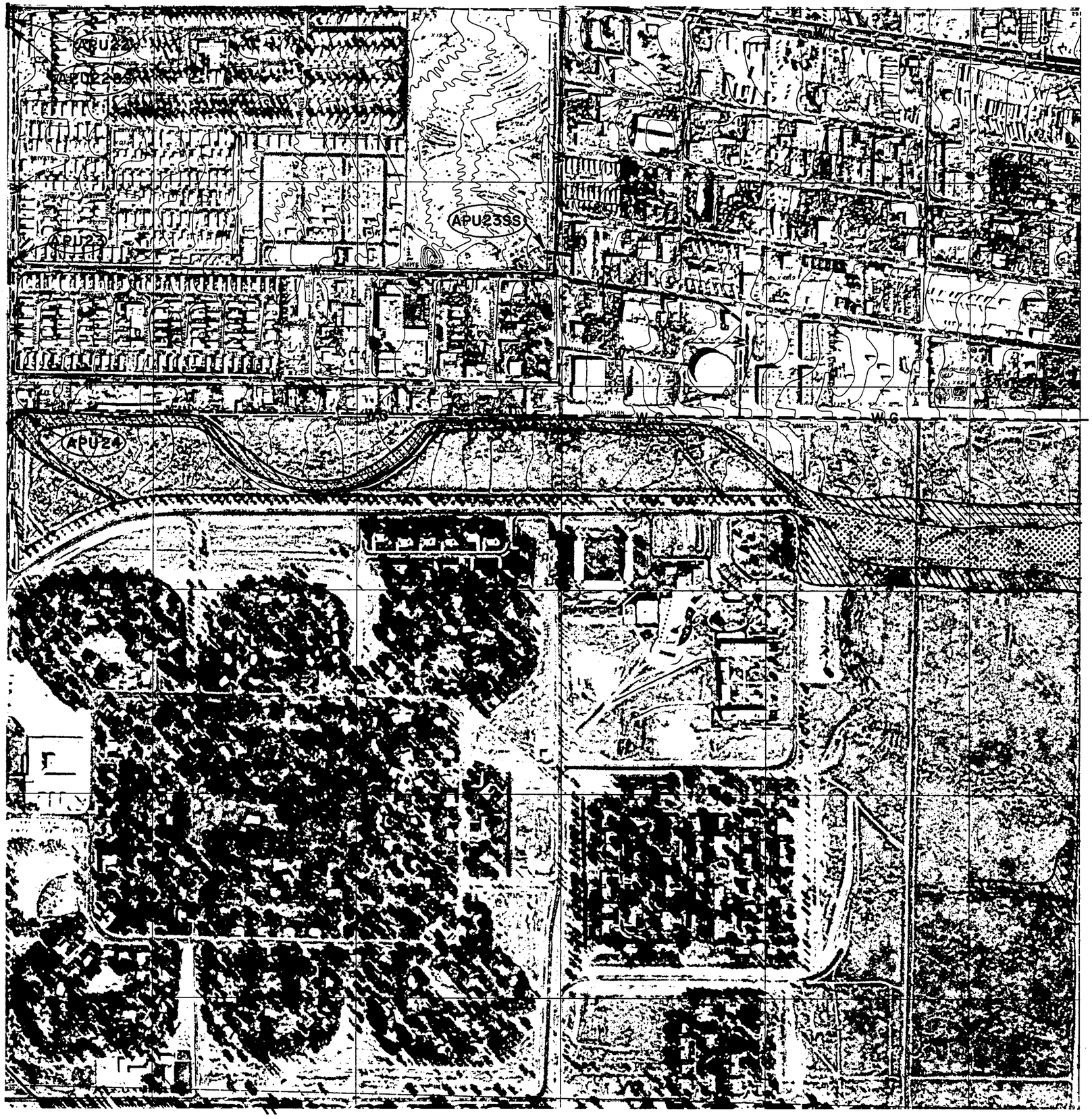

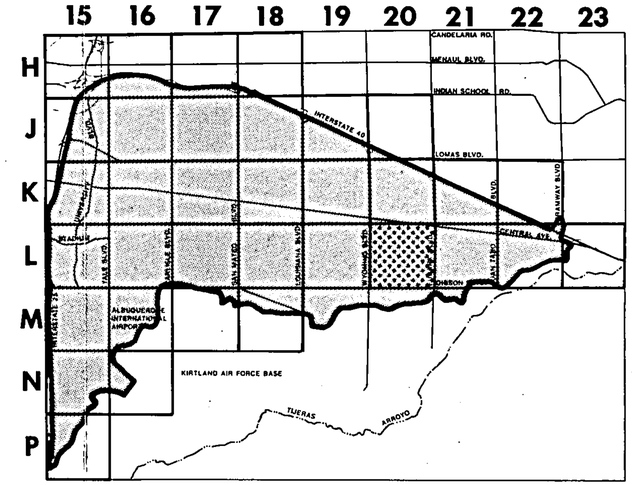

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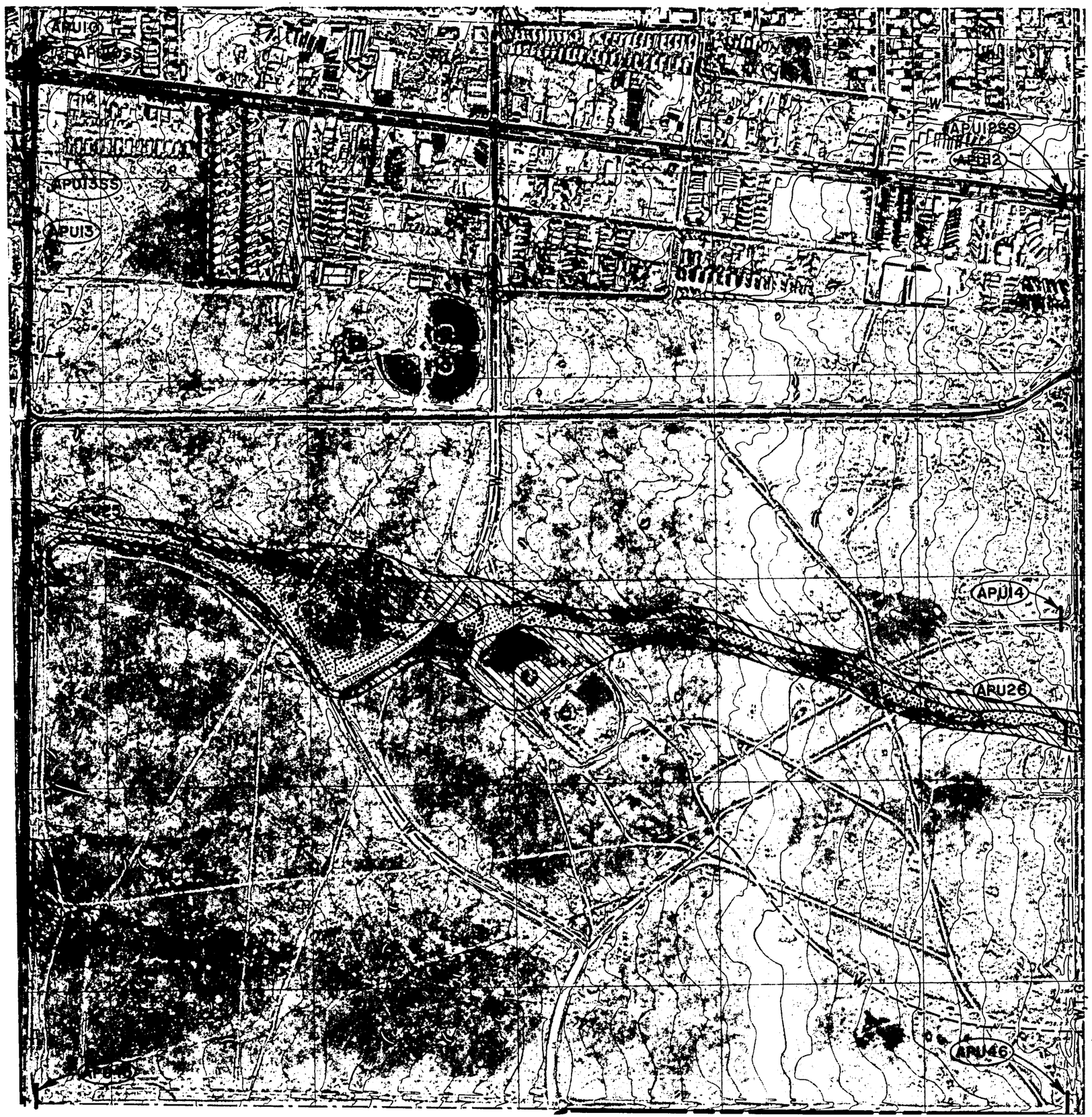

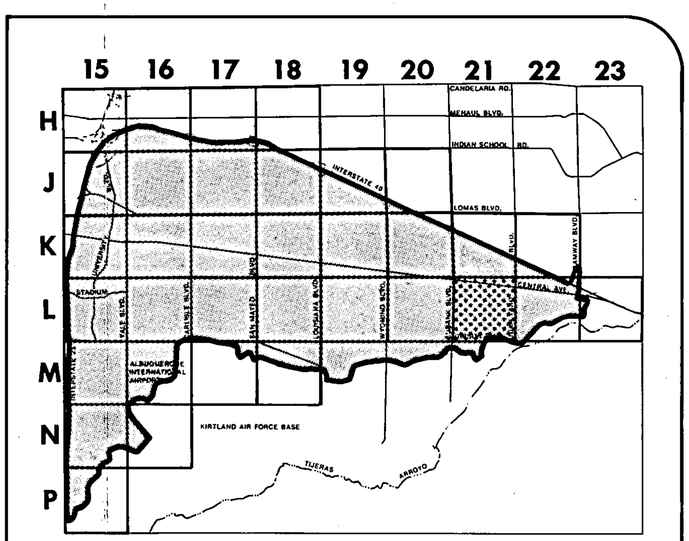

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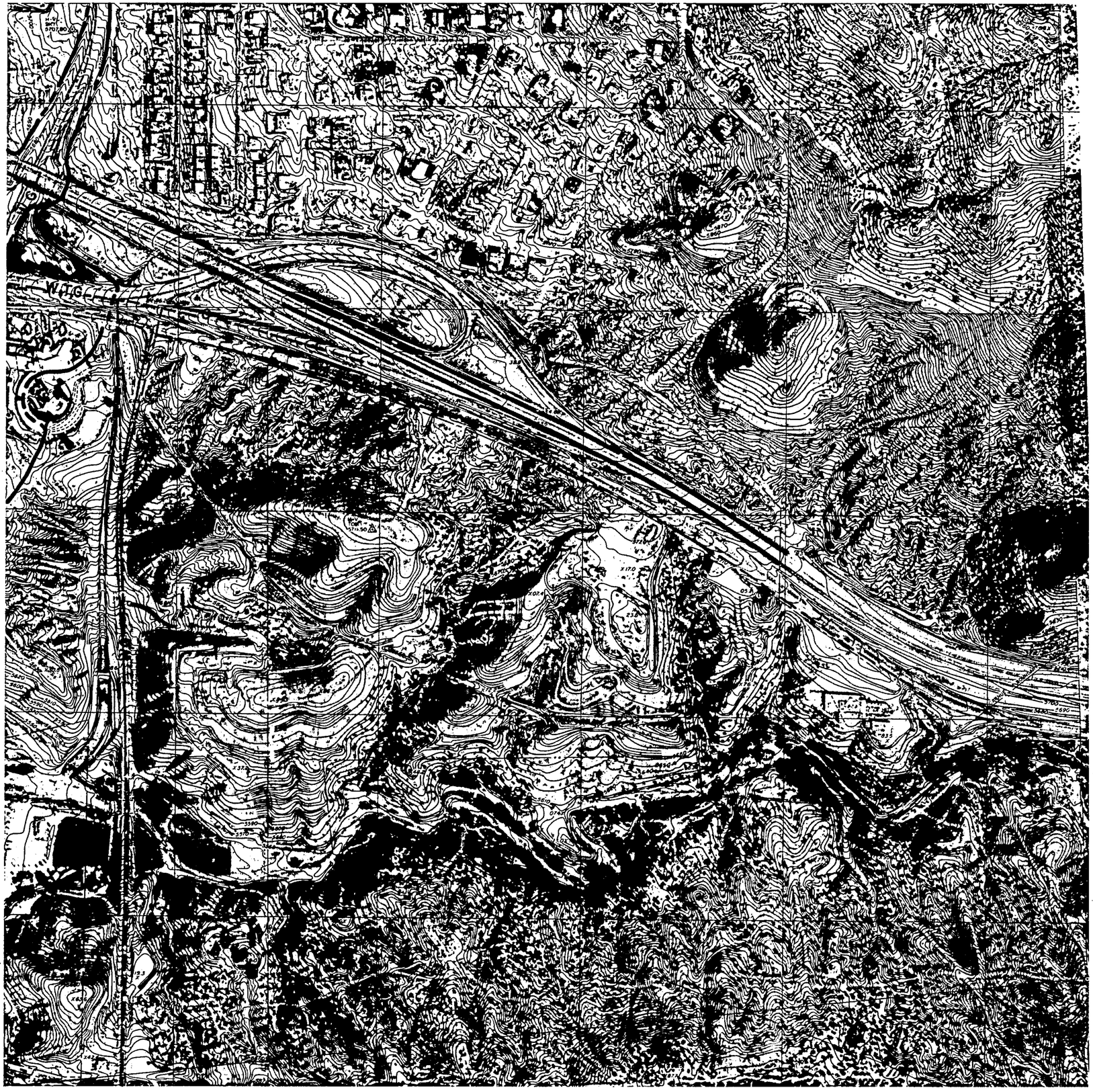
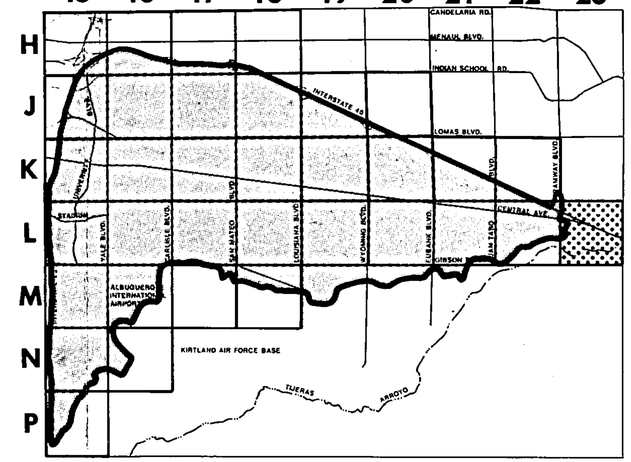
BOHANNAN HUSTON INC.

L-21

APu1SS
APU1
W,T,G
W,S,T,G
W,T,G
W,S
S
S
S
W
W
S
S
S





MAP KEY

LEGEND

- STUDY AREA BOUNDARY
- APC8 — BASIN ANALYSIS POINT
- 10 YEAR FLOOD HAZARD AREA
- 100 YEAR FLOOD HAZARD AREA
- —36— PROPOSED STORM SEWER

EXISTING UTILITIES

- ---S--- SANITARY SEWER, SIZE 10" OR LARGER
- ---G--- GAS LINE, SIZE 6" OR LARGER
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- ---T--- UNDERGROUND TELEPHONE LINE

NOTE: Existing utilities are shown on arterial and collector streets and where drainage facilities are proposed

0 500 1000 1500 2000

SCALE IN FEET

1"=500'

Orthophotography and topography by Bohannan Huston, Inc. from aerial photograph dated 10-8-80.

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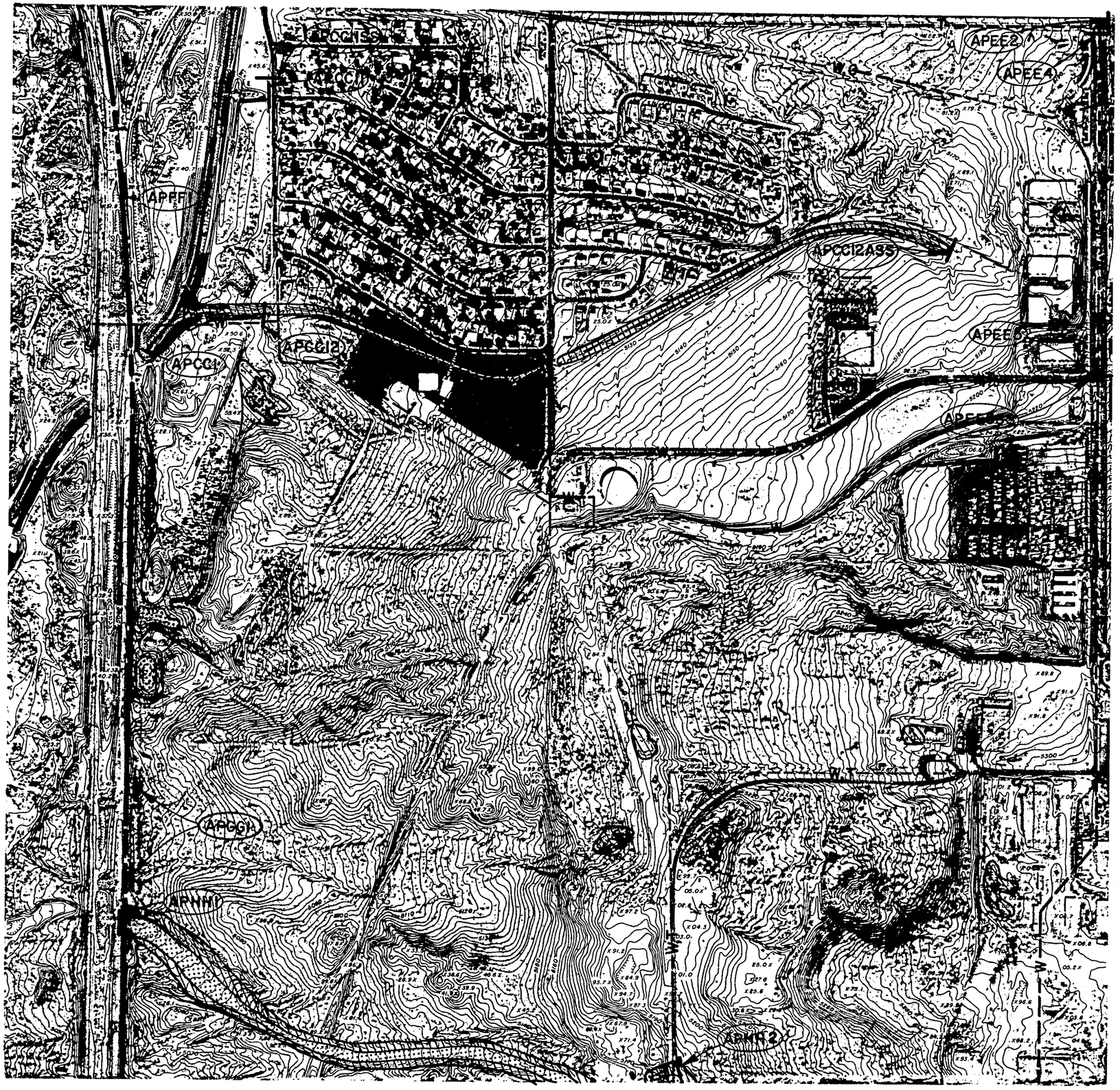
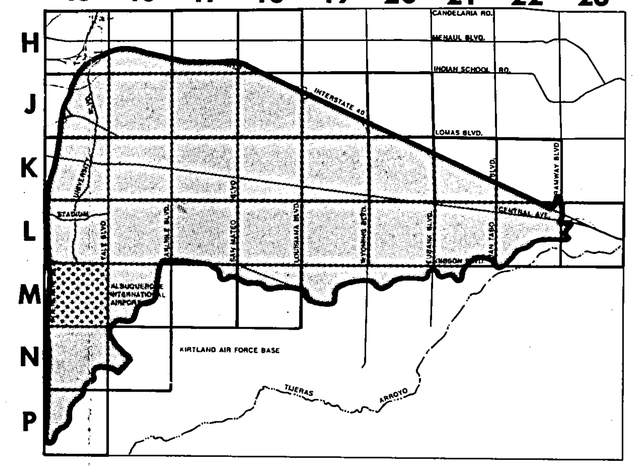
L-23





MAP KEY

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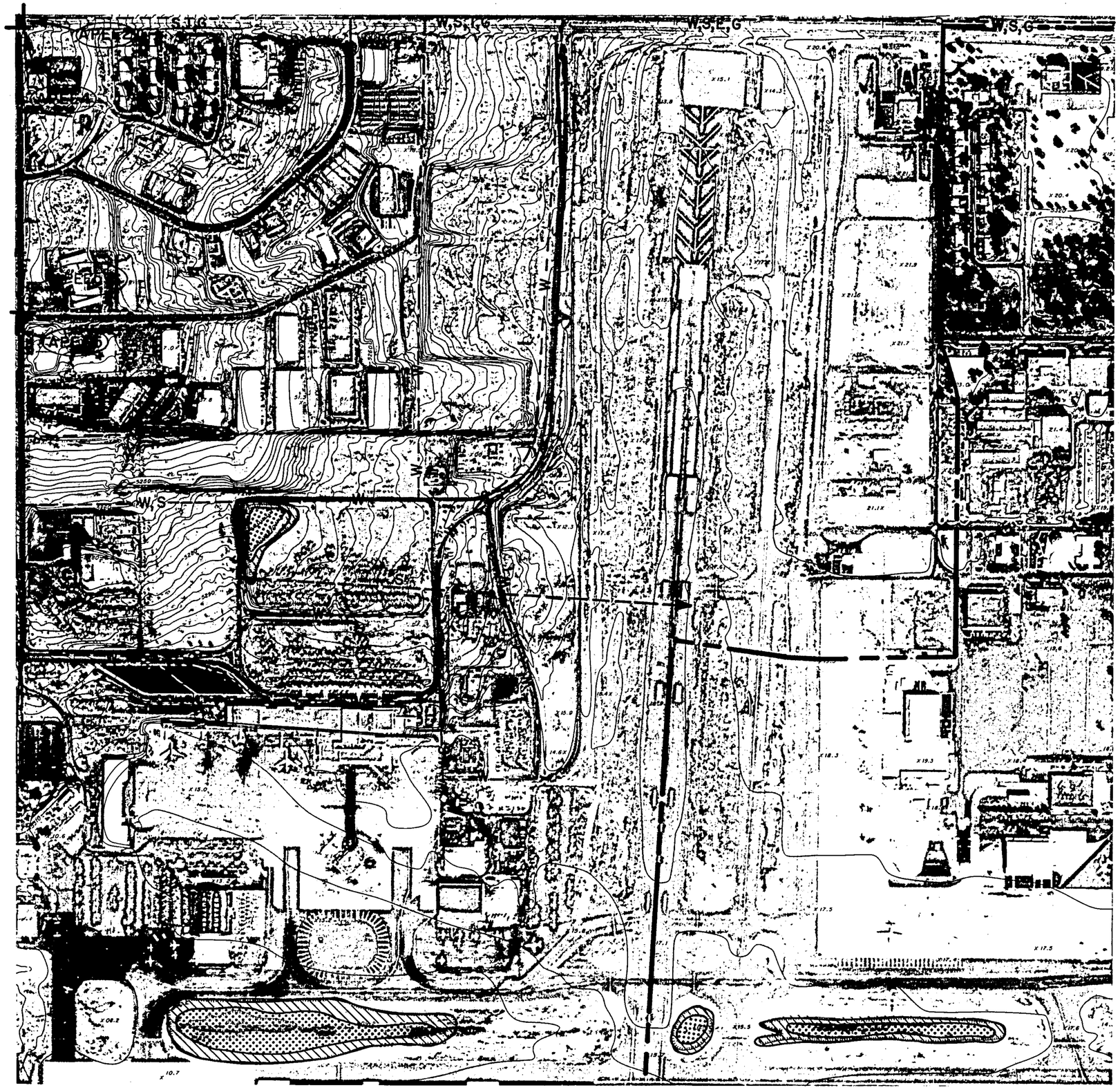
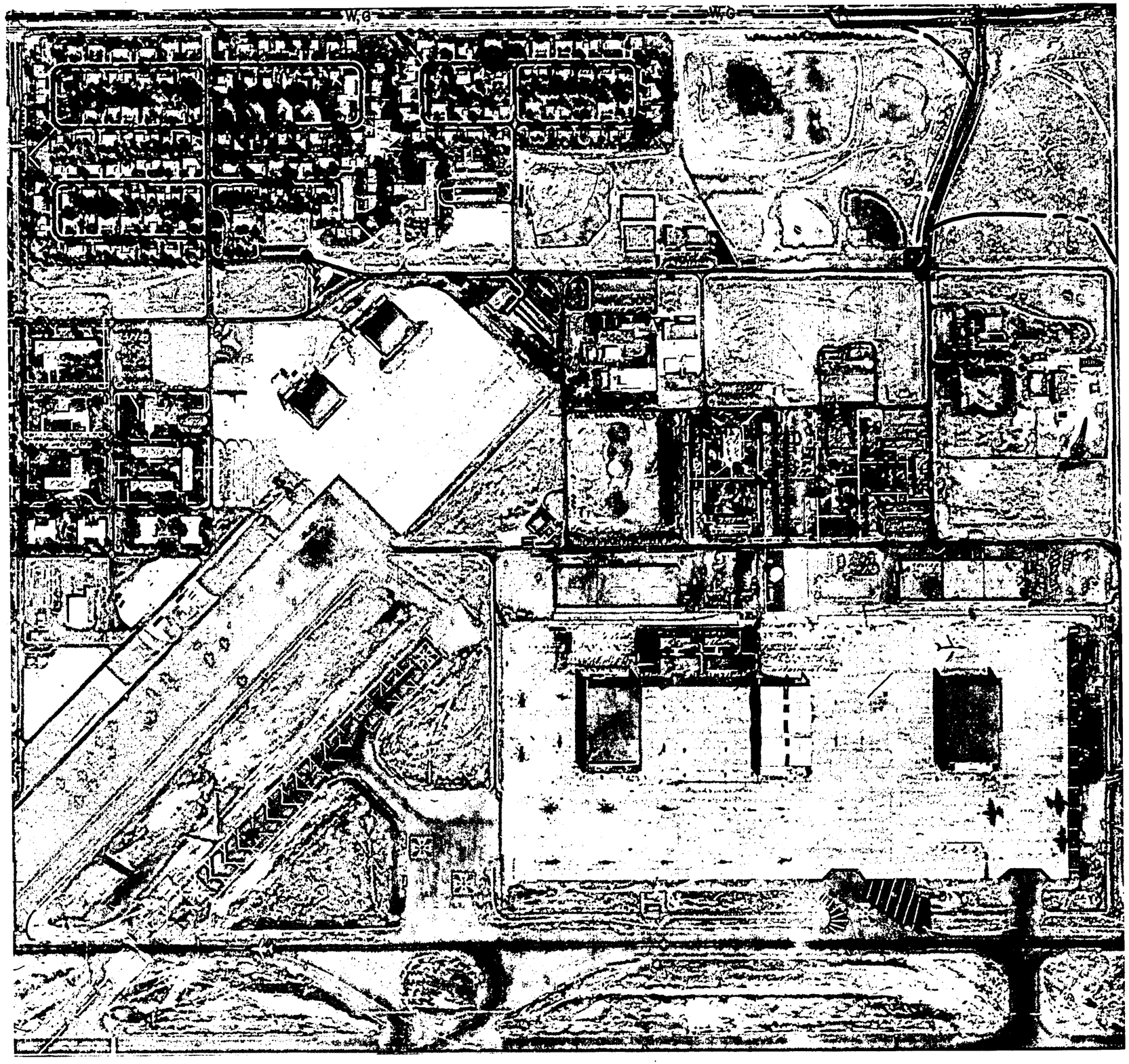
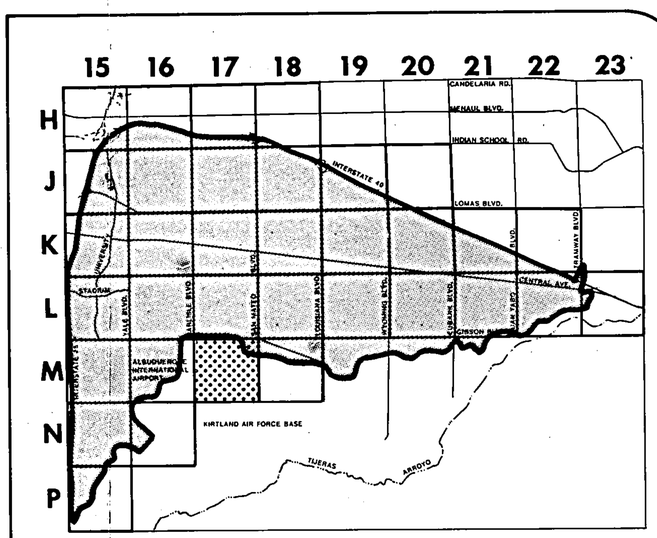
M-15


W,S,G
W,S,G
W,S,G
W,S
W
W
W





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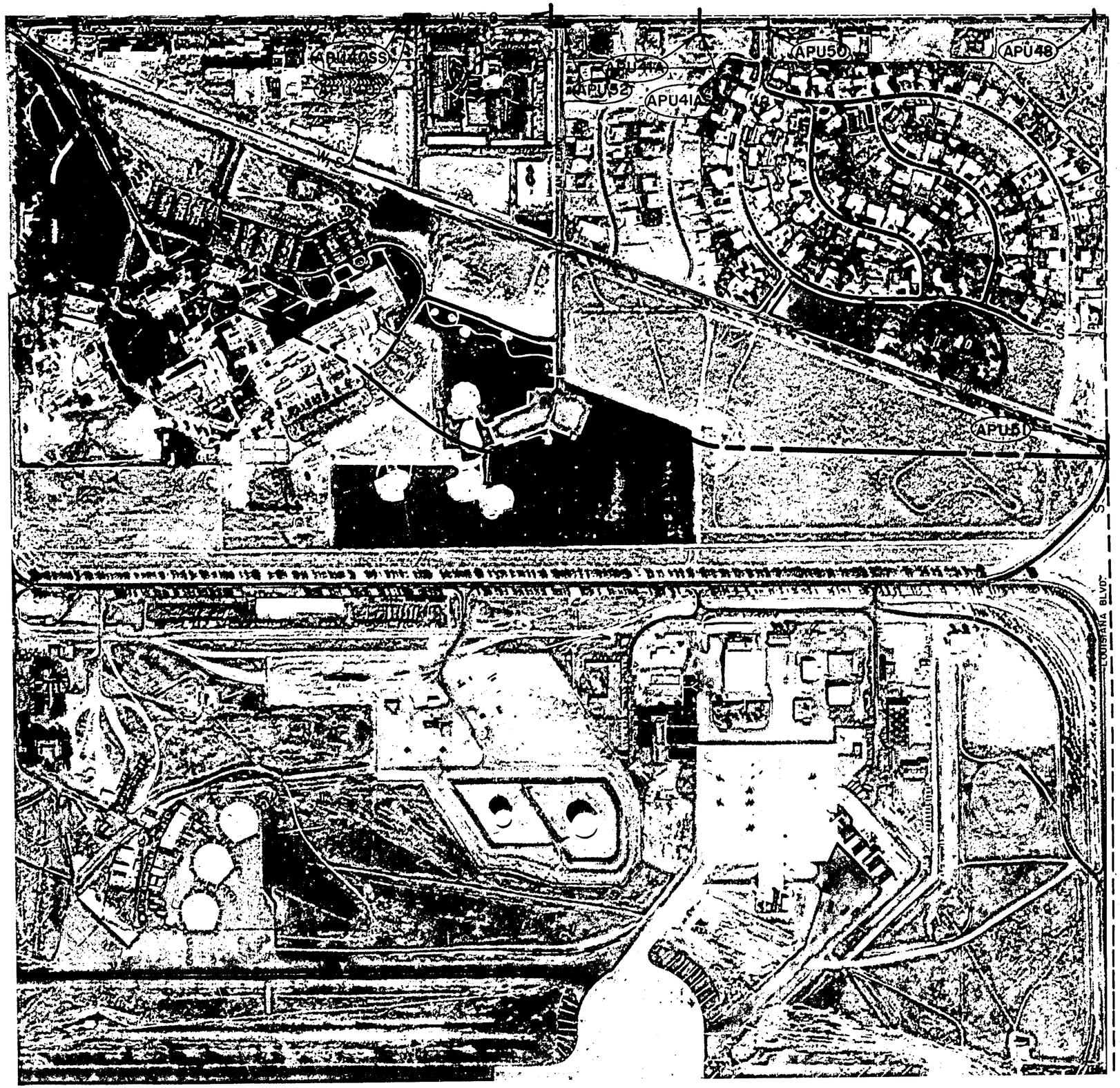
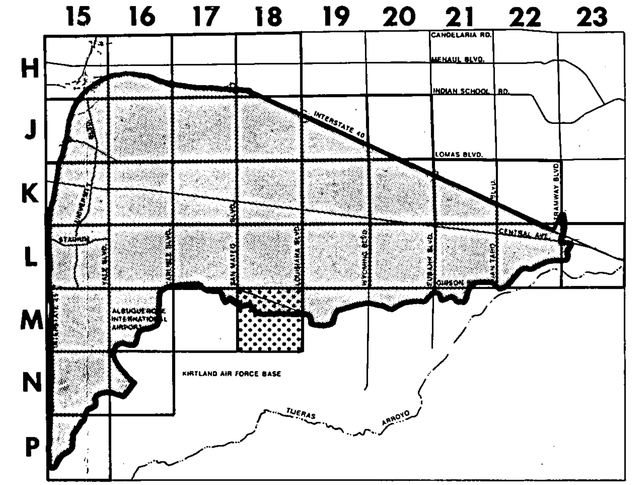
M-17





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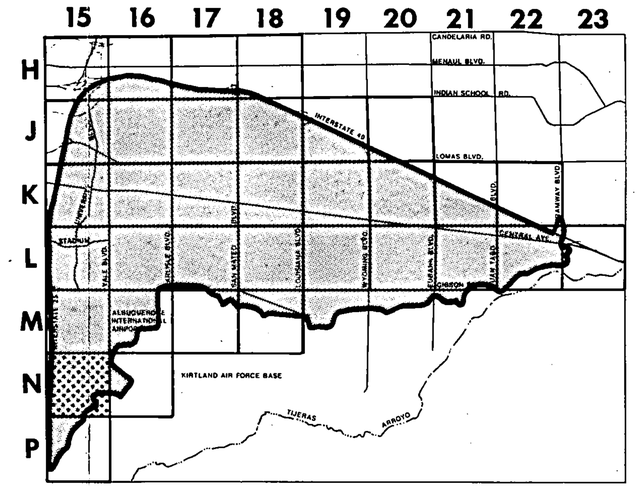
DECEMBER 1986

M-18



MAP KEY

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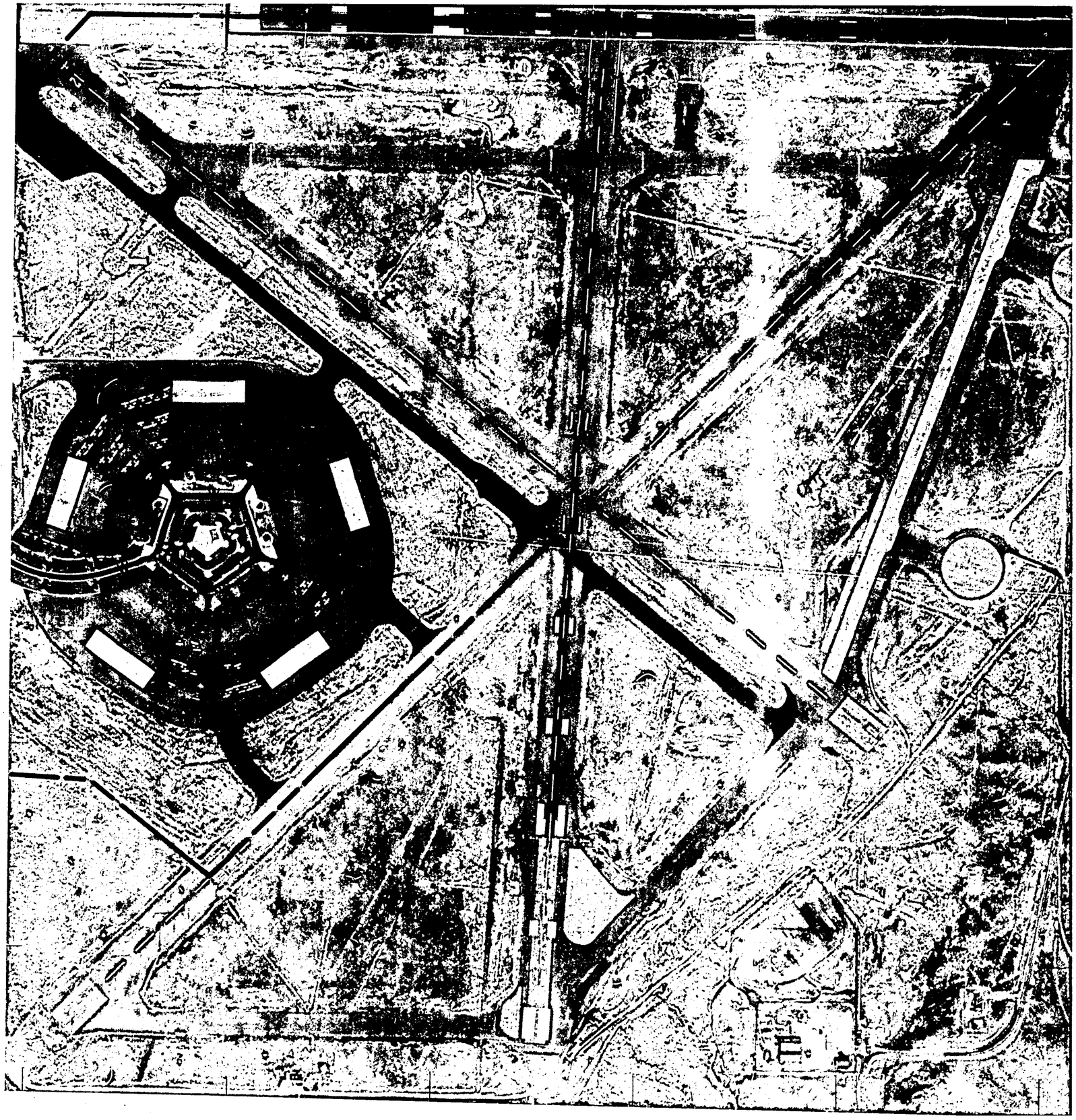
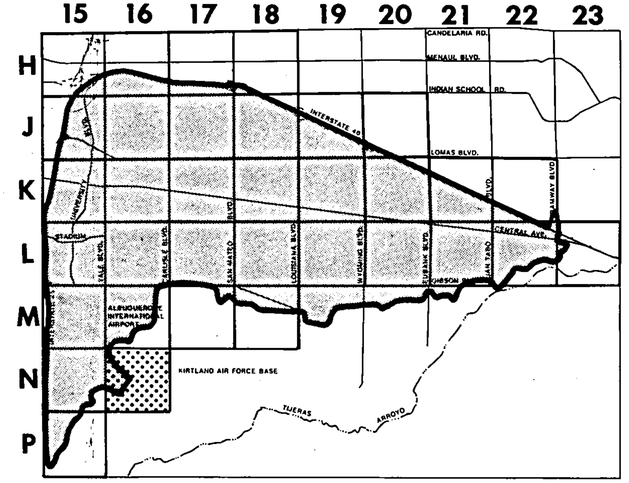
N-15





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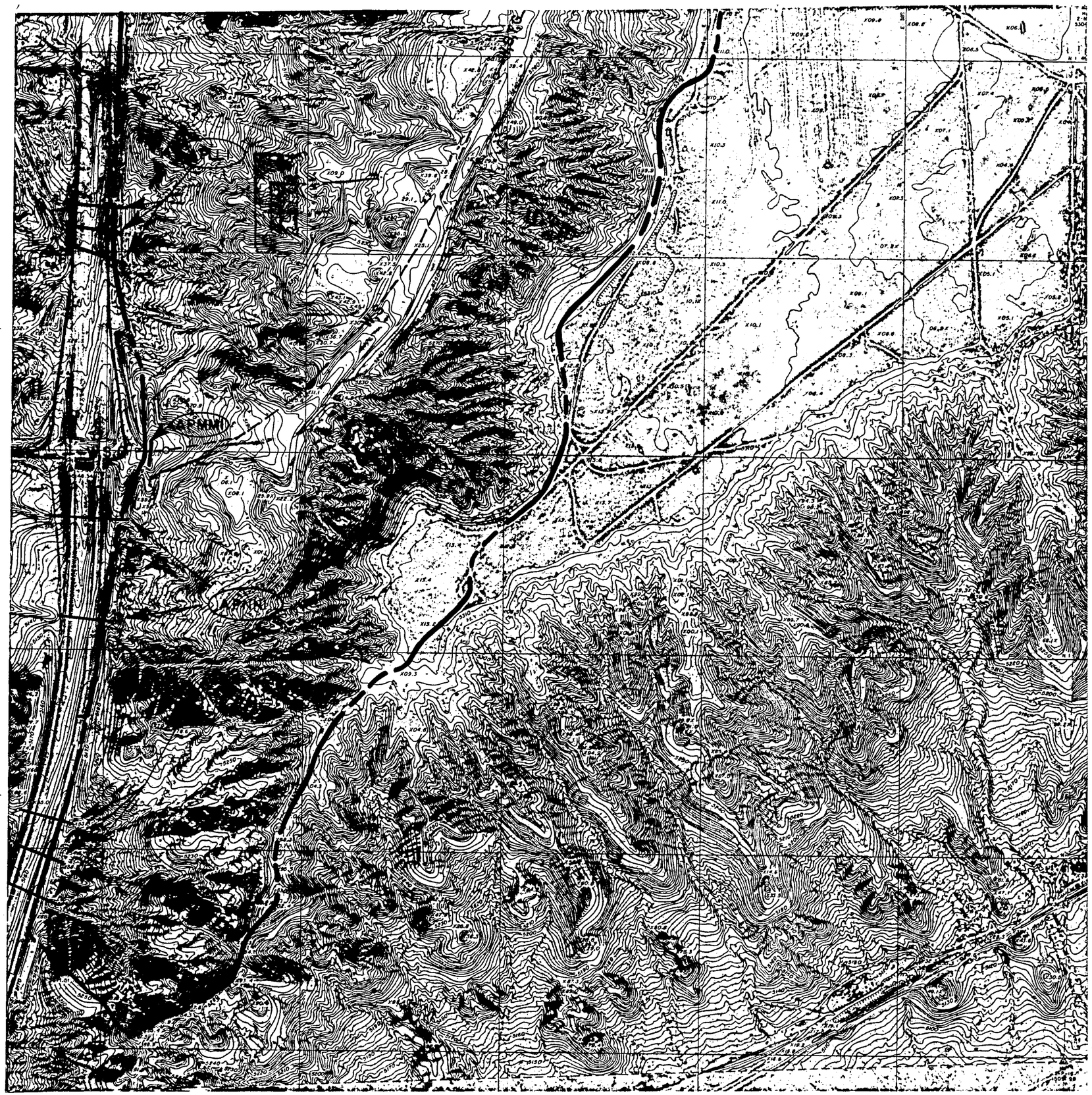
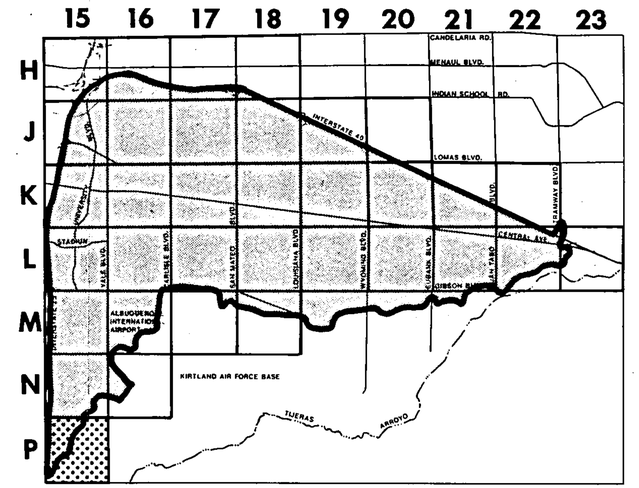
DECEMBER 1986

N-16





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Orthophotography and topography by Bohannan Huston, Inc. from aerial photograph dated 10-8-80.

DECEMBER 1986

BOHANNAN HUSTON INC.

P-15

HYDRAULIC PERFORMANCE OF EXISTING STRUCTURES AT ANALYSIS POINTS

Analysis Point	Tab II Page No.	Analysis Point	Tab II Page No.
APA1	16	APCC3	24
APA1SS	16	APCC5	24
APA2	16	APCC5SS	23
APA3	16	APCC5SS-OL	23
APA3SS	16	APCC6	23
APA4	16	APCC6SS	23
APA4SS	16	APCC7	23
APA5	16	APCC8	23
APA5SS	16	APCC8SS	23
APAA1	2	APCC9	23
APB1	17	APCC9SS	23
APBB1	23	APCC10	23
APBB2-1	22	APCC11	24
APBB2-2	22	APCC11SS	24
APBB3	22	APCC12A	24
APBB3SS-48	22	APD1	16
APBB3SS-54	22	APD1A	16
APBB4	22	APD1ASS	16
APBB4SS-36	22	APDD1	24
APBB4SS-48	22	APE1	1
APBB4-1	22	APEE1	24
APBB5	22	APEE1SS	24
APBB5SS	22	APEE2	24
APBB6	22	APEE2SS	24
APBB6SS	21	APEE3 Not Used	
APBB7	21	APEE4	24
APBB7SS	21	APEE5	24
APBB8	23	APEE5SS	24
APBB8SS-1	22	APF1	17
APBB8SS-2	22	APF2	17
APBB8-1	22	APF2ASS	17
APBB9	21	APF2BSS	17
APBB9SS	21	APF3	17
APBB10	21	APF4	17
APBB10SS	21	APF5	17
APBB11	21	APF6	17
APBB12	21	APF6A	17
APBB12SS	21	APF6ASS	17
APC1	1	APF7	17
APCC1	24	APF7SS48	17
APCC2	24	APF7SS60	17
APCC2SS1-SS	23	APFF1	2
APCC2SS1-OL	23	APG1	19

HYDRAULIC PERFORMANCE OF EXISTING STRUCTURES AT ANALYSIS POINTS

Analysis Point	Tab II Page No.	Analysis Point	Tab II Page No.
APG1SS	19	APM1A	7
APGG1	1	APM1B	6
APH1	19	APM1C	6
APH1SS	19	APM1D	6
APHH1	1	APM1E	6
APHH2	1	APM1F	6
APJ1	1	APM1G	5
APJJ1	1	APM2	7
APK1	19	APM2A	5
APK1SS	19	APM3	5
APK2	19	APM3SS	5
APK2SS	19	APM3ASS	5
APK3	19	APM4	5
APK3SS	18	APM4SS	5
APK4	18	APM5	5
APK4A	19	APM5SS48	5
APK4ASS	19	APM5SS84	5
APK5	18	APM6	5
APK5SS	18	APM6SS	4
APK6	18	APM7	4
APK6SS	18	APM7SS	4
APK7	18	APM7A	4
APK7SS	18	APM7ASS	4
APK8	18	APM8	4
APK8SS	18	APM8SS	4
APK9	18	APM9	4
APK9SS	18	APM9SS	4
APKK1	1	APM10	3
APKK2	1	APM10SS	3
APL1	20	APM10A	3
APL2	20	APM10ASS	3
APL2SS	20	APM11	3
APL3	20	APM11SS	3
APL3SS	20	APM12	3
APL4	20	APM12SS	2
APL4SS	20	APM13	2
APL4A	20	APM13SS	2
APL4ASS	20	APM14	2
APL5	20	APM14SS	2
APL5SS	19	APM15	2
APL6	19	APM15SS	2
APLL1	2	APM16	3
APM1	7	APM17	4

HYDRAULIC PERFORMANCE OF EXISTING STRUCTURES AT ANALYSIS POINTS

Analysis Point	Tab II Page No.	Analysis Point	Tab II Page No.
APM17SS	4	APU7	12
APM18	3	APU7SS	11
APM19	3	APU7A	11
APM20	4	APU7ASS	11
APM21	4	APU8	11
APM21SS	3	APU8SS	11
APM22	6	APU9	11
APM23	6	APU9SS	11
APM24	6	APU10	11
APM25	6	APU10SS	11
APM26	6	APU11	10
APM27	5	APU11SS	10
APM28	5	APU12	10
APM29	4	APU12SS	10
APMM1	1	APU13	11
APN1	1	APU13SS	11
APNN1	2	APU14	10
APP1	1	APU15	13
APQ1	1	APU15SS	13
APR1	1	APU16	13
APS1	20	APU16SS	13
APT1	2	APU16ASS	9
APU1	14	APU17	13
APU1SS	14	APU17A	13
APU1A	14	APU17ASS	12
APU1ASS	14	APU18	12
APU2	14	APU18SS	12
APU2SS	14	APU19	12
APU2A	14	APU19SS	12
APU2ASS	14	APU20	11
APU2B	14	APU20SS	11
APU2C	14	APU21	11
APU2CSS	14	APU21SS	11
APU3	14	APU22	11
APU3SS	14	APU22SS	11
APU4	13	APU23	10
APU4SS	13	APU23SS	10
APU5	13	APU24	10
APU5SS	13	APU25	10
APU5A	12	APU26	10
APU5ASS	12	APU27	13
APU6	12	APU27SS	13
APU6SS	12	APU28	12

HYDRAULIC PERFORMANCE OF EXISTING STRUCTURES AT ANALYSIS POINTS

Analysis Point	Tab II Page No.	Analysis Point	Tab II Page No.
APU28SS	12	APU48SS	7
APU29	12	APU49	7
APU29SS	12	APU50	8
APU30	12	APU51	8
APU31	10	APU52	8
APU31SS	10	APU53	13
APU32	9	APU53SS	13
APU32SS	9	APU54	13
APU33	9	APV1	15
APU33SS	9	APV1A	14
APU34	9	APV1ASS	15
APU34SS	9	APV2	15
APU34A	9	APV2SS	15
APU34ASS	9	APV3	15
APU35	9	APV3SS	15
APU35SS	9	APW1	16
APU36	9	APW1SS-1	16
APU36SS	9	APW1SS-2	16
APU37	9	APW1SS-3	16
APU37SS	8	APW2	15
APU38	8	APW3	15
APU38SS	8	APW3SS	15
APU39	8	APW4	15
APU39SS	8	APW4SS	15
APU40	8	APW4SS-OL	15
APU40SS	8	APW5	15
APU41	8	APW5SS	15
APU41A	8	APX1	15
APU41ASS	8	APX1SS	15
APU42 Not Used		APY1	21
APU43	7	APY1SS	20
APU43SS	7	APY2	20
APU44	7	APY2SS	20
APU45	7	APZ1	21
APU46	7	APZ1SS-1	21
APU47	7	APZ1SS-2	21
APU48	7		

ANALYSIS POINT ID NO.	LOCATION	ZONE ATLAS MAP NO.	SS DIAM.	Q10 (CFS)	D10	V10 (FPS)	Q100 (CFS)	D100	V100 (FPS)
			DRAINAGE SYSTEM C, E, J, P, N, Q, R, GG, HH, JJ, KK, MM						
APC1	Wyoming Blvd. @ I-40	J-20	1-30"	4	7"	4.8	8	10"	6.0
			2-42"	8/2	6"	4.6	20/2	10"	6.2
APE1	Pennsylvania Ave. @ I-40	J-19	24"	1	4"	3.2	5	8"	5.1
APJ1	Washington St. @ I-40	H-17	30"	13	12"	6.7	40	30"	7.2
APP1	University Blvd. @ I-40	H-15	--	5	.22'	2.2	12	.30'	2.8
APN1	Extension of Princeton Dr. @ I-40	H-16	6'x6'CBC	4	.14'	4.9	8	.25'	5.5
APQ1	Interchange of I-25 & I-40	H-15	24"	3	6"	4.5	6	9"	5.5
APR1	North of Indian School Rd. @ I-25	J-15	30"	12	13"	5.7	57	30"	6.1
APGG1	@ I-25	M-15	4-36"	10/4	4"	6.7	30/4	6"	9.0
APHH1	Extension of Viewpoint Rd.	M-15	8'x6' CBC	21	.3'	9	79	.7'	15.0
APHH2	University Blvd. @ Extension of Viewpoint Rd.	M-15	--	36	.16'	1.3'	92	1.1'	2.7
APJJ1	Railroad @ I-25	N-15	3-36"	3/3	2"	2.0	9/3	5"	4.0
APKK1	@ I-25	N-15	8'x6' CBC	30	.39'	9.6	104	.85'	15.1
APKK2	@ I-25	N-15	--	45	.79'	2.3	118	.48'	1.9
APMM1	Rio Bravo Blvd. @ I-25	P-15	30"	6	8"	5.3	15	14"	7.0
			36"	9	10"	6.1	15	13"	7.0

ANALYSIS POINT ID NO.	LOCATION	ZONE ATLAS MAP NO.	SS DIAM.	Q10 (CFS)	D10	V10 (FPS)	Q100 (CFS)	D100	V100 (FPS)
			DRAINAGE SYSTEM LL, FF, NN, T, AA, M						
APLL1	@ I-25 north of Rio Bravo Blvd.	N-15	4-36"	4/4	3"	2.9	10/4	5"	4.0
			42"	1	3"	2.8	5	7"	5.0
			4-48"	4/4	3"	2.7	10/4	5"	4.1
			60"	3	5"	4.1	8	8"	5.5
APFF1	Extension of Alamo Ave. @ I-25	M-15	6'x4'CBC	3	.15'	4.7	5	.2'	5.6
APNN1	@ I-25	P-15	3-30"	0	0	0	9/3	10"	6.0
			42"	1	3"	2.8	6	7"	5.0
			48"	1	3"	2.7	8	8"	5.5
APT1	Between Mountain Rd. & Lomas Blvd. @ I-25	J-15	30"	1	3"	6.4	2	4"	7.7
APAA1	@ I-25	K-15	15"	2	6"	4.1	6	15"	4.6
			30"	4	7"	4.9	11	11"	6.3
			2-60"	14/2	10"	6.3	40/2	12"	7.1
APM15SS	Copper Ave. @ Wyoming Blvd.	K-20	54"	92	30"	10.1	153	54"	9.6
APM15	Copper Ave. @ Wyoming Blvd.	K-20	--	0	0	0	52	.50'	3.9
APM14SS	Copper Ave. @ Pennsylvania St.	K-19	66"	135	36"	10.1	231	66"	9.2
APM14	Copper Ave. @ Pennsylvania St.	K-19	--	0	0	0	94	.87'	3.5
APM13SS	Dallas St. @ Quintin Pl.	K-19	72"	144	42"	8.5	225	72"	8.0
APM13	Dallas St. @ Grand Ave.	K-19	--	0	0	0	32	.44'	3.6
APM12SS	Marquette Ave. @ San Pablo St.	K-19	36"	12	10"	8.0	48	21"	11.8

ANALYSIS POINT ID NO.	LOCATION	ZONE ATLAS MAP NO.	SS DIAM.	DRAINAGE SYSTEM M Q10 (CFS)	D10	V10 (FPS)	Q100 (CFS)	D100	V100 (FPS)
APM12	Marquette Ave. @ San Pablo St.	K-19	--	0	0	0	0	0	0
APM11SS	Louisiana Blvd. @ Roma Ave. (extensioin of)	K-19	66"	62	25"	7.6	203	66"	8.7
APM11	Marquette Ave. @ Louisiana Blvd.	K-19	--	0	0	0	0	0	0
APM16	San Juan Rd. (extension of) @ Indiana (extension of)	K-18	48"	48	32"	5.5	66	48"	4.9
APM10ASS	San Juan Rd. (Extension of) @ Georgia St. (Extension of)	K-18	48"	48	32"	5.5	66	48"	4.9
APM10A	South of Lomas Blvd. between Florida St. & Georgia St.	K-18	--	0	0	0	0	0	0
APM10SS	Lomas Blvd. @ San Pedro Blvd.	K-18	54"	49	23"	7.7	77	30"	8.7
APM10	San Pedro Blvd. between Lomas Blvd. & Roma Ave.	K-18	--	0	0	0	0	0	0
APM19	Marquette Ave. @ San Pedro Blvd.	K-18	--	27	.42'	3.4	60	.58'	4.6
APM18	San Pedro Blvd. @ San Juan Rd.	K-18	--	11	.30'	2.6	25	.40'	3.5
APM21SS	San Mateo Blvd. @ Marquette Ave.	K-18	96"	230	40"	11.8	539	69"	14.3

ANALYSIS POINT ID NO.	LOCATION	ZONE ATLAS MAP NO.	SS DIAM.	DRAINAGE SYSTEM M Q10 (CFS)	D10	V10 (FPS)	Q100 (CFS)	D100	V100 (FPS)
APM21	San Mateo Blvd. @ Marquette Ave.	K-18	--	0	0	0	0	0	0
APM17SS	San Mateo Blvd. between Lomas Blvd. & Roma Ave.	K-18	102"	268	42"	12.2	633	73"	14.8
APM17	San Mateo Blvd. @ Roma Ave.	K-18	--	0	0	0	0	0	0
APM9SS	San Mateo Blvd. north of Lomas Blvd.	J-18	102"	271	42"	12.1	642	73"	14.8
APM9	San Mateo Blvd. @ Lomas Blvd.	J-18	--	0	0	0	0	0	0
APM20	Washington St. between Roma Ave. & Brockmont Ave.	K-17	--	30	.50'	30	67	.73'	3.8
APM29	Washington St. @ Courtney Ave.	K-17	--	7	.35'	1.0	15	.46'	1.4
APM8SS	Lomas Blvd. @ Washington St.	J-17	48"	28	18"	6.5	71	32"	8.2
APM8	Lomas Blvd. @ Washington St.	J-17	--	0	0	0	0	0	0
APM7ASS	Lomas Blvd. @ Montclair Dr.	K-17	30"	0	0	0	0	0	0
APM7A	Lomas Blvd. @ Montclair Dr.	K-17	--	0	0	0	0	0	0
APM7SS	Lomas Blvd. @ Tulane Dr.	J-16	30"	43	30"	8.6	43	30"	8.6
APM7	Lomas Blvd. @ Tulane Dr.	J-16	--	23	.26'	7.0	114	.45'	12.0
APM6SS	Summit Ave. @ Mountain Rd.	J-16	36"	60	36"	8.5	60	36"	8.5

ANALYSIS POINT ID NO.	LOCATION	ZONE ATLAS MAP NO.	SS DIAM.	DRAINAGE SYSTEM M Q10 (CFS)	D10	V10 (FPS)	Q100 (CFS)	D100	V100 (FPS)
APM6	Summit Ave. @ Mountain Rd.	J-16	--	14	22'	1.2	130	1.1'	38
APM5SS84	Wilson Pl. @ Girard Blvd.	J-16	84"	78	25"	7.8	202	44"	10.3
APM5	Constitution Ave. between Vassar Dr. & Girard Blvd.	J-16	--	0	0	0	0	0	0
APM5SS48	Constitution Ave. between Vassar Dr. & Girard Blvd.	J-16	48"	0	0	0	0	0	0
APM4SS	Princeton Dr. @ Vassar Dr.	J-16	48"	24	13"	8.4	58	22"	10.7
APM4	Princeton Dr. @ Vassar Dr.	J-16	--	0	0	0	0	0	0
APM3ASS	@ Indian School Rd.	J-16	84"	81	26"	8.1	210	44"	10.4
APM3SS	Princeton Dr. @ Indian School Rd.	J-16	48"	34	28"	4.1	48	48"	3.8
APM3	Princeton Dr. @ Indian School Rd.	J-16	--	0	0	0	35	.41'	4.5
APM2A	Girard Blvd. @ North Diversion Channel	H-16	84"	84	26"	8.0	217	45"	10.5
APM28	Yale Blvd. @ North Diversion Channel	J-16	--	17	.10'	1.3	31	.15'	2.0
APM1G	West of Yale Blvd. @ North Diversion Channel	J-15	--	233	2.0'	8.0	466	2.93'	10.0
APM27	Tucker St. (extension of) @ North Diversion Channel	J-15	--	4	.10'	0.7	13	.13'	1.0

ANALYSIS POINT ID NO.	LOCATION	ZONE ATLAS MAP NO.	SS DIAM.	DRAINAGE SYSTEM M Q10 (CFS)	D10	V10 (FPS)	Q100 (CFS)	D100	V100 (FPS)
APM1F	Constitution Ave. (extension of) @ North Diversion Channel	J-15	--	232	2.0'	8.0	470	2.94'	10.0
APM26	Constitution Ave. (extension of) @ North Diversion Channel	J-15	--	5	.10'	1.0	22	.15'	1.3
APM1E	Constitution Ave. (extension of) @ North Diversion Channel	J-15	--	234	2.0'	8.0	478	2.96'	10.1
APM25	Hannett Ave. (extension of) @ North Diversion Channel	J-15	--	1	.06'	0.6	4	.1'	.8
APM1D	Hannett Ave. (extension of) @ North Diversion Channel	J-15	--	233	2.0'	8.0	480	2.96'	10.1
APM24	Haines Ave. (extension of) @ North Diversion Channel	J-15	--	20	.32'	2.2	53	.4'	2.5
APM1C	Haines Ave. (extension of) @ North Diverison Channel	J-15	--	251	2.1'	8.4	528	3.1'	10.3
APM23	Cutter Ave. (extension of) @ North Diversion Channel	J-15	--	5	.10'	0.7	14	.13'	1.0
APM1B	Cutler Ave. (extension of) @ North Diversion Channel	J-15	--	251	2.7'	4.5	531	4.1'	5.6
APM22	Stanford Dr. @ Indian School Rd.	J-16	--	6	.21'	2.9	13	.28'	3.4

ANALYSIS POINT ID NO.	LOCATION	ZONE ATLAS MAP NO.	SS DIAM.	DRAINAGE SYSTEM M, U Q10 (CFS)	D10	V10 (FPS)	Q100 (CFS)	D100	V100 (FPS)
APM2	Apache Dr. @ North Diverison Channel	H-16	--	59	Pumped	Pumped	141	Pumped	Pumped
APM1A	Apache Dr. (extension of) @ North Diversion Channel	H-16	--	232	2.6'	4.4	552	4.2'	5.7
APM1	North Diversion Channel @ I-40	H-16	--	319	3.1'	4.9	773	5.0'	6.2
APU46	Juan Tabo Blvd. South of La Puerta Ave.	L-22	--	12	.31'	2.8	30	.41'	3.8
APU45	Eubank Blvd. @ "D" St.	L-21	--	7	.25'	2.3	34	.43'	3.9
APU44	Eubank Blvd. between La Brea Pl., & "D" St.	L-21	--	11	.31'	2.5	52	.5'	3.0
APU47	Pennsylvania St. north of "F" St.	M-19	--	43	--	--	116	.1'	2.0
APU43SS	North of intersection of Gibson Blvd. & San Pablo St.	L-19	30"	25	17"	9.0	31	19"	8.3
APU43	West of intersection of 49th St. & 45th Pl.	L-19	--	0	0	0	0	0	0
APU49	North of Perimeter Dr. & East of Louisiana blvd.	M-19	--	25	.46'	2.8	70	.73'	3.9
APU48SS	Louisiana Blvd. @ Gibson Blvd.	M-19	36"	37	18"	10.3	72	36"	11.0
APU48	Louisiana Blvd. @ Gibson Blvd.	M-19	--	0	0	0	33	.35'	5.7

ANALYSIS POINT ID NO.	LOCATION	ZONE ATLAS MAP NO.	SS DIAM.	DRAINAGE SYSTEM U Q10 (CFS)	D10	V10 (FPS)	Q100 (CFS)	D100	V100 (FPS)
APU51	Ridgecrest Dr. @ Louisiana Blvd.	M-18	--	18	.40'	2.5	45	.58'	3.4
APU50	Dakota St. south of Gibson Blvd.	M-18	--	23	.44'	2.6	55	.63'	3.6
APU41ASS	Gibson Blvd. @ California St.	L-18	48"	59	48"	5.1	59	48"	5.1
APU41A	Gibson Blvd. @ California St.	L-18	--	9	.2'	4.0	120	.53'	7.7
APU52	San Pedro Blvd. south of Gibson Blvd.	M-18	--	10	.23'	4.1	31	.35'	5.4
APU41	Gibson Blvd. @ San Pedro Blvd.	L-18	--	20	.28'	4.9	155	.59'	8.3
APU40SS	Gibson Blvd. @ Cardenas Dr.	L-18	48"	42	29"	5.1	60	48"	5.1
APU40	Gibson Blvd. @ Cardenas Dr.	L-18	--	0	0	0	140	.81'	3.8
APU39SS	California St. north of Eastern Ave.	L-18	48"	60	48"	5.1	60	48"	5.1
APU39	Eastern Ave. @ San Pedro Blvd.	L-18	--	5	.24'	2.0	13	.35'	2.7
APU38SS	California St. north of Ross Ave.	L-18	48"	60	48"	5.1	60	48"	5.1
APU38	Ross Ave. @ San Pedro Blvd.	L-18	--	14	34"	2.6	41	.50'	4.1
APU37SS	California St. @ Anderson Ave.	L-18	48"	60	48"	5.1	60	48"	5.1

ANALYSIS POINT ID NO.	LOCATION	ZONE ATLAS MAP NO.	SS DIAM.	DRAINAGE SYSTEM U Q10 (CFS)	D10	V10 (FPS)	Q100 (CFS)	D100	V100 (FPS)
APU37	Anderson Ave. @ San Pedro Blvd.	L-18	--	14	.32'	2.7	40	.47'	4.1
APU36SS	Ortiz Dr. @ Kathryn Ave.	L-18	60"	63	32"	6.0	115	60"	5.9
APU36	Ortiz Dr. @ Kathryn Ave.	L-18	--	0	0	0	195	.95'	2.7
APU35SS	Kathryn Ave. @ San Pedro Blvd.	L-18	54"	69	25"	9.6	91	30"	10.3
APU35	Kathryn Ave. @ Palomas Dr.	L-18	--	0	0	0	0	0	0
APU34ASS	Kathryn Ave. @ Palomas Dr.	L-18	54"	76	31"	8.3	109	39"	9.0
APU34A	Kathryn Ave. @ Palomas Dr.	L-18	--	0	0	0	0	0	0
APU34SS	Kathryn Ave. @ San Mateo Blvd.	L-18	66"	72	28"	7.6	205	66"	8.8
APU34	Kathryn Ave. @ San Mateo Blvd.	L-18	--	0	0	0	128	.81'	5.1
APU32SS	San Mateo Blvd. between Trumbull Ave. & Bell Ave.	L-18	66"	91	34"	7.4	159	49"	8.6
APU32	San Mateo Blvd. @ Trumbull Ave.	L-18	--	0	0	0	0	0	0
APU33SS	Kathryn Ave. @ Quincy St.	L-17	60"	100	29"	10.7	225	60"	11.6
APU33	Kathryn Ave. @ Quincy St.	L-17	--	0	0	0	160	.92'	6.0
APU16ASS	Graceland Dr. @ Pershing Ave.	L-17	36"	67	27"	11.7	72	36"	10.3

ANALYSIS POINT ID NO.	LOCATION	ZONE ATLAS MAP NO.	SS DIAM.	DRAINAGE SYSTEM U Q10 (CFS)	D10	V10 (FPS)	Q100 (CFS)	D100	V100 (FPS)
APU31SS	Madison St. @ Zuni Rd.	K-17	78"	162	33"	12.2	428	63"	15.0
APU31	Madison St. @ Zuni Rd.	K-17	--	0	0	0	0	0	0
APU26	Juan Tabo Blvd. @ Singing Arrow Ave.	L-22	--	15	.32'	1.6	43	.47'	2.1
APU25	Eubank Blvd. @ La Entrada Ave.	L-21	--	18	.74'	1.0	82	1.3'	1.6
APU24	Wyoming Blvd. @ Southern Ave.	L-20	--	10	.29'	2.5	47	.51'	4.5
APU23SS	Park Ave. @ Moon St.	L-20	36"	23	25"	4.6	29	36"	4.4
APU23	Trumbull Ave. @ Wyoming Blvd.	L-20	--	0	0	0	31	.41'	4.0
APU11SS	Juan Tabo Blvd. @ Central Ave.	L-22	54"	35	14"	10.3	100	25"	13.9
APU11	Juan Tabo Blvd. @ Central Ave.	L-22	--	0	0	0	0	0	0
APU12SS	Juan Tabo Blvd. @ Central Ave.	L-21	60"	38	17"	8.6	109	29"	11.3
APU12	Juan Tabo Blvd. @ Central Ave.	L-22	--	0	0	0	0	0	0
APU14	Juan Tabo Blvd. between Southern Ave. & La Caridad Ave.	L-22	--	12	.30'	2.8	30	.40'	3.9

ANALYSIS POINT ID NO.	LOCATION	ZONE ATLAS MAP NO.	SS DIAM.	DRAINAGE SYSTEM U Q10 (CFS)	D10	V10 (FPS)	Q100 (CFS)	D100	V100 (FPS)
APU13SS	Eubank Blvd. @ Cochiti Rd.	L-21	54"	33	18"	6.8	114	39"	9.4
APU13	Eubank Blvd. @ Acoma Rd.	L-21	--	0	0	0	0	0	0
APU10SS	Eubank Blvd. @ Central Ave.	L-21	60"	76	29"	8.0	160	60"	8.1 ←
APU10	Eubank Blvd. @ Central Ave.	L-21	--	0	0	0	82	.59'	4.6 ←
APU9SS	Central Ave. @ Moon St.	K-20	42"	42	21"	9.1	87	42"	9.7
APU9	Central Ave. @ Moon St.	K-20	--	0	0	0	39	.44'	3.9
APU8SS	Central Ave. @ Chenault St.	K-20	48"	6	8"	5.1	30	17"	8.0
APU8	Central Ave. @ Wyoming Blvd.	K-20	--	0	0	0	0	0	0
APU22SS	Zuni Rd. @ Wyoming Blvd.	L-20	54"	62	23"	9.9	136	54"	8.3
APU22	Zuni Rd. @ Wyoming Blvd.	L-20	--	0	0	0	14	.32'	2.8
APU21SS	Zuni Rd. @ Pennsylvania St.	L-19	66"	79	28"	8.5	195	50"	10.2
APU21	Zuni Rd. @ Pennsylvania St.	L-19	--	0	0	0	0	0	0
APU20SS	Alcazar St. north of Zuni Rd.	K-19	60"	60	27"	7.3	143	60"	7.8
APU20	Zuni Rd. @ Louisiana Blvd.	L-19	--	0	0	0	28	.5'	2.2
APU7ASS	Dallas St. @ Chico Rd.	K-19	66"	90	29"	8.8	221	66"	9.3
APU7A	Central Ave. @ Pennsylvania St.	K-19	--	0	0	0	3	.16'	1.9
APU7SS	Central Ave. @ Alcazar St.	K-19	84"	71	26"	6.8	177	44"	8.8

ANALYSIS POINT ID NO.	LOCATION	ZONE ATLAS MAP NO.	SS DIAM.	DRAINAGE SYSTEM U Q10 (CFS)	D10	V10 (FPS)	Q100 (CFS)	D100	V100 (FPS)
APU7	Central Ave. @ Louisiana Blvd.	K-19	--	0	0	0	0	0	0
APU30	Southern Ave. @ Louisiana Blvd.	L-19	--	4	.21'	1.8	14	.35'	2.6
APU29SS	Bell Ave. @ Arizona St.	L-18	48"	42	23"	7.1	58	48"	4.4
APU29	Bell Ave. @ San Pedro Blvd.	L-18	--	0	0	0	38	.60'	2.8
APU19SS	San Pedro Blvd. between Cochiti Rd. & Acoma Rd.	K-18	54"	69	28"	8.6	133	54"	9.0
APU19	San Pedro Blvd. between Cochiti Rd. & Acoma Rd.	K-18	--	0	0	0	11	.39'	1.5
APU28SS	Zuni Rd. @ San Mateo Blvd.	K-18	84"	108	35"	7.9	221	50"	9.1
APU28	Zuni Rd. @ San Mateo Blvd.	K-18	--	0	0	0	0	0	0
APU18SS	San Mateo Blvd. @ Coal Ave.	K-18	96"	176	36"	10.0	389	58"	12.3
APU18	San Mateo Blvd. @ Coal Ave.	K-17	--	0	0	0	0	0	0
APU6SS	Central Ave. @ San Mateo Blvd.	K-18	96"	185	37"	10.1	414	61"	12.5
APU6	Central Ave. @ San Mateo	K-18	--	0	0	0	0	0	0
APU5ASS	Madison St. @ Silver Ave.	K-17	36"	8	10"	4.9	18	15"	6.3
APU5A	Central Ave. @ Jefferson St.	K-17	--	0	0	0	0	0	0
APU17ASS	Coal Ave. @ Jefferson St.	K-17	42"	108	28"	16.1	140	42"	14.9

ANALYSIS POINT ID NO.	LOCATION	ZONE ATLAS MAP NO.	SS DIAM.	DRAINAGE SYSTEM U Q10 (CFS)	D10	V10 (FPS)	Q100 (CFS)	D100	V100 (FPS)
APU17A	Silver Ave. @ Madison St.	K-17	--	0	0	0	0	0	0
APU27SS	Lead Ave. @ Washington St.	K-17	60"	112	28"	12.1	143	33"	13.0
APU27	Lead Ave. @ Washington St.	K-17	--	0	0	0	0	0	0
APU17	Silver Ave. @ Washington St.	K-17	--	9	.26'	2.3	21	.38'	3.1
APU16SS	Lead Ave. @ Solano Dr.	K-17	78"	141	37"	9.3	278	57"	10.7
APU16	Lead Ave. @ Montclaire Dr.	K-17	--	0	0	0	0	0	0
APU15SS	Central Ave. @ Solano Dr.	K-17	78"	145	37"	9.3	292	59"	10.8
APU15	Central Ave. @ Solano Dr.	K-17	--	0	0	0	0	0	0
APU5SS	Central Ave. @ Hermosa Dr.	K-17	72"	152	34"	11.9	314	53"	13.8
APU5	Central Ave. @ Hermosa Dr.	K-17	--	0	0	0	0	0	0
APU54	Domingo Ave. @ San Pedro Blvd.	K-18	--	17	.37'	2.6	52	.57'	4.1
APU53SS	Copper Ave. @ San Mateo Blvd.	K-18	96"	213	39"	11.4	493	68"	12.9
APU53	Copper Ave. @ San Mateo Blvd.	K-18	--	0	0	0	0	0	0
APU4SS	Campus Blvd. @ Monte Vista Blvd.	K-16	72"	192	37"	13.1	367	72"	13.0
APU4	Campus Blvd. @ Monte Vista Blvd.	K-16	--	0	0	0	44	.63'	2.9

ANALYSIS POINT ID NO.	LOCATION	ZONE ATLAS MAP NO.	SS DIAM.	DRAINAGE SYSTEM U, V Q10 (CFS)	D10	V10 (FPS)	Q100 (CFS)	D100	V100 (FPS)
APU3SS	Campus Blvd. @ Girard Blvd.	K-16	78"	207	41"	12.1	393	78"	11.9
APU3	Campus Blvd. @ Girard Blvd.	K-16	--	0	0	0	34	.32'	2.5
APU2CSS	Lomas Blvd. @ North Diversion Channel	J-16	78"	199	39"	11.9	393	78"	11.9
APU2C	Lomas Blvd. @ North Diversion Channel	J-16	--	0	0	0	5	.2'	2.0
APU2B	Lomas Blvd. @ Cornell Dr. (extension of)	J-16	--	0	0	0	5	.2'	2.0
APU2ASS	Lomas Blvd. @ Yale Blvd. (extension of)	J-15	42"	42	26"	6.9	62	42"	6.4
APU2A	Lomas Blvd. @ Yale Blvd. (extension of)	X-15	--	0	0	0	22	.38'	2.8
APU2SS	Lomas Blvd. @ University Blvd.	J-18	36"	0	0	0	22	15"	7.6
APU2	Lomas Blvd. @ University Ave.	J-15	--	0	0	0	0	0	0
APU1ASS	University Ave. north of Lomas	J-15	48"	34	19"	7.4	62	27"	8.7
APU1A	Lomas Blvd. @ Medical Arts Ave.	J-15		0	0	0	0	0	0
APU1SS	Lomas Blvd. @ I-25	J-15	30"	0	0	0	0	0	0
APU1	Lomas Blvd. @ I-25	J-15	--	0	0	0	0	0	0
APV1A	Grand Ave. @ Cedar St.	K-15	--	0	0	0	0	0	0

ANALYSIS POINT ID NO.	LOCATION	ZONE ATLAS MAP NO.	SS DIAM.	DRAINAGE SYSTEM V, X, W Q10 (CFS)	D10	V10 (FPS)	Q100 (CFS)	D100	V100 (FPS)
APV1	Grand Ave. @ I-25	K-15	--	20	.29'	.48	52	.42'	6.1
APV1ASS	Tijeras @ Cedar	K-15	60"	47	20"	8.2	128	35"	10.6
APV3SS	Univ. Blvd. @ Tijeras	K-15	12"	1	12"	1.4	1	12"	1.4
APV3	Univ. Blvd. @ Tijeras	K-15	--	13	--	--	36	--	--
APV2SS	Univ. Blvd. @ Grand	K-15	18"	3	18"	1.8	3	18"	1.8
APV2	Ash @ Univ. Blvd.	K-15	--	10	.31'	1.8	32	.49'	2.4
APX1SS	Lead Ave. @ Mulberry	K-15	96"	83	21"	10.2	240	36"	13.7
APX1	Lead Ave. @ I-25	K-15	--	0	0	0	0	0	0
APW5SS	Lead Ave. @ Girard Blvd.	K-16	27"	18	27"	5.1	18	27"	5.1
APW5	Lead Ave. @ Girard Blvd.	K-16	--	4	.2'	1.9	41	.47'	3.7
APW3SS	Girard @ Silver	K-16	24"	11	12"	6.9	21	24"	7.4
APW3	Silver Ave. @ Girard Blvd.	K-16	--	0	0	0	9	.25'	2.5
APW4SS	Yale Blvd. between Silver Ave. & Lead Ave.	K-15	30"	30	21"	7.2	35	30"	8.5
APW4SS-OL	Yale @ Silver	K-15	--	0	0	0	4.8	.2'	2.5
APW4	University @ Silver	K-15	--	20	.3'	4.3	67	.46'	5.7
APW2	Central Ave. @ University Blvd.	K-15	--	19	.29'	4.2	40	.38'	5.0

ANALYSIS POINT ID NO.	LOCATION	ZONE ATLAS MAP NO.	SS DIAM.	DRAINAGE SYSTEM W, A, D Q10 (CFS)	D10	V10 (FPS)	Q100 (CFS)	D100	V100 (FPS)
APW1SS-1	Gold Ave. @ University	K-15	30"	29	21"	8.1	35	30"	7.2
APW1SS-2	Central Ave. @ Cedar St.	K-15	96"	82	24"	8.4	233	41"	11.3
APW1SS-3	Tijeras Ave. @ I-25	K-15	30"	9	11"	5.8	23	19"	7.3
APW1	Central Ave. @ I-25	K-15	--	0	0	0	0	0	0
APA4SS	Buena Ventura Rd. @ Jane St.	K-21	48"	36	17"	9.5	88	27"	12
APA4	Juan Tabo Blvd. @ Buena Ventura Rd.	K-21	--	0	0	0	0	0	0
APA3SS	Chico Rd. @ Morris St.	K-21	54"	75	24"	11.0	177	43"	13.2
APA3	Chico Rd. @ Morris St.	K-21	--	0	0	0	0	0	0
APA5SS	Chico Rd. @ Eubank Blvd.	K-21	60"	159	60"	8.7	159	60"	8.7
APA5	Chico Rd. @ Eubank Blvd.	K-21	--	19	.36'	2.8	259	.96'	5.2
APA2	Morris @ I-40	K-21	--	17	.65'	3.5	37	1.0'	4.27
APA1SS	Copper Ave. @ Eubank Blvd.	K-21	60"	159	60"	8.7	159	60"	8.7
APA1	Eubank Blvd. @ I-40	K-21	--	20	.36'	2.8	272	.98'	5.3
APD1ASS	Utah St. (Extension of) @ I-40	J-19	48"	110	29"	14.3	167	48"	13.3
APD1A	Utah St. (Extension of) @ I-40	J-19	--	0	0	0	59	.56'	4.8
APD1	Summer Ave. (Extension of) @ I-40	J-19	66"	3	7"	2.2	59	32"	5.1

ANALYSIS POINT ID NO.	LOCATION	ZONE ATLAS MAP NO.	SS DIAM.	DRAINAGE SYSTEM F, B Q10 (CFS)	D10	V10 (FPS)	Q100 (CFS)	D100	V100 (FPS)
APF7SS60	Vermont St. @ Lomas Blvd.	J-19	60"	111	33"	10.0	188	60"	9.6
APF7SS48	Utah St. @ Lomas Blvd.	J-19	48"	0	0	0	40	2.1"	7.8
APF7	Lomas Blvd. @ Wyoming Blvd.	J-20	--	0	0	0	0	0	0
APF6ASS	Dallas St. @ Lomas Blvd.	J-19	72"	162	42"	9.5	251	72"	8.9
APF6A	Dallas St. @ Lomas Blvd.	J-19	--	0	0	0	40	.49'	2.9
APF6	Louisiana Blvd. @ Lomas Blvd.	J-19	--	13	.33'	2.5	57	.56'	3.8
APF5	Marble Ave. @ Pennsylvania St.	J-19	--	15	.35'	2.6	34	.46'	3.6
APF4	Mountain Rd. @ Pennsylvania St.	J-19	--	13	.33'	2.5	28	.42'	3.5
APF3	Louisiana Blvd. @ Constitution Ave.	J-19	--	24	.41'	2.6	56	.58'	3.2
APF2ASS	San Pablo St. @ I-40	J-19	30"	23	13"	10.7	51	22"	13.0
APF2BSS	Dallas St. @ I-40	J-19	72"	157	41"	9.4	251	72"	8.9
APF2	I-40 @ Louisiana Blvd. eastbound on ramp	J-19	--	27	.40'	2.7	63	.64'	3.4
APF1	Louisiana Blvd. @ I-40	J-19	60"	27	14"	7.7	63	39"	6.0
APB1	Lomas Blvd. @ I-40 on ramp	J-20	2-7'x9' CBCs	17	.5'	3.6	126	1.8'	5.0

ANALYSIS POINT ID NO.	LOCATION	ZONE ATLAS MAP NO.	SS DIAM.	DRAINAGE SYSTEM K Q10 (CFS)	D10	V10 (FPS)	Q100 (CFS)	D100	V100 (FPS)
APK9SS	Marble Ave. @ San Pedro Blvd.	J-18	54"	50	23"	7.7	82	31"	8.7
APK9	Marble Ave. @ San Pedro Blvd.	J-18	--	0	0	0	0	0	0
APK8SS	Mountain Rd. @ San Pedro Blvd.	J-18	54"	50	23"	7.7	86	32"	8.8
APK8	Mountain Rd. @ San Pedro Blvd.	J-18	--	0	0	0	0	0	0
APK7SS	Summer Ave. @ San Pedro Blvd.	J-18	54"	61	26"	8.1	125	54"	8.9
APK7	Summer Ave. @ San Pedro Blvd.	J-18	--	0	0	0	19	.37'	3
APK6SS	Constitution Ave. @ San Pedro Blvd.	J-18	54"	88	31"	9.6	144	54"	9.7
APK6	Constitution Ave. @ San Pedro Blvd.	J-18	--	0	0	0	51	.53'	4.0
APK5SS	Constitution Ave. @ San Mateo Blvd.	J-18	96"	339	51"	12.4	612	96"	13.1
APK5	Constitution Ave. @ San Mateo Blvd.	J-18	--	0	0	0	222	1.38'	4.23
APK4	Aspen @ San Mateo Blvd.	J-18	--	23	.4'	3.3	52	.53'	4.4
APK3SS	Sunningdale Ave. @ Washington St.	J-17	96"	359	43"	16.5	806	73"	19.7

ANALYSIS POINT ID NO.	LOCATION	ZONE ATLAS MAP NO.	SS DIAM.	DRAINAGE SYSTEM K, G, H; L Q10 (CFS)	D10	V10 (FPS)	Q100 (CFS)	D100	V100 (FPS)
APK3	Sunningdale Ave. @ Washington St.	J-17	--	0	0	0	0	0	0
APK2SS	Sunningdale Ave. @ Indian School Rd.	J-17	96"	371	44"	16.7	833	75"	20
APK2	Thorne Dr. @ Indian School Rd.	J-17	--	0	0	0	0	0	0
APK1SS	West of Morningside Dr. @ I-40	H-17	96"	371	44"	16.7	833	75"	20
APK4ASS	San Pedro Blvd. @ Aspen	J-18	54"	98	31"	10.9	166	54"	10.5
APK4A	San Pedro Blvd. @ Aspen	J-18	--	0	0	0	0	0	0
APK1	East of Morningside Dr. @ I-40	H-17	--	3	.1'	1.0	13	.2'	6.5
APG1SS	San Pedro Blvd. @ Haines Ave.	J-18	60"	119	31"	11.7	221	48"	13.1
APG1	San Pedro Blvd. @ Haines Ave.	J-18	48"	0	0	0	0	0	0
APH1SS	Off ramp (northbound) @ I-40	H-18	42"	68	28"	10.0	85	42"	9.1
APH1	Off ramp @ I-40	H-18	--	0	0	0	66	Floods Freeway	
APL6	Marble Ave. @ Washington St.	J-17	--	9	.28'	2.4	21	.37'	3.1
APL5SS	Mountain Ave. @ Washington St.	J-17	24"	19	24"	6.2	19	24"	6.2

ANALYSIS POINT ID NO.	LOCATION	ZONE ATLAS MAP NO.	SS DIAM.	DRAINAGE SYSTEM L, S, Y Q10 (CFS)	D10	V10 (FPS)	Q100 (CFS)	D100	V100 (FPS)
APL5	Mountain Ave. @ Washington St.	J-17	--	22	.38'	3.2	72	.61'	5.1
APL4ASS	Morningside Dr. @ Mackland Ave.	J-17	48"	27	16"	7.5	112	48"	10.3
APL4A	Morningside Dr. @ Mackland Ave.	J-17	--	0	0	0	69	.59'	5.0
APL4SS	La Hacienda Dr. @ Carlisle Blvd.	J-16	60"	109	31"	10.9	211	60"	10.8
APL4	La Hacienda Dr. @ Carlisle Blvd.	J-16	--	0	0	0	52	.58'	3.8
APL3SS	Rita Dr. @ Haines Ave.	J-16	60"	150	47"	9.3	159	60"	8.1
APL3	Rita Dr. @ Haines Ave.	J-16	--	0	0	0	178	.88'	7.4
APL2SS	Indian Playa Dr. @ Indian School Rd.	H-16	72"	158	34"	11.8	342	72"	12.1
APL2	Indian Playa Dr. @ Indian School Rd.	H-16	--	0	0	0	18	.24'	3.2
APL1	Drainage easement @ I-40	H-16	3-5'x8' CBC	182/3	12"	7.5	418/3	1.75'	10.0
APS1	Mountain Rd. @ I-25	J-15	3-120"	40/3	8"	5.6	103/3	13"	7.7
APY2SS	University Blvd. @ Lead Ave.	K-15	48"	43	18"	10.5	83	25"	12.4
APY2	Sycamore St. @ Coal Ave.	K-15	--	0	0	0	0	0	0
APY1SS	Mulberry St. @ Coal Ave.	K-15	96"	85	18"	12.7	245	31"	17.5

ANALYSIS POINT ID NO.	LOCATION	ZONE ATLAS MAP NO.	SS DIAM.	DRAINAGE SYSTEM Y, Z, BB Q10 (CFS)	D10	V10 (FPS)	Q100 (CFS)	D100	V100 (FPS)
APY1	Coal Ave. @ I-25	K-15	--	0	0	0	0	0	0
APZ1SS-1	South of Coal Ave. & East of I-25	K-15	108"	134	29"	9.5	353	50"	12.3
APZ1SS-2	South of Coal Ave. & East of I-25	K-15	48"	0	0	0	0	0	0
APZ1	South of Coal Ave. & East of I-25	K-15	--	0	0	0	0	0	0
APBB12SS	Smith Ave. @ Carlisle Blvd.	L-16	27"	12	20"	3.6	13	27"	3.6
APBB12	Smith Ave. @ Amherst Dr.	L-16	--	0	0	0	19	.32'	4.2
APBB11	Girard Ave. @ San Joaquin Ave.	L-16	--	8	.22'	3.3	33	.37'	5.0
APBB10SS	Girard Ave. @ Santa Clara Ave.	L-16	18"	11	18"	6.6	11	18"	6.6
APBB10	Girard Ave. @ Santa Clara Ave.	L-16	--	2	.14'	2.5	25	.33'	4.7
APBB9SS	Princeton Dr. @ Santa Clara Ave.	L-16	18"	1.2	4"	4	11	18"	7.2
APBB9	Yale Blvd. @ Santa Clara Ave.	L-16	--	13	.3'	2.7	75	.57'	4.2
APBB7SS	Girard Ave. @ Garfield Ave.	K-16	36"	61	22"	13.3	87	36"	13.3
APBB7	Girard Ave. @ Garfield Ave.	K-16	--	0	0	0	59	.45'	6.3
APBB6SS	Princeton Dr. @ Coal Ave.	K-16	36"	38	36"	6.0	38	36"	6.0

ANALYSIS POINT ID NO.	LOCATION	ZONE ATLAS MAP NO.	SS DIAM.	DRAINAGE SYSTEM BB Q10 (CFS)	D10	V10 (FPS)	Q100 (CFS)	D100	V100 (FPS)
APBB6	Princeton Dr. @ Coal Ave.	K-16	--	5	.24'	1.5	46	.58'	3.2
APBB4-1	Yale Blvd. @ Coal Ave.	K-16	--	0	0	0	29	.42'	3.5
APBB5SS	Princeton Dr. @ Garfield Ave.	K-16	48"	100	48"	8.6	100	48"	8.6
APBB5	Garfield Ave. @ Columbia Dr.	K-16	--	17	.35'	2.8	134	.86'	5.7
APBB4SS-36	Yale Blvd. @ Garfield Ave.	K-16	36"	40	36"	6.2	40	36"	6.2
APBB4SS-48	Yale Blvd. @ Garfield Ave.	K-16	48"	77	31"	9	104	48"	8.9
APBB4	Yale Blvd. @ Garfield Ave.	K-16	--	0	0	0	129	.81'	4.6
APBB2-2	Bell Ave. @ Yale	L-15	--	0	0	0	125	.99'	4.2
APBB2-1	Bell Ave. & Buena Vista	L-15	--	0	0	0	121	1.34'	2.6
APBB8SS-1	Stadium Blvd. @ Buena Vista Dr.	L-15	42"	65	32"	8.3	70	42"	7.9
APBB8-1	Stadium Blvd. @ University Blvd.	L-15	--	0	0	0	59	.6'	2.5
APBB3SS-54	Hazeldine Ave. @ University Blvd.	K-15	54"	75	30"	8.4	109	38"	9.0
APBB3SS-48	Hazeldine Ave. @ University Blvd.	K-15	36"	0	0	0	0	0	0
APBB3	Coal Ave. @ University Blvd.	K-15	--	0	0	0	0	0	0
APBB8SS-2	University Blvd. @ Stadium Blvd.	L-15	48"	80	27"	10.7	127	48"	10.9

ANALYSIS POINT ID NO.	LOCATION	ZONE ATLAS MAP NO.	SS DIAM.	DRAINAGE SYSTEM BB, CC Q10 (CFS)	D10	V10 (FPS)	Q100 (CFS)	D100	V100 (FPS)
APBB8	Stadium Blvd. @ University	L-15	--	0	0	0	114	.57'	6.4
APBB1	Stadium Blvd. @ I-25	L-15	--	0	0	0	0	0	0
APCC9SS	Vail Ave. @ Girard Blvd.	L-16	27"	20	12"	11.5	47	27"	13
APCC9	Vail Ave. @ Girard Blvd.	L-16	--	0	0	0	12	.24'	4.0
APCC10	Yale Blvd. south of Ross Ave.	L-16	--	6	.44'	2.7	18	.6'	3.4
APCC8SS	Buena Vista Dr. north of Ross Ave.	L-15	54"	95	20"	17.6	174	28"	21.6
APCC8	Buena Vista Dr. @ Ross Ave.	L-15	--	0	0	0	0	0	0
APCC6SS	Kathryn Ave. @ Yale Blvd.	L-16	36"	20	11"	11.0	45	17"	13.7
APCC6	Kathryn Ave. @ Yale Blvd.	L-16	--	0	0	0	0	0	0
APCC7	Yale Blvd. south of Kathryn Ave.	L-16	--	1	.11'	2.2	9	.23'	3.6
APCC5SS	South of Kathryn Ave. on Buena Vista Dr.	L-15	36"	75	26"	13.7	83	36"	13
APCC5SS-OL	South of Kathryn Ave. on Buena Vista Dr.	L-15	--	0	0	0	30	.51'	3.3
APCC2SS1-SS	Univesity Blvd. north of Sunshine Terrace	L-15	54"	153	40"	12.1	173	54"	11.6
APCC2SS1-OL	University Blvd. north of Sunshine Terrace	L-15	--	0	0	0	59	.47'	4.8

ANALYSIS POINT ID NO.	LOCATION	ZONE ATLAS MAP NO.	SS DIAM.	DRAINAGE SYSTEM CC, EE, DD Q10 (CFS)	D10	V10 (FPS)	Q100 (CFS)	D100	V100 (FPS)
APCC5	University Blvd. south of Sunshine Terrace	L-15	--	108	1.12'	6.6	289	1.86'	9.0
APCC3	Stadium Blvd. @ South Diversion Channel	L-15	--	120	3.6'	1.9	406	6.62'	2.6
APCC2	Gibson Blvd. @ South Diversion Channel	L-15	--	350	4.34'	3.4	763	6.64'	4.2
APCC12A	Buena Vista Dr. south of Renard Pl.	M-15	--	55	.53'	4.7	154	.94'	6.6
APCC1	I-25 @ Diversion Channel	M-15	--	354	4.3'	3.4	913	7.07'	4.4
APCC11		M-15	--	0	0	0	0	0	0
APCC11SS		M-15	66"	25	12"	8.2	69	20"	10.9
APEE5SS	Yale Blvd. @ Alamo Ave.	M-16	66"	55	14"	14.8	154	25"	19.8
APEE5	Yale Blvd. @ Alamo Ave.	M-16	--	0	0	0	0	0	0
APEE4	Yale Blvd. @ Gibson Ave.	M-16	--	20	.37'	2.7	55	.58'	3.6
APEE2SS	Yale Blvd. @ Gibson Ave.	L-16	42"	70	29"	10.1	87	42"	9.7
APEE2	Yale Blvd. @ Gibson Ave.	L-16	--	0	0	0	112	.54'	6.8
APEE1SS	Mulberry Ave. (extension of) @ Gibson Ave.	L-15	--	9	Curb Cut		104	Curb Cut	
APEE1	Gibson Ave. @ I-25	L-15	--	0	0	0	0	0	0
APDD1	Gibson Ave. @ I-25	L-15	3-8'x5' CBCs	2/3	.1'	2	13/3	.1'	2.0

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BASIN HYDROLOGY

Subbasin ID No.	Area (Sq. Mi.)	Curve No.	% IMP	TP	K	Q10	Q100
A1	0.1562	68	7	0.2760	0.1380	20	62
A2	0.0536	68	18	0.1781	0.0891	17	37
A3	0.1604	70	18	0.3093	0.1546	43	97
A4	0.1506	68	14	0.1985	0.0993	36	88
A5	0.1857	69	15	0.3418	0.1709	39	93
AA1	0.1235	70	12	0.2028	0.1014	19	57
B1	0.1465	69	19	0.3027	0.1514	12	46
BB1	0.0932	70	11	0.1527	0.0764	15	46
BB2	0.2187	59	5	0.2824	0.1412	11	22
BB3	0.0325	71	14	0.1016	0.0508	8	22
BB4	0.0576	71	18	0.1898	0.0949	14	35
BB5	0.1796	71	18	0.2966	0.1483	36	89
BB6	0.1112	70	18	0.2342	0.1171	25	66
BB7	0.1654	70	18	0.3385	0.1693	31	82
BB8	0.1009	68	6	0.2293	0.1146	7	29
BB9	0.1103	70	18	0.1938	0.0969	27	72
BB10	0.0696	70	18	0.1690	0.0845	18	48
BB11	0.0784	68	7	0.1344	0.0672	8	33
BB12	0.0740	68	18	0.3684	0.1842	12	32
C1	0.0410	68	18	0.1956	0.0978	12	28
CC1	0.0255	62	85	0.1694	0.0847	30	51
CC2	0.1313	72	20	0.2233	0.1116	34	83
CC3	0.0835	66	5	0.1628	0.0814	6	21
CC4	0.1421	78	0	0.1354	0.0677	32	105
CC5	0.1932	70	16	0.1935	0.0967	40	105
CC6	0.0609	72	18	0.1470	0.0735	20	45
CC7	0.0287	68	0	0.0882	0.0441	1	11
CC8	0.0602	72	9	0.1708	0.0854	9	31
CC9	0.1357	68	10	0.2660	0.1330	15	51
CC10	0.0947	62	5	0.1910	0.0955	6	18
CC11	0.1320	70	15	0.1960	0.0980	26	69
CC12	0.2028	58	4	0.1767	0.0884	11	20
D1	0.0540	68	8	0.1861	0.0930	9	26
DD1	0.0323	70	0	0.0426	0.0213	2	13
E1	0.0049	70	9	0.0288	0.0144	2	5
EE1	0.1066	70	6	0.2310	0.1155	9	37
EE2	0.2384	72	16	0.2065	0.1033	55	158
EE4	0.0658	71	18	0.1175	0.0588	20	55
EE5	0.2688	70	16	0.2245	0.1123	55	154
F1	0.0102	55	5	0.0860	0.0430	1	2
F2	0.0416	68	9	0.2104	0.1052	7	20
F3	0.2409	68	18	0.3823	0.1912	53	118
F4	0.0365	68	18	0.1389	0.0694	13	28
F5	0.0572	68	18	0.2674	0.1337	15	34
F6	0.2622	68	18	0.6446	0.3223	43	95
F7	0.2458	69	6	0.3754	0.1877	29	88
FF1	0.0152	59	15	0.1469	0.0735	3	5

Identification Number EE3 was not used.

BASIN HYDROLOGY

(Continued)

Subbasin ID No.	Area (Sq. Mi.)	Curve No.	% IMP	TP	K	Q10	Q100
G1	0.0955	69	18	0.2357	0.1178	28	62
GG1	0.3202	65	6	0.2006	0.1003	10	30
H1	0.3000	70	18	0.4252	0.2126	68	152
HH1	0.0631	66	0	0.1289	0.0644	1	12
HH2	0.3087	65	8	0.1246	0.0623	36	92
J1	0.0974	70	6	0.3282	0.1641	14	40
JJ1	0.1785	65	0	0.1583	0.0791	4	35
K1	0.0333	72	0	0.1312	0.0656	3	13
K2	0.1074	70	15	0.1911	0.0956	31	74
K3	0.1993	69	18	0.2588	0.1294	55	125
K4	0.1281	70	18	0.3374	0.1687	33	74
K5	0.2456	70	18	0.4115	0.2057	57	127
K6	0.0768	70	18	0.2538	0.1269	22	51
K7	0.1309	69	18	0.3219	0.1609	33	74
K8	0.0209	70	18	0.1206	0.0603	8	19
K9	0.0310	71	18	0.1135	0.0568	13	30
KK1	0.1660	59	0	0.2012	0.1006	1	12
KK2	0.3126	65	9	0.1183	0.0592	45	118
L1	0.1471	70	18	0.2855	0.1427	29	71
L2	0.0752	68	18	0.1812	0.0906	17	40
L3	0.2457	72	18	0.2413	0.1207	56	142
L4	0.2575	68	18	0.3090	0.1545	63	141
L5	0.1566	68	18	0.2710	0.1355	41	91
L6	0.0228	68	18	0.0861	0.0430	9	21
LL1	0.4575	59	2	0.1630	0.0815	12	33
M1	0.0301	95	100	0.8684	0.4342	14	25
M2	0.1853	68	18	0.2281	0.1141	38	90
M3	0.1084	68	18	0.2185	0.1092	23	54
M4	0.2050	68	18	0.3344	0.1672	35	83
M5	0.0838	68	18	0.1565	0.0783	20	47
M6	0.1274	68	18	0.2779	0.1390	24	57
M7	0.2317	68	18	0.4273	0.2137	48	107
M8	0.1610	68	12	0.3310	0.1655	28	71
M9	0.0272	68	20	0.1939	0.0970	9	19
M10	0.0878	68	11	0.2797	0.1398	15	40
M11	0.2747	68	18	0.5062	0.2531	52	115
M12	0.0520	68	18	0.3462	0.1731	12	27
M13	0.0465	68	18	0.2143	0.1072	14	30
M14	0.2041	68	18	0.3282	0.1641	48	108
M15	0.4939	68	18	0.5144	0.2572	92	205
M16	0.0872	74	5	0.3382	0.1691	13	45
M17	0.1078	68	18	0.2795	0.1397	28	62
M18	0.0299	68	18	0.1177	0.0588	11	25
M19	0.0862	68	18	0.1796	0.0898	27	60
M20	0.1350	68	18	0.3796	0.1898	30	67
M21	0.0832	68	18	0.3637	0.1819	19	42
M22	0.0203	66	18	0.1215	0.0607	6	13
M23	0.0300	68	9	0.0805	0.0403	5	14

BASIN HYDROLOGY

(Continued)

Subbasin ID No.	Area (Sq. Mi.)	Curve No.	% IMP	TP	K	Q10	Q100
M24	0.1418	70	15	0.3884	0.1942	20	53
M25	0.0231	65	0	0.0967	0.0483	1	4
M26	0.0451	72	3	0.1142	0.0571	5	21
M27	0.0308	68	9	0.1271	0.0635	4	13
M28	0.0779	60	16	0.1471	0.0735	17	31
M29	0.0230	68	18	0.1936	0.0968	7	15
MM1	0.0856	62	12	0.1120	0.0560	15	30
N1	0.0200	64	0	0.1918	0.0959	4	8
NN1	0.1148	64	0	0.0773	0.0386	2	23
P1	0.0646	63	6	0.1749	0.0875	5	12
Q1	0.0084	64	24	0.0607	0.0303	3	6
R1	0.1616	71	7	0.1765	0.0882	12	57
S1	0.1277	69	6	0.1942	0.0971	11	43
T1	0.0066	62	10	0.0977	0.0488	1	2
U1	0.1554	61	18	0.1962	0.0981	34	62
U2	0.2462	63	18	0.3129	0.1565	42	84
U3	0.1471	62	18	0.2423	0.1211	29	55
U4	0.2230	65	18	0.3046	0.1523	50	108
U5	0.0463	68	20	0.2457	0.1228	11	26
U5A	0.0256	68	22	0.1714	0.0857	8	18
U6	0.0580	68	18	0.3534	0.1767	10	26
U7	0.1463	68	18	0.3513	0.1757	25	65
U8	0.0237	68	20	0.2005	0.1002	6	15
U9	0.1332	68	18	0.2899	0.1450	25	65
U10	0.0754	68	18	0.3344	0.1672	13	34
U11	0.2689	67	9	0.3259	0.1630	35	100
U12	0.0356	68	18	0.1631	0.0815	5	16
U13	0.2935	70	9	0.4052	0.2026	29	100
U14	0.0527	68	18	0.2114	0.1057	12	30
U15	0.0546	68	18	0.1967	0.0983	13	32
U16	0.3774	67	18	0.3153	0.1576	67	169
U17	0.0255	68	22	0.1195	0.0598	9	21
U17A	0.0898	68	22	0.2950	0.1475	25	56
U18	0.1044	68	18	0.2536	0.1268	21	54
U19	0.2028	66	18	0.3193	0.1596	36	86
U20	0.4303	66	18	0.4909	0.2454	58	145
U21	0.1300	68	18	0.2649	0.1324	26	66
U22	0.1215	68	18	0.2765	0.1382	24	61
U23	0.1350	68	18	0.3529	0.1764	23	60
U24	0.1189	67	5	0.3854	0.1927	6	28
U25	0.2851	67	5	0.3491	0.1745	15	70
U26	0.1114	68	14	0.3482	0.1741	15	43
U27	0.0624	68	18	0.2173	0.1087	14	35
U28	0.1266	68	18	0.3272	0.1636	23	58
U29	0.1744	66	18	0.1809	0.0904	42	96
U30	0.0509	68	9	0.4736	0.2368	4	14
U31	0.2282	66	18	0.3302	0.1651	39	95
U32	0.1228	68	18	0.3093	0.1547	23	58

BASIN HYDROLOGY

(Continued)

Subbasin ID No.	Area (Sq. Mi.)	Curve No.	% IMP	TP	K	Q10	Q100
U33	0.2546	64	18	0.2280	0.1140	54	127
U34	0.1192	66	18	0.3167	0.1584	21	56
U35	0.0645	66	18	0.2142	0.1071	14	36
U36	0.1988	66	18	0.4428	0.2214	29	78
U37	0.0926	66	18	0.2862	0.1431	17	46
U38	0.0963	66	18	0.2915	0.1457	18	47
U39	0.0376	66	18	0.3978	0.1989	6	16
U40	0.1245	64	18	0.1912	0.0956	29	67
U41	0.0806	66	18	0.2974	0.1487	15	39
U43	1.0258	68	16	0.6036	0.3018	116	315
U44	0.2180	67	5	0.3748	0.1874	11	52
U45	0.1086	67	0	0.3389	0.1694	3	21
U46	0.0505	68	18	0.1874	0.0937	12	30
U47	0.3604	68	16	0.5565	0.2782	43	116
U48	0.0673	68	16	0.2856	0.1428	12	31
U49	0.1534	68	14	0.2483	0.1241	25	70
U50	0.0962	66	18	0.2059	0.1030	21	55
U51	0.1164	66	18	0.3843	0.1921	18	45
U52	0.1002	66	9	0.2738	0.1369	10	36
U53	0.1187	66	18	0.2644	0.1322	29	70
U54	0.0920	71	6	0.1921	0.0960	17	52
V1	0.1960	72	18	0.1787	0.0894	50	130
V2	0.0527	72	18	0.1812	0.0906	13	35
V3	0.0576	72	18	0.1983	0.0991	14	37
W1	0.1726	71	18	0.2587	0.1293	36	92
W2	0.1301	70	18	0.3350	0.1675	23	58
W3	0.0552	71	18	0.3121	0.1561	11	30
W4	0.1105	70	18	0.3290	0.1645	20	50
W5	0.0831	72	18	0.1978	0.0989	22	59
X1	0.0319	71	18	0.1697	0.0849	8	20
Y1	0.0198	71	18	0.0996	0.0498	6	15
Y2	0.0999	71	18	0.2172	0.1086	23	58
Z1	0.0654	71	12	0.2024	0.1012	11	32

Identification Number U42 was not used.

MASTER PLAN
OF
DRAINAGE
CITY OF ALBUQUERQUE, NEW MEXICO
AND ENVIRONS
1963

TEXT AND COST ESTIMATES

GORDON HERKENHOFF AND ASSOCIATES
CONSULTING ENGINEERS
ALBUQUERQUE, NEW MEXICO

INSTRUCTIONS FOR DRAINAGE STUDY REVISIONS

ACADEMY ACRES, UNIT 9

1. Page i, Drainage Study Index: Change title of Exhibit "F" to "STREETS AND MAXIMUM RUNOFFS".

2. Page 4, RECOMMENDATIONS AND CONCLUSIONS: Delete sentence "Recommended street grades are contained in Exhibit 'F'" and replace it with "Design streets to carry maximum runoffs contained in Exhibit 'F' through use of tables in Exhibit 'E'".

3. Page 12, Exhibit "E": Delete "n(composite)=.006" and table "R/W TO R/W, OVERFLOW SECTIONS CAPACITIES".

4. Replace page 14 with revised page 14A, 14B, 14C, and 14D.

5. Replace page 15 with revised page 15 entitled "STREETS AND MAXIMUM RUNOFFS".

REVISED MAY 1972

50' R.O.W. 32' FACE-FACE 8" CURB 8" CROWN .2' OVERFLOW

N	A	P	S	S1/2	R	R2/3	V	Q
.0165	15.3611	51.3731	.0050	.0707	.2990	.4472	2.8545	43.8477
.0165	15.3611	51.3731	.0100	.1000	.2990	.4472	420368	62.0100
.0165	15.3611	51.3731	.0150	.1225	.2990	.4472	4.9441	75.9464
.0165	15.3611	51.3731	.0200	.1414	.2990	.4472	5.7089	87.6954
.0165	15.3611	51.3731	.0250	.1581	.2990	.4472	6.3828	98.0464
.0165	15.3611	51.3731	.0300	.1732	.2990	.4472	6.9920	107.4045
.0165	15.3611	51.3731	.0350	.1871	.2990	.4472	7.5522	116.0101
.0165	15.3611	51.3731	.0400	.2000	.2990	.4472	8.0736	124.0200

50' R.O.W. 32' FACE-FACE 8" CURB 6" CROWN .2' OVERFLOW

N	A	P	S	S1/2	R	R2/3	V	Q
.0165	18.9167	51.3569	.0050	.0707	.3683	.5138	3.2801	62.0482
.0165	18.9167	51.3569	.0100	.1000	.3683	.5138	4.6387	87.7494
.0165	18.9167	51.3569	.0150	.1225	.3683	.5138	5.6813	107.4706
.0165	18.9167	51.3569	.0200	.1414	.3683	.5138	6.5602	124.0964
.0165	18.9167	51.3569	.0250	.1581	.3683	.5138	7.3345	138.7439
.0165	18.9167	51.3569	.0300	.1732	.3683	.5138	8.0345	151.9864
.0165	18.9167	51.3569	.0350	.1871	.3683	.5138	8.6783	164.1640
.0165	18.9167	51.3569	.0400	.2000	.3683	.5138	9.2775	175.4988

50' R.O.W. 32' FACE-FACE 8" CURB 4" CROWN .2' OVERFLOW

N	A	P	S	S1/2	R	R2/3	V	Q
.0165	22.4722	51.3453	.0050	.0707	.4377	.5765	3.6797	82.6903
.0165	22.4722	51.3453	.0100	.1000	.4377	.5765	5.2038	116.9417
.0165	22.4722	51.3453	.0150	.1225	.4377	.5765	6.3734	143.2238
.0165	22.4722	5123453	.0200	.1414	.4377	.5765	7.3593	165.3806
.0165	22.4722	51.3453	.0250	.1581	.4377	.5765	8.2280	184.9011
.0165	22.4722	51.3453	.0300	.1732	.4377	.5765	9.0133	202.5490
.0165	22.4722	51.3453	.0350	.1871	.4377	.5765	9.7355	218.7780
.0165	22.4722	51.3453	.0400	.2000	.4377	.5765	10.4077	233.8835

50' R.O.W. 32' FACE-FACE 8" CURB 2" CROWN .2' OVERFLOW

N	A	P	S	S1/2	R	R2/3	V	Q
.0165	26.0278	51.3384	.0050	.0707	.5070	.6358	4.0585	105.6348
.0165	26.0278	51.3384	.0100	.1000	.5070	.6358	5.7396	149.3901
.0165	26.0278	51.3384	.0150	.1225	.5070	.6358	7.0296	182.9648
.0165	26.0278	51.3384	.0200	.1414	.5070	.6358	8.1171	211.2696
.0165	26.0278	51.3384	.0250	.1581	.5070	.6358	9.0752	236.2066
.0165	26.0278	51.3384	.0300	.1732	.5070	.6358	9.9414	258.7513
.0165	26.0278	51:3384	.0350	.1871	.5070	.6358	10.7379	279.4834
.0165	26.0278	51.3384	.0400	.2000	.5070	.6358	11.4793	298.7803

50' R.O.W. 32' FACE-FACE 8" CURB 8" CROWN .4' OVERFLOW

N	A	P	S	S1/2	R	R2/3	V	Q
.0165	24.3611	51.3904	.0050	.0707	.4740	.6080	3.8805	94.5336
.0165	24.3611	51.3904	.0100	.1000	.4740	.6080	5.4879	133.6907
.0165	24.3611	51.3904	.0150	.1225	.4740	.6080	6.7212	163.7370
.0165	24.3611	51.3904	.0200	.1414	.4740	.6080	7.7610	189.0672
.0165	24.3611	51.3904	.0250	.1581	.4740	.6080	8.6771	211.3836
.0165	24.3611	51.3904	.0300	.1732	.4740	.6080	9.5053	231.5591
.0165	24.3611	51.3904	.0350	.1871	.4740	.6080	10.2669	250.1124
.0165	24.3611	51.3904	.0400	.2000	.4740	.6080	10.9757	267.3814

50' R.O.W. 32' FACE-FACE 8" CURB 6" CROWN .4' OVERFLOW

N	A	P	S	S1/2	R	R2/3	V	Q
.0165	27.9167	51.3742	.0050	.0707	.5434	.6659	4.2502	118.6523
.0165	27.9167	51.3742	.0100	.1000	.5434	.6659	6.0107	167.7996
.0165	27.9167	51.3742	.0150	.1225	.5434	.6659	7.3616	205.5117
.0165	27.9167	51.3742	.0200	.1414	.5434	.6659	8.5005	237.3045
.0165	27.9167	51.3742	.0250	.1581	.5434	.6659	9.5038	265.3145
.0165	27.9167	51.3742	.0300	.1732	.5434	.6659	10.4109	290.6375
.0165	27.9167	51.3742	.0350	.1871	.5434	.6659	11.2451	313.9244
.0165	27.9167	51.3742	.0400	.2000	.5434	.6659	12.0215	335.5993

50' R.O.W. 32' FACE-FACE 8" CURB 4" CROWN .4' OVERFLOW

N	A	P	S	S1/2	R	R2/3	V	Q
.0165	31.4722	51.3626	.0050	.0707	.6127	.7214	4.6045	144.9124
.0165	31.4722	51.3626	.0100	.1000	.6127	.7214	6.5117	204.9371
.0165	31.4722	51.3626	.0150	.1225	.6127	.7214	7.9751	250.9957
.0165	31.4722	51.3626	.0200	.1414	.6127	.7214	9.2089	289.8248
.0165	31.4722	51.3626	.0250	.1581	.6127	.7214	10.2959	324.0340
.0165	31.4722	51.3626	.0300	.1732	.6127	.7214	11.2786	354.9615
.0165	31.4722	51.3626	.0350	.1871	.6127	.7214	12.1822	383.4022
.0165	31.4722	51.3626	.0400	.2000	.6127	.7214	13.0234	409.8742

50' R.O.W. 32' FACE-FACE 8" CURB 2" CROWN .4' OVERFLOW

N	A	P	S	S1/2	R	R2/3	V	Q
.0165	35.0278	51.3557	.0050	.0707	.6821	.7748	4.9454	173.2267
.0165	35.0278	51.3557	.0100	.1000	.6821	.7748	6.9939	244.9795
.0165	35.0278	51.3557	.0150	.1225	.6821	.7748	8.5657	300.0374
.0165	35.0278	51.3557	.0200	.1414	.6821	.7748	9.8908	346.4534
.0165	35.0278	51.3557	.0250	.1581	.6821	.7748	11.0583	387.3466
.0165	35.0278	51.3557	.0300	.1732	.6821	.7748	12.1137	424.3170
.0165	35.0278	51.3557	.0350	.1871	.6821	.7748	13.0843	458.3147
.0165	35.0278	51.3557	.0400	.2000	.6821	.7748	13.9877	489.9590

50' R.O.W. 32' FACE-FACE 8" CURB 8" CROWN .6' OVERFLOW

N	A	P	S	S1/2	R	R2/3	V	Q
.0165	33.3611	51.4236	.0050	.0707	.6488	.7494	4.7822	159.5397
.0165	33.3611	51.4236	.0100	.1000	.6488	.7494	6.7631	225.6231
.0165	33.3611	51.4236	.0150	.1225	.6488	.7494	8.2830	276.3308
.0165	33.3611	51.4236	.0200	.1414	.6488	.7494	9.5644	319.0793
.0165	33.3611	51.4236	.0250	.1581	.6488	.7494	10.6933	356.7415
.0165	33.3611	51.4236	.0300	.1732	.6488	.7494	11.7140	390.7908
.0165	33.3611	51.4236	.0350	.1871	.6488	.7494	12.6525	422.1023
.0165	33.3611	51.4236	.0400	.2000	.6488	.7494	13.5261	451.2463

50' R.O.W. 32' FACE-FACE 8" CURB 6" CROWN .6' OVERFLOW

N	A	P	S	S1/2	R	R2/3	V	Q
.0165	36.9167	51.4074	.0050	.0707	.7181	.8019	5.1172	188.9082
.0165	36.9167	51.4074	.0100	.1000	.7181	.8019	7.2367	267.1566
.0165	36.9167	51.4074	.0150	.1225	.7181	.8019	8.8632	327.1987
.0165	36.9167	51.4074	.0200	.1414	.7181	.8019	10.2343	377.8165
.0165	36.9167	51.4074	.0250	.1581	.7181	.8019	11.4423	422.4116
.0165	36.9167	51.4074	.0300	.1732	.7181	.8019	12.5344	462.7288
.0165	36.9167	51.4074	.0350	.1871	.7181	.8019	13.5387	499.8042
.0165	36.9167	51.4074	.0400	.2000	.7181	.8019	14.4735	534.3132

50' R.O.W. 32' FACE-FACE 8" CURB 4" CROWN .6' OVERFLOW

N	A	P	S	S1/2	R	R2/3	V	Q
.0165	40.4722	51.3958	.0050	.0707	.7875	.8527	5.4414	220.2241
.0165	40.4722	51.3958	.0100	.1000	.7875	.8527	7.6953	311.4440
.0165	40.4722	51.3958	.0150	.1225	.7875	.8527	9.4247	381.4394
.0165	40.4722	51.3958	.0200	.1414	.7875	.8527	10.8827	440.4483
.0165	40.4722	51.3958	.0250	.1581	.7875	.8527	12.1673	492.4362
.0165	40.4722	51.3958	.0300	.1732	.7875	.8527	13.3286	539.4368
.0165	40.4722	51.3958	.0350	.1871	.7875	.8527	14.3965	582.6583
.0165	40.4722	51.3958	.0400	.2000	.7875	.8527	15.3905	622.8879

50' R.O.W. 32' FACE-FACE 8" CURB 2" CROWN .6' OVERFLOW

N	A	P	S	S1/2	R	R2/3	V	Q
.0165	44.0278	51.3889	.0050	.0707	.8568	.9021	5.7560	253.4243
.0165	44.0278	51.3889	.0100	.1000	.8568	.9021	8.1402	358.3961
.0165	44.0278	51.3889	.0150	.1225	.8568	.9021	9.9697	438.9438
.0165	44.0278	51.3889	.0200	.1414	.8568	.9021	11.5120	506.8487
.0165	44.0278	51.3889	.0250	.1581	.8568	.9021	12.8708	566.6741
.0165	44.0278	51.3889	.0300	.1732	.8568	.9021	14.0993	620.7603
.0165	44.0278	51.3889	.0350	.1871	.8568	.9021	15.2290	670.4978
.0165	44.0278	51.3889	.0400	.2000	.8568	.9021	16.2805	716.7923

50' R.O.W. 32' FACE-FACE 8" CURB 8" CROWN .7' OVERFLOW

N	A	P	S	S1/2	R	R2/3	V	Q
.0165	37.8611	51.4461	.0050	.0707	.7359	.8151	5.2008	196.9066
.0165	37.8611	51.4461	.0100	.1000	.7359	.8151	7.3550	278.4680
.0165	37.8611	51.4461	.0150	.1225	.7359	.8151	9.0080	341.0523
.0165	37.8611	51.4461	.0200	.1414	.7359	.8151	10.4015	393.8132
.0165	37.8611	51.4461	.0250	.1581	.7359	.8151	11.6293	440.2966
.0165	37.8611	51.4461	.0300	.1732	.7359	.8151	12.7392	482.3207
.0165	37.8611	51.4461	.0350	.1871	.7359	.8151	13.7599	520.9659
.0165	37.8611	51.4461	.0400	.2000	.7359	.8151	14.7100	556.9360

50' R.O.W. 32' FACE-FACE 8" CURB 6" CROWN .7' OVERFLOW

N	A	P	S	S1/2	R	R2/3	V	Q
.0165	41.4167	51.4299	.0050	.0707	.8053	.8656	5.5225	228.7225
.0165	41.4167	51.4299	.0100	.1000	.8053	.8656	7.8100	323.4624
.0165	41.4167	51.4299	.0150	.1225	.8053	.8656	9.5652	396.1589
.0165	41.4167	51.4299	.0200	.1414	.8053	.8656	11.0449	457.4449
.0165	41.4167	51.4299	.0250	.1581	.8053	.8656	12.3486	511.4390
.0165	41.4167	51.4299	.0300	.1732	.8053	.8656	13.5272	560.2533
.0165	41.4167	51.4299	.0350	.1871	.8053	.8656	14.6111	605.1428
.0165	41.4167	51.4299	.0400	.2000	.8053	.8656	15.6199	646.9248

50' R.O.W. 32' FACE-FACE 8" CURB 4" CROWN .7' OVERFLOW

N	A	P	S	S1/2	R	R2/3	V	Q
.0165	44.9722	51.4184	.0050	.0707	.8746	.9146	5.8349	262.4101
.0165	44.9722	51.4184	.0100	.1000	.8746	.9146	8.2518	371.1039
.0165	44.9722	51.4184	.0150	.1225	.8746	.9146	10.1064	454.5076
.0165	44.9722	51.4184	.0200	.1414	.8746	.9146	11.6699	524.8202
.0165	44.9722	51.4184	.0250	.1581	.8746	.9146	13.0473	586.7668
.0165	44.9722	51.4184	.0300	.1732	.8746	.9146	14.2926	642.7709
.0165	44.9722	51.4184	.0350	.1871	.8746	.9146	15.4378	694.2719
.0165	44.9722	51.4184	.0400	.2000	.8746	.9146	16.5037	742.2079

50' R.O.W. 32' FACE-FACE 8" CURB 2" CROWN .7' OVERFLOW

N	A	P	S	S1/2	R	R2/3	V	Q
.0165	48.5278	51.4114	.0050	.0707	.9439	.9622	6.1390	297.9142
.0165	48.5278	51.4114	.0100	.1000	.9439	.9622	8.6819	421.3143
.0165	48.5278	51.4114	.0150	.1225	.9439	.9622	10.6331	516.0025
.0165	48.5278	51.4114	.0200	.1414	.9439	.9622	12.2781	595.8284
.0165	48.5278	51.4114	.0250	.1581	.9439	.9622	13.7273	666.1564
.0165	48.5278	51.4114	.0300	.1732	.9439	.9622	15.0375	729.7377
.0165	48.5278	51.4114	.0350	.1871	.9439	.9622	16.2424	788.2068
.0165	48.5278	51.4114	.0400	.2000	.9439	.9622	17.3638	842.6286

EXHIBIT F

STREETS AND MAXIMUM RUNOFFS

STREET DESCRIPTION & LOCATION	MAXIMUM RUNOFF (in c.f.s.)
A. East-west street 145' north of Burlison Drive, N.E. between Louisiana Blvd., N.E., and Truchas Drive, N.E. - 50' R/W, 32' face to face	15
B. East-west street 435' north of Burlison Drive, N.E., between Louisiana Blvd., N.E., and Truchas Drive, N.E. - 50' R/W, 32' face to face	26
C. Avenida La Costa, N.E., between Louisiana Blvd., N.E., and Truchas Drive, N.E., 50' R/W, 32' face to face	29

REVISED MAY 1972
AA #9

MASTER PLAN OF DRAINAGE

CITY OF ALBUQUERQUE, NEW MEXICO

AND ENVIRONS

The preparation of this document was financially aided through a Federal Grant from the Urban Renewal Administration of the Housing and Home Finance Agency, under the Urban Planning Assistance Program authorized by Section 701 of the Housing Act of 1954, as amended.

1221 Luisa Street
Santa Fe, New Mexico

302 Eighth Street, NW
Albuquerque, New Mexico

Gordon Herkenhoff & Associates, Inc.

Consulting Engineers

March 1, 1963

Mr. Edmund L. Engel, City Manager
City Hall Building
Albuquerque, New Mexico

Dear Mr. Engel:

In accordance with the terms of our contract, transmitted herewith is the Master Plan of Drainage.

In addition to the report, we have prepared ten copies of larger scale map books for use by the various departments which are required to examine drainage requirements.

Since the drainage system will be governed to a large degree by the development of the street system, detailed examination will be required as streets are platted. However, all data necessary to facilitate detailed analyses of specific requirements is included.

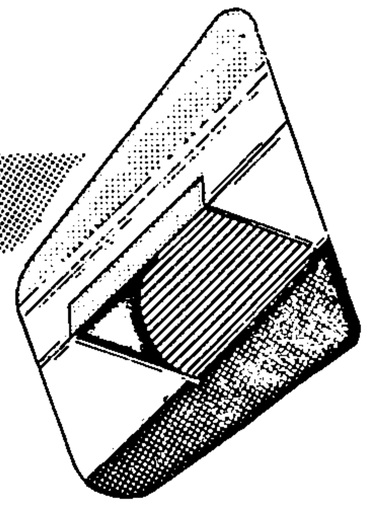
We believe the recommendations contained herein should be adopted in a timely manner.

Respectfully submitted,

Gordon Herkenhoff & Associates, Inc.

Gordon Herkenhoff

GH/db



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CITY OF ALBUQUERQUE, NEW MEXICO

AND ENVIRONS

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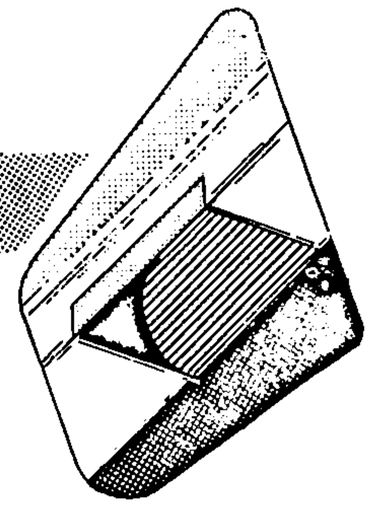

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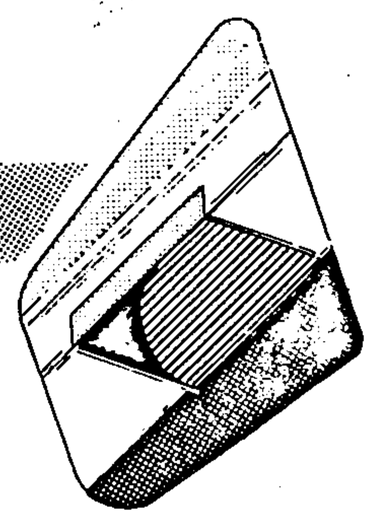

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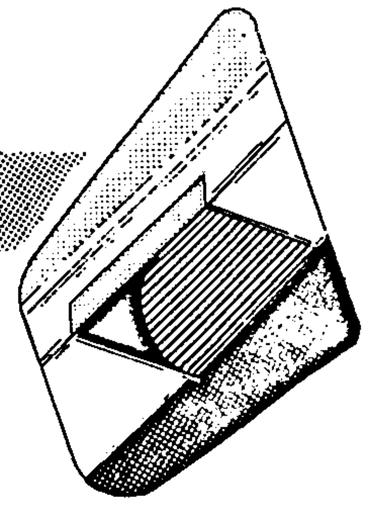

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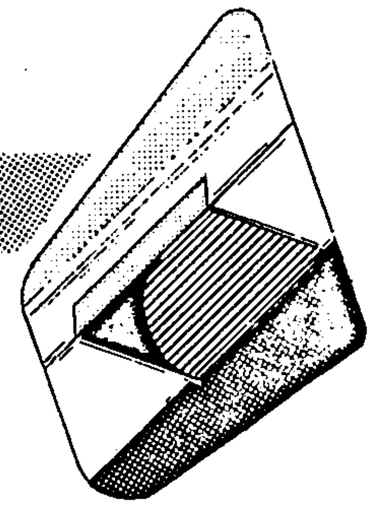

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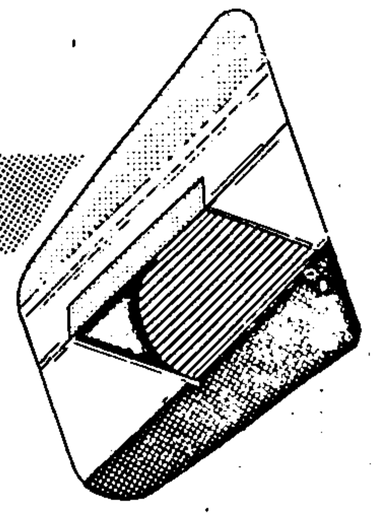

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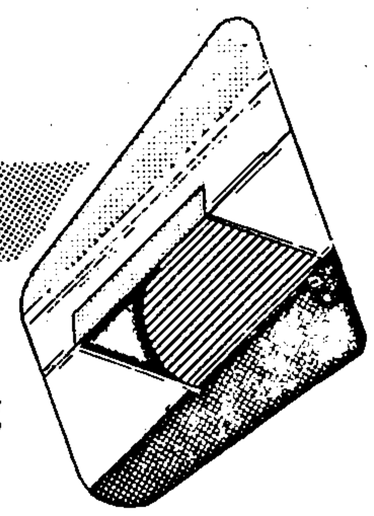
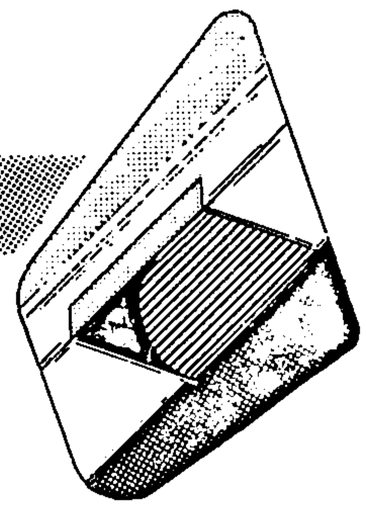


PREFACE

This Master Plan of Drainage presents analyses of existing drainage and flood problems together with data and discussions of methods which may be employed to deal with them or other anticipated conditions.

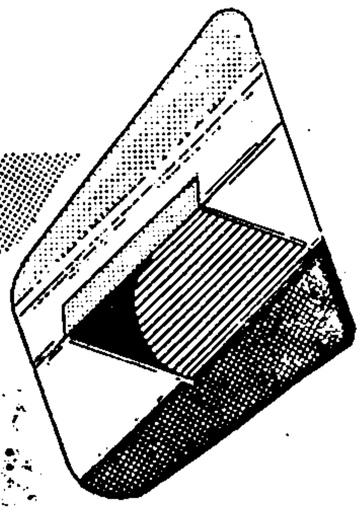
Most of the existing problems have resulted from natural topographic features and climatic conditions. Others have been created or complicated by the manner in which the metropolitan area has developed. Flood potential and effects of floods define difficulties which are possible as a result of storm drainage as well as indicating treatment which is necessary. Previous efforts to alleviate flood problems are also pertinent for completing the analysis.

Developing solutions for these problems requires a study of existing and potential facilities used for handling runoff. Existing natural facilities consist of ephemeral streams (arroyos) and ponding areas. Artificial installations such as streets, drainage channels, pump stations and storm sewers are also available. The most effective means of solving problems will probably require extending the use of these and other similar facilities. In order to determine the best function of existing facilities, it is necessary to evaluate all of their characteristics and hydraulic potential.



Methods for handling storm runoff are established from basic criteria that has been developed. Data regarding rainfall, runoff and hydraulic capacities establish a basis for determining amounts of and means for handling storm runoff. Broad variations in topographic and geographic features within the master plan area require that subdivision into characteristic areas be accomplished in order to simplify the application of basic criteria. The development of the plan is accomplished by applying basic data derived for these characteristic areas. Fixed locations have not been established for most installations in order to permit maximum flexibility in developing the system. Actual design has been avoided, since this can obscure the fact that a number of solutions are possible for each problem. Conflicting requirements of all necessary services are thereby made less difficult to resolve.

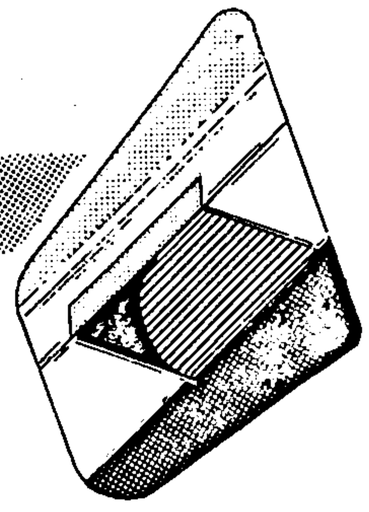
The developed plan consists of: (1) identification of drainage problems; (2) determination of existing facilities available along with those which can be developed to alleviate these problems; (3) derivation of basic criteria to facilitate analyses; (4) presentation of data applicable to the various facilities; (5) subdivision of the master plan area in accordance with topographic and geographic influences; and (6) application of basic criteria and data to characteristic areas in order to evaluate the possible use of each type



of facility.

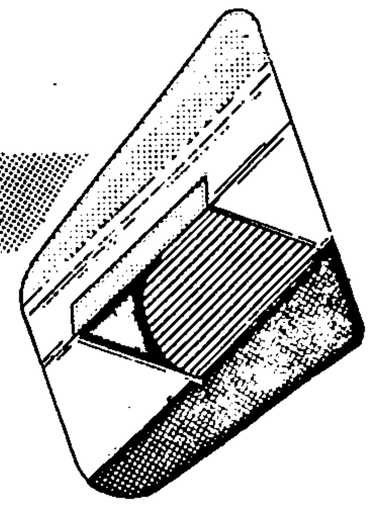
The Master Plan of Drainage is presented in three parts: (1) Text; (2) Technical Analysis; and (3) Maps. The text is in narrative form to facilitate its use by all interested organizations. Technical appendices contain data and criteria necessary to analyze specific and detailed problems. It consists of charts, tables, data, computations and technical discussions. The maps define each arroyo and its natural contributing drainage area along with the area which can be expected to contribute runoff after development. Recommended major collection systems are indicated or delineated on the maps. The overall plan is designed to furnish sufficient technical and general data to enable technicians and other individuals or agencies to determine the best methods for handling storm runoff.

In considering the treatment of any drainage problem, the following are of paramount importance: (1) costs to construct, operate and maintain facilities; and (2) methods of financing these costs. Discussions of cost estimates and financing are presented with the text. Information regarding facilities and their development are based largely upon economic considerations in order to provide means for finalizing plans. Plans based on maximum use of all available facilities provide the most economical treatment.



Cost estimates contained in this report are predicated on making maximum use of all available facilities. If this is not accomplished, these estimates will require revision or the degree of protection must be altered.

It must be emphasized that the large undeveloped area covered within this report precludes detailed treatment of specific areas. However, the data, concepts and discussion developed herein must all be considered when detailed analyses are made. Specific information set forth in the plan must be recognized if sound and economical solutions are to be accomplished. Recommendations contained herein outline basic policy required to establish effective planning for drainage control.



MASTER PLAN OF DRAINAGE

CITY OF ALBUQUERQUE, NEW MEXICO

PURPOSE

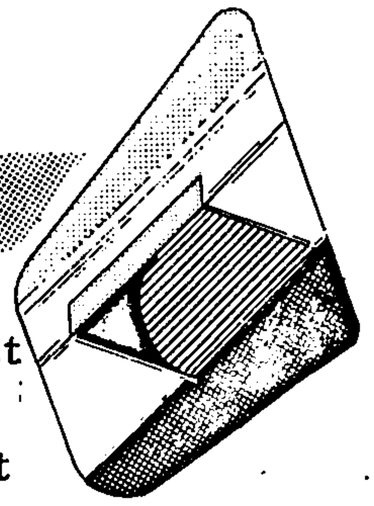
The purpose of this Master Plan of Drainage is to develop and apply general information, data and analyses pertaining to handling storm runoff.

SCOPE

The scope of this Master Plan is limited to the general drainage treatment of that portion of Bernalillo County bounded on the north by Sandoval County and the Sandia Pueblo Grant, on the east by the Cibola National Forest Boundary, on the south by the Isleta Pueblo Grant and on the west by the drainage divide between the Rio Grande and Rio Puerco watersheds, excluding therefrom all Military Reservations.

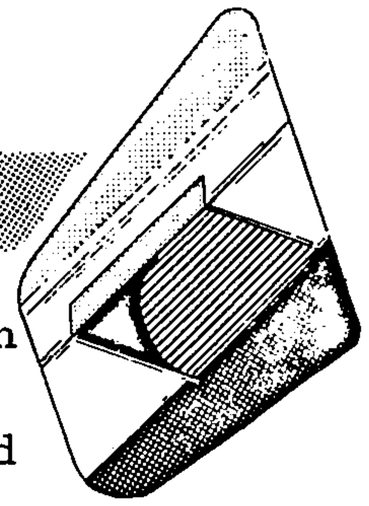
INTRODUCTION

AREA DESCRIPTION: The master plan area consists of a 372 square mile area situated in the Middle Rio Grande River Valley of southwestern United States. Natural vegetation is sparse and consists largely of grass and plant life characteristic of desert and semi-arid climates. The original community was largely agricultural, thus settlements were established in the proximity of the river. Those developments later grew as the City's importance as a trading center increased. The coming of the A.T. & S. F. Railroad east of the established community initiated new



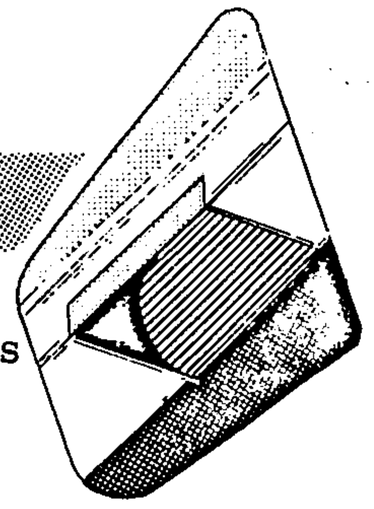
growth in that direction. Due to recent accelerated growth, most of the population is presently concentrated in highland areas east of the valley floor. Major industries are retail trade, services, and government agencies. The present center of these activities is located in a twenty-four square block area in the lowlands east of the river, but recent trends indicate that the business community is gradually moving eastward toward centers of new population.

Topographical features vary widely. The valley floor is approximately twenty-two miles long in a north-south direction and varies between two to three miles in width. There is little or no transverse gradient, with a small amount of natural slope prevailing from north to south. The bed of the Rio Grande River lies as much as six feet higher than adjacent lowlands in some localities. Gradients west of the river vary considerably. Transition slopes rising from the valley floor are generally abrupt, and slopes west of the transition vary from steep to gradual. East of the river abrupt transitions upward from the valley floor occur at a few locations, but generally gradients gradually increase from the lowlands to the foothills of the Sandia and Manzano Mountains, where very steep mountain slopes begin. Within the area treated there are four major topographical subdivisions as follows: (1)



Valley floor with low to flat gradients; (2) Highland areas with low to medium slopes; (3) Highland areas with steep slopes, and (4) Mountain areas with very steep slopes.

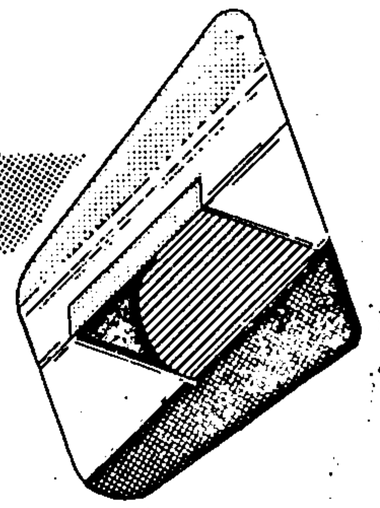
Absence of abundant vegetation or natural bodies of water in this region precludes formation of water vapor by transpiration or evaporation. Possibilities of rainfall are therefore dependent on overland movements of moist air masses which initiate over the Pacific Ocean and the Gulf of Mexico. Fifty-five years of weather records indicate that 45% of the annual precipitation in Albuquerque occurs in the form of rain during the July through September period. Thus, air mass movements during this period are of primary significance. During these months, Tropical Gulf Air invades this region in two separate movements. The most important is a direct movement in a northwesterly direction from the Gulf of Mexico. The second is a circuitous movement across Mexico and the southwestern part of the State. Invasions from the south or east are not materially obstructed by topographic barriers, and therefore can reach this area with enough moisture to produce precipitation. Air movement directly up the Rio Grande Valley has the least topographic interference, and therefore, is capable of producing the most severe storm. Invasions from the southwest must pass over the rugged Gila Wilderness, and therefore, the possibility of retaining moisture is less probable.



Occasionally, Polar Pacific air masses will arrive during this period, but generally this will occur during winter months.

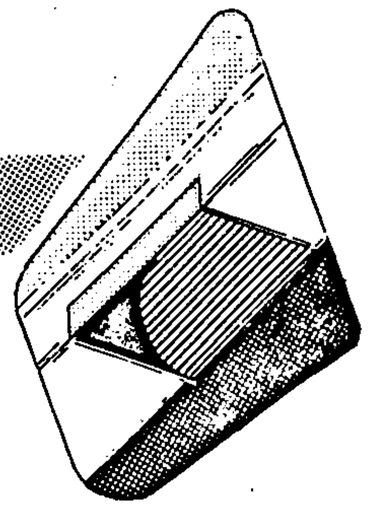
FLOOD POTENTIAL: Diverging opinions regarding the magnitude and effect of flooding possible in the Albuquerque area have seriously hampered flood control activities. Technical information available is such that most hydrologists and engineers are convinced that storms causing severe inundation can occur. Other groups insist that these forecasts are absurd and exaggerated. Those charged with responsibility for effecting reasonable flood protection are besieged by conflicting statements.

No documented records exist to substantiate the occurrence of floods of the magnitude probable for this area. Verbal and newspaper accounts do relate that storms of considerable proportions have occurred. Until comparatively recently, no detailed appraisals of floods were made when they occurred, since technicians trained for this type of evaluation were not available. There is evidence in existence which indicates that very severe flooding has occurred. Certain stratas of the alluvial plain east of the valley floor contain large boulders which were transported from mountain areas to their present position in recent geologic times. Most geologists concur that this transportation was accomplished



by unusually large flows of storm runoff. The usual markers which aid geologists in determining magnitudes and fixing dates of such occurrences are absent, since growth of plant and animal life generally deposited to form these indicators has been limited by the semi-arid climate. Thus, little information is available to indicate when storms took place or how large and widespread they may have been. However, facts in evidence disclose that floods of very large proportions must have occurred.

During the period of the City's most rapid growth, development proceeded without much regard for difficulties created by storm runoff. During this time, understaffed departments of the City were hard pressed to handle day to day problems, and consequently, little long range planning was accomplished. This fact, coupled with a prolonged period without severe rainstorms, caused undesirable methods for handling drainage to be used. Characteristics of ephemeral stream beds in this region are such that they are not very impressive to the observer when they are not flowing. Consequently, many of these natural drainage courses were disregarded or pre-empted. This resulted in the accumulation of heavy concentrations of storm runoff in streets not capable of transporting large flows. Consequently, overflowing often caused damage to adjacent properties and many streets became

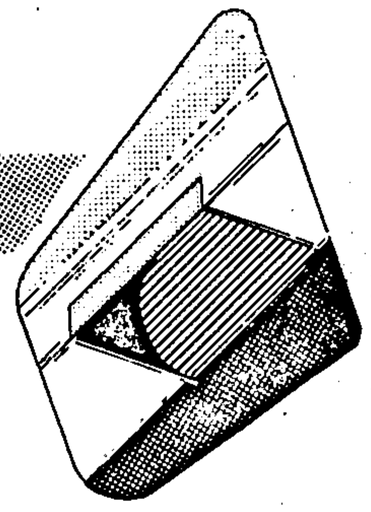


dangerous and impassible during rainstorms. As development progressed, the more rapid accumulation of runoff increased problems in both highland and lowland areas.

The total quantity and intensity of precipitation is a primary factor in flood potential. Causes of rainfall are not really pertinent to the problem, although meteorological phenomena do have an interesting relationship to the occurrences of storms. Therefore, a limited treatise is presented in Appendix A.

Projecting flood probabilities requires using data available from storms of record and applying scientific methods for purposes of analysis and formulation. It is improper to attempt to arrive at solutions by employing only empirical data. The degree of protection afforded must be determined in order to establish its relationship to cost and financing. While flood potential defined herein may appear somewhat nebulous, it must be emphasized that data derived is based upon valid scientific principles.

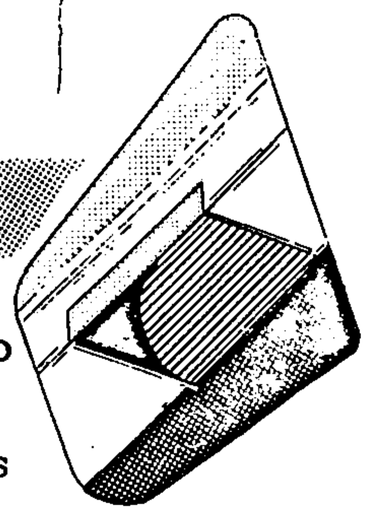
The accuracy of predicting flood potential is directly related to the availability of basic data. In this connection, establishment of a number of observation and measuring stations for operation during the July through September period will produce much valuable information. Information on temperature, relative humidity, amounts of precipitation, length, areal extent and area of concen-



tration of storms along with accurate measurements and records concerning times of accumulation, rates, and total times of run-off will be invaluable in determining the amount and extent of flood potential. Establishing these facilities would benefit the Albuquerque area and the entire Rocky Mountain region. Stations which accumulate climatic data can be in more or less arbitrary locations. Measurements of runoff should be made in lined channels or at structures since permeable material composing the floor of many channels has a tendency to become fluid during periods of flow. This fact renders slope-area measurements in unlined channels inaccurate. Suggested locations for measuring stations are shown in the map section of this report.

Detailed analyses of drainage problems and design of drainage facilities are complicated by the fact that no accurate comprehensive topographic maps are available. Detailed flood and drainage control require such maps. A systematic mapping program should be initiated as soon as funds can be made available.

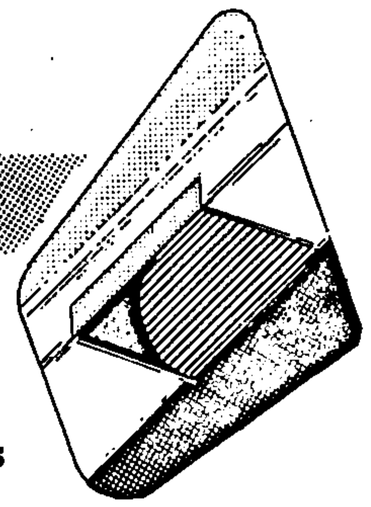
EFFECTS OF FLOODS: Storm runoff produces two categories of problems. Peak rates of flow cause damage in highland areas while the total volume of runoff may produce inundation in lowlands. The overflow of both natural and man-made carriers traversing the highlands creates the major problems in these areas. Since



there are no natural water courses crossing the valley floor to the river, the total quantity of flow into these areas determines the extent of damage. Since the river bed is higher than much of the adjacent land, large areas can be flooded before any flow into the river can occur.

Flood hazards have precluded the development of large tracts of land in the metropolitan area. Investors, insurers and property owners are reluctant to promote or participate in developments or improvements in areas subject to inundation. In many instances, the damage to both public and private property that can result from a single storm will justify the expenditure of large sums for flood protection. Thus, protecting public and private property and providing means of controlling storm runoff is currently a major City problem.

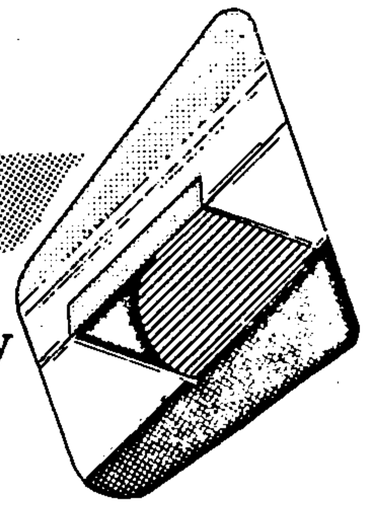
EFFORTS TO ALLEVIATE FLOODING: Early attempts to minimize flooding were based largely upon empirical considerations. Areas subject to the most severe inundation were treated first. Since many natural drainage courses had been eliminated, considerable difficulty was encountered in locating carriers where storm sewer systems could be discharged. Some conditions required excessive lengths of storm sewer lines to effect drainage in relatively small local areas. Major carrying facilities had to



be devised and constructed before any overall plans were possible, therefore, early efforts were confined to providing local systems scattered throughout the developed area. These projects contributed little toward a solution of the overall problem. Certain local areas were given some relief, but no comprehensive plans were developed for handling major concentrations of runoff.

These early programs established the need for maintaining major natural drainage courses to provide outlets for storm sewers. A study was performed in 1950 and many natural channels were designated to be incorporated into future developments as drainage easements. Some policies were also established which fixed the physical areas to be applied to each system of channels. This was the first step attempted toward master planning for drainage.

During this same period, the Corps of Engineers was authorized by the United States Congress to study and make recommendations regarding requirements necessary to effect flood protection along the main stem and tributaries of the Rio Grande in the vicinity of the City. The Corps of Engineers Report ("Review Report on Survey for Flood Control, Rio Grande and Tributaries, Albuquerque, New Mexico and Vicinity - 1953") recommended the construction of the North and South Diversion Channels as a federal project

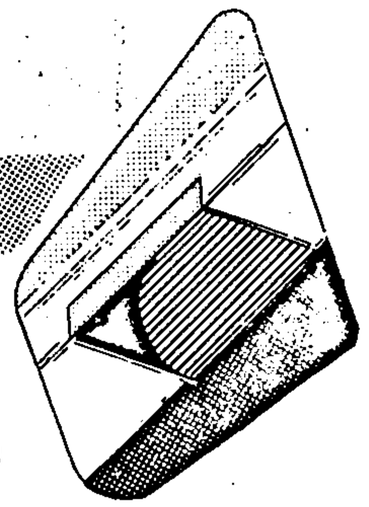


provided that funds for rights-of-way, severence damages, utility and street relocations were provided by local interests.

The City and the County initiated steps, conforming with State Statutes, to establish a Conservancy District for the purpose of acting as local sponsor for the federal project and to provide additional flood protection.

This combined local and federal effort provided a means of developing and financing plans which were to establish a primary system for controlling and handling storm runoff. Thus, the Corps of Engineers Report and the Master Plan Report for the Sandia Conservancy District presented a means of effecting primary flood control. With these primary facilities defined, engineers were able to begin a methodical analysis of requirements for secondary and branch facilities.

No construction of facilities was accomplished through the Sandia Conservancy District, since protests from property owners regarding the benefits assessed to various sectors of the District ultimately caused it to be dissolved. The source of funds required for participation in the Corps of Engineers project was, therefore, eliminated. The Federal authorization for the construction of these flood control channels expires in November, 1963. If the project is to be salvaged, a local agency must assume responsibility for



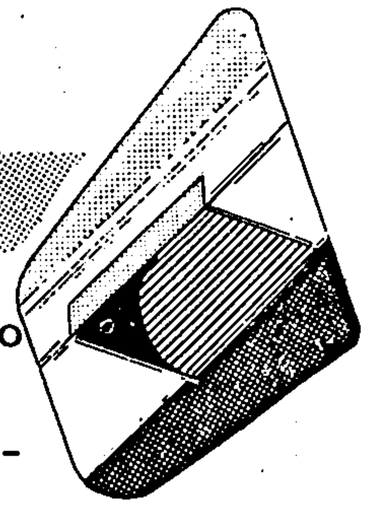
participation.

After dissolution of the Sandia Conservancy District, efforts were made to find other means to build projects recommended in its master plan. Some construction was done by the City with general obligation bond funds, and a few facilities were built by incorporating them into other public works projects. A complete primary system cannot be provided by this method, since constitutional limitations on general obligation indebtedness precludes financing the entire project.

The limited development of the primary system has precluded construction of many branch facilities that had been contemplated; however, the establishment of City policies, together with diligent planning for controlling storm runoff has insured the availability of most rights-of-way and easements which are required. Some feasible temporary control measures have been accomplished, and, where primary facilities exist to accommodate discharge, a few local storm sewer systems consistent with overall planning have been built.

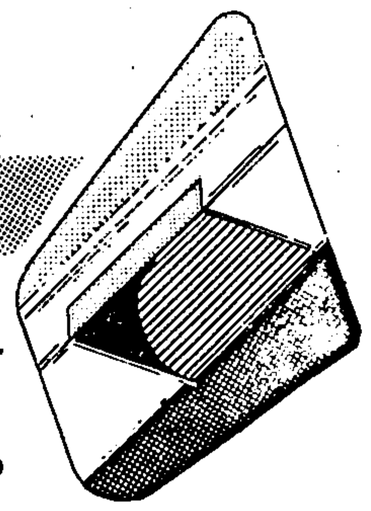
DRAINAGE FACILITIES

EXISTING FACILITIES: Within the area to be treated, arroyos, drainage channels, streets, storm sewers, pump stations and ponding areas constitute existing drainage facilities. Arroyos



are natural collectors and carriers which generally discharge onto the valley floor some distance from the river. Most of the drainage channels have been constructed in existing arroyo beds or they replace arroyos that have been eliminated. These channels generally empty into other arroyos, none of which have direct access for discharge into the river. Some runoff occasionally overflows into Middle Rio Grande Conservancy District (MRGCD) drains and in this manner ultimately reaches the river. Streets and storm sewers discharge into arroyos or drainage channels and thus their use for drainage purposes is limited to local service. A few pump stations have been constructed to provide outlets to the river for valley storm sewer systems. These are the first facilities within the system which actually provide the proper terminus for storm runoff by discharging it to the river without delay. Some existing MRGCD drains do provide a measure of protection, although even those that can be utilized will require major reconstruction. Some ponding areas exist, but flood waters have no means to escape except by infiltration or evaporation.

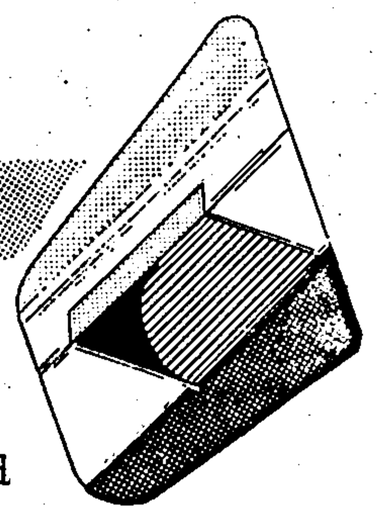
POSSIBLE FACILITIES: Plans to collect, transport and dispose of storm runoff is the most important aspect of this study. This is accomplished primarily by using the same type of facilities as those which exist. This plan establishes the general location of



areas to be serviced by installations together with information pertinent to efficient and economical use of drainage facilities. No actual design was performed, but all data and criteria required for basic design considerations is set forth. Therefore, the plan is schematically comprehensive and flexible, since necessary information is provided without imposing fixed requirements that can impair land planning and use.

Land planning and grading have marked effects on requirements for drainage facilities. Location, routing and final profiles of residential and arterial streets determines their ultimate use for drainage purposes. The order and type of development has considerable effect on the character and requirements of drainage carriers. Land planning must be carefully co-ordinated with drainage requirements if maximum benefits are to be realized.

Streets, storm sewers, and drainage channels comprise the main collectors and vehicles for storm runoff within the complex. Ponding areas serve only as relief facilities. Streets act as major collectors and their limited use as carriers must be considered. Storm sewers intercept and discharge runoff from streets. Drainage channels serve primarily to collect and transport flows discharged from storm sewers and streets, thus they are the major carriers of storm drainage. Ponding areas are occasionally

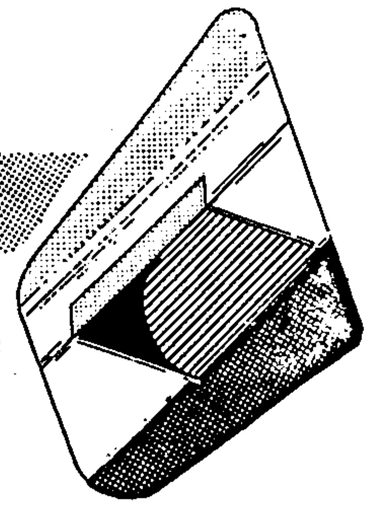


required to relieve carrier facilities when flows exceed capacities.

The primary function of a street is to efficiently collect and move traffic from one point to another in safety and comfort. However, they also collect storm runoff. If an area is developed, rainfall in excess of that which is absorbed, ponded and evaporated will ultimately be deposited in a street. Portland cement concrete, asphaltic concrete or a combination of these are the principle materials used in street construction. Since curbs are required, a channel is created when a street is constructed. These possess good hydraulic characteristics, thus their use as carriers is enhanced.

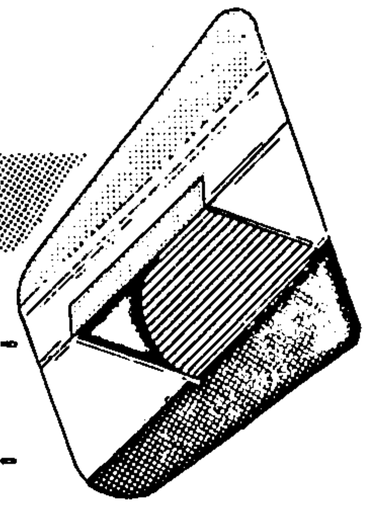
The primary purpose of a storm sewer is to intercept and discharge runoff from streets. Occasionally, development will occur which necessitates using storm sewers as major interceptors for branch systems. This is extremely costly and it must be avoided whenever possible. The purpose of storm sewers is to prevent overflowing of or ponding in streets. Complete draining of streets except at predetermined intervals is uneconomical. Storm sewer construction is the most expensive treatment in the complex; and, therefore, this treatment must be used only when other facilities are not available.

Storm sewer systems consist of: catch basins (inlets) to intercept flows; branch lines to deliver flows to mainlines; manholes to



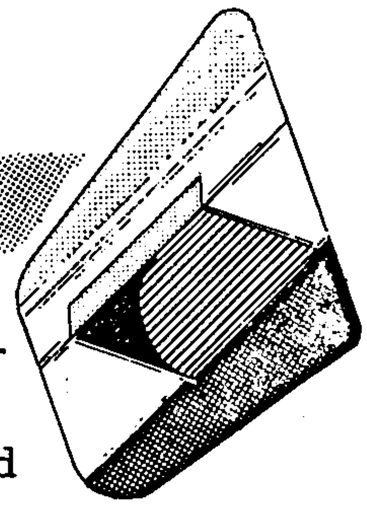
provide for junctions with branch and mainlines and to accommodate changes in alignment of mainlines; major transmission lines; occasional junction boxes to turn large flows in lines or to deliver large branch line flows to trunk lines; and discharge or outlet facilities. Catch basins are reinforced concrete boxes with steel inlet grates. Branch and main lines are generally circular conduits or pipes made from plain or reinforced concrete or steel. Manholes and junction boxes are constructed of concrete block and/or reinforced concrete, with steel or cast iron covers to facilitate maintenance and inspection. Outlet or discharge facilities are constructed of reinforced concrete and/or stone riprap. Protective devices are occasionally installed at outlets to preclude children from entering and playing in the open ends of pipe. While these devices are required for public safety, they often present functional problems.

Since drainage channels are the main carriers within the system, their proper location, design and construction is particularly significant. A planned system of drainage ditches will produce the most efficient and economical scheme for handling storm runoff. Locating channels in or near natural drainage courses allows beneficial use of land which is often difficult and unwise to develop because of storm drainage problems. Construction of channels is relatively inexpensive when compared to the cost of other facilities. If over



use of ditch systems is avoided, their effect on overall land development will be minimal. However, some difficulty in developing individual tracts may be anticipated. In certain portions of the complex, runoff from large areas is collected in undefined water courses. Developing these areas demands extensive use of storm sewers, unless artificial channels can be provided. Such channels are indicated in all feasible locations.

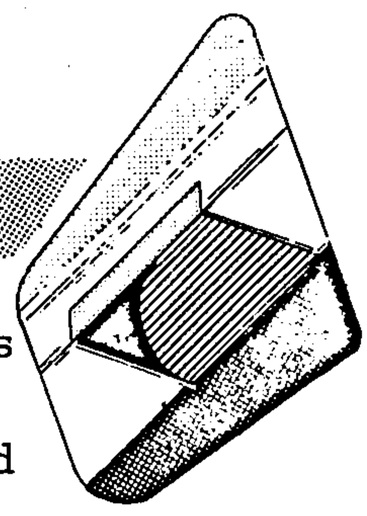
Drainage channels can be constructed with or without linings, and cross sections can vary widely depending upon design requirements and economic considerations. Economics will generally govern the choice of whether a channel is to be lined or not. Easement requirements, cost of construction and maintenance costs must all be evaluated. Cross section requirements involve several design considerations as well as economic appraisal. Quantities of flow and energy-momentum conditions fix hydraulic design requirements. Economic appraisals consist of comparing construction and right-of-way costs. Energy-momentum considerations in channels are most critical at points where convergence or abrupt changes in alignment or gradient occur. An infinite variety of channel sections are possible for a given quantity of flow. A deep, narrow section is hydraulically most efficient and requires less right-of-way. Wide, shallow sections are less costly to construct, but



increase right-of-way requirements. Generally, the most efficient section will be indicated when channels are paved or lined because the required quantity of lining will be minimum. Wide, shallow cross sections are easily and economically constructed with standard mass-excavation equipment. Most earth channels will consist of this type of section. Widths of easements and sectional treatment have not been indicated in the plan in order to allow private owners the widest possible latitude in land development which is consistent with the public interest. Any combination of easement width and cross section will suffice as long as it will safely transport the indicated flow.

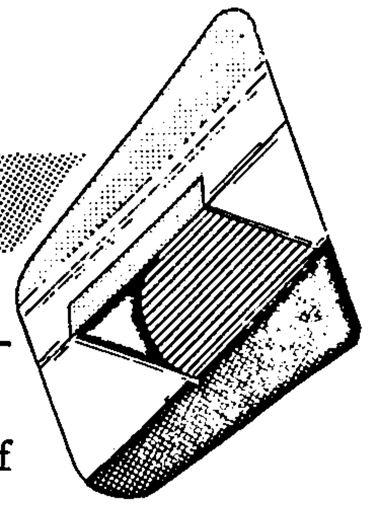
Drainage structures are required at intersections of channels and arterial streets in order to facilitate the movement of traffic during periods when channels are flowing together with preventing excessive damage to streets at these crossings. Increasing the use of channels within the complex will create a need for added proportions of drainage structures.

Materials commonly used in constructing drainage structures are: reinforced concrete or corrugated steel pipe; corrugated steel multiplate arches set on reinforced concrete foundations and floors; reinforced concrete box culverts; and corrugated steel multiplate pipe arches. Entrance and outlet treatment at drainage struc-



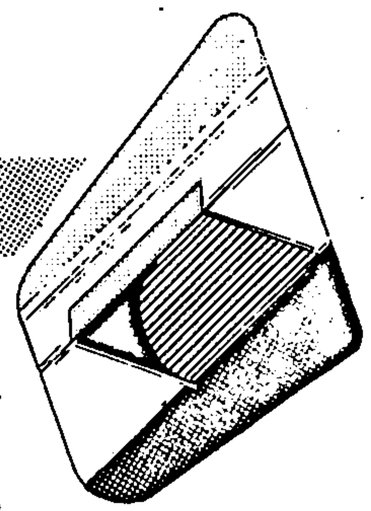
tures is required. This generally consists of paving channel walls and floors with reinforced concrete or wire enclosed and dumped riprap adjacent to the structure. A combination of these treatments is frequently employed. Concrete slope paving of street embankments is usually necessary to protect both conduits and street subgrades. Riprap protection at both entrance and outlet ends is desirable, although it can be eliminated on the upstream side in many instances. Scouring which occurs on the discharge side of structures usually dictates that riprap be placed downstream.

Ponding areas can be used to a limited extent within the complex. This application should be confined to areas where no other methods of treatment are feasible. Either temporary or permanent ponding areas may be utilized. Temporary ponds are drained by controlled gravity discharge or by pumping. Permanent ponds dissipate their content by infiltration and evaporation. Temporary ponding is usually effective, although large areas of land are required. Ponding areas developed in conjunction with large recreation facilities may prove successful. In order to make ultimate recreational use of this type of installation, a permanent pool can be supplemented with well water. Draining ponding areas into existing irrigation ditches of the MRGCD at a controlled rate,



has considerable merit. This provides the most economical method of draining these ponds and will not impair the primary use of the ditches. Permanent ponds with no provision to dispose of their content are undesirable. Although evaporation and infiltration would not permit water to stand for prolonged periods, hazards to health and safety preclude this application. Economic appraisals reveal that some ponding is indicated, but this method has many disadvantages unless some use in conjunction with recreation facilities can be devised.

The foregoing descriptions of facilities available for use within the drainage complex sets forth the large measure of flexibility that is possible. Each installation can be designed to comply with almost any need, but economic considerations will generally govern selections. Therefore, the criteria developed has broad and varied application to fulfill requirements of the master plan area. The discussions which follow will deal with these possibilities.

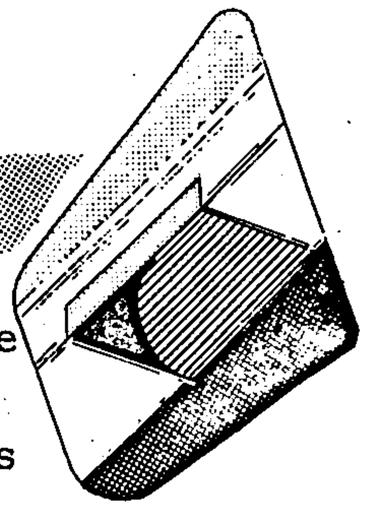


DEVELOPMENT OF THE PLAN

DERIVATION OF BASIC CRITERIA - RAINFALL AND RUN-OFF: The initial and one of the most significant requirements for developing the plan is to establish a method for estimating amounts of rainfall and runoff that can be anticipated. Statistical data available for storms of record together with research relating experienced rainfall to storm frequencies provide the basis for predictions. Data presented to estimate rainfall and runoff were derived by combining information regarding storms of record, regional rainfall studies and interpretations and official precipitation records.

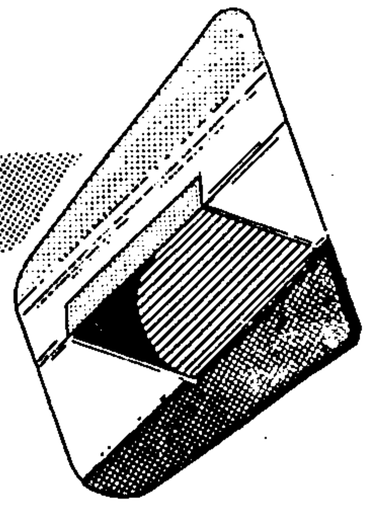
Storms of record provide valuable data regarding peak rates and total quantities of flow which result from a given storm. While records of this nature for the Albuquerque area are limited and incomplete, available information permits making reasonably accurate predictions. Estimating the quantities of ponding and sedimentation possible in lowland areas and the anticipated peak flows in the highlands is made possible by analyzing these records.

Although design of most drainage facilities is based upon volumes of flow much smaller than anticipated maximums, the possibility of extraordinary quantities of runoff must be considered. For this purpose, the Corps of Engineers selected the Las Cruces,



New Mexico storm of August 29-30, 1935, as the most intense storm likely to occur in this area. After correcting for differences in elevation and topography, the storm was transposed to Albuquerque. Various patterns and concentrations were plotted, and peak flows and volumes of discharge were computed for all major drainage areas. From this study, anticipated peak rates of flow and design requirements were established.

Intensity and duration and the manner in which they are related require explicit definition before being applied to runoff computations. The most common errors made result from misunderstanding the definition of the intensity-duration relationship. The following hypothetical situation will illustrate the application of the various components. For any single storm, lines connecting points of equal rainfall form closed figures when they are plotted on a map. The arrangement of this group of closed lines forms a pattern in which peripheral lines delineate the lowest quantities of rainfall, with incremental increases toward the center of the storm. This results in what is termed an isohyetal map of the storm. Individual lines forming the closed figures are called isohyetes. For example, assume a 24 hour storm six miles in diameter which has isohyetes formed by circles with radii of one, two and three miles. Rainfall at the center is 3 inches and adjacent isohyetes

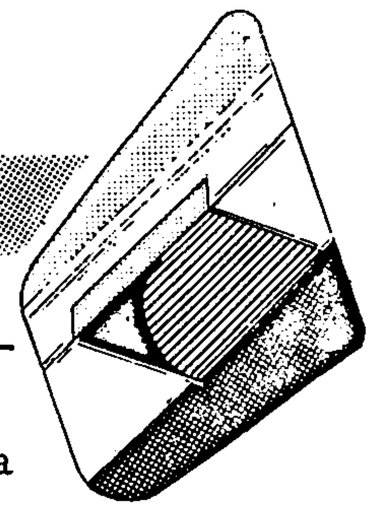


represent rainfalls of 2, 1 and 0 inches respectively. In this region, 90% of 24 hour rainfall will usually occur in some two hour period. From the preceding example, information is obtained as follows: (1) The storm has an areal extent of 28.26 square miles or 18,086 acres (the area of a circle 6 miles in diameter); (2) the average rainfall is 1.06 inches based upon an area-mean calculation; (3) the total rainfall is 19,171 acre-inches (18,086 acres x 1.06 inches); and (4) the average intensity is 0.49 inches per hour if 90% of the 24 hour total rainfall occurs in a period of 2 hours (1.06 inches x 90%/2 hours). Total runoff is obviously 19,171 acre-inches less that quantity which is absorbed, ponded and evaporated.

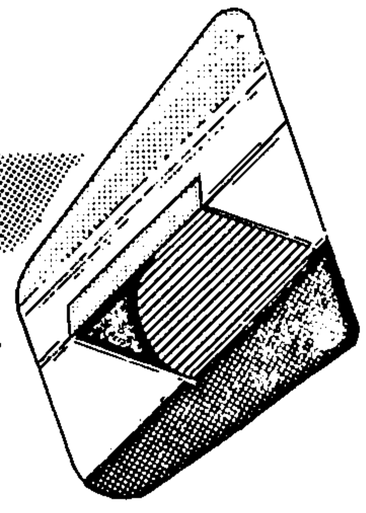
Methods of determining peak rates of runoff are not so apparent, since no consideration has been given to the time required for flows to accumulate. To calculate peak rates of flow, the following information is required: (1) the average intensity of rainfall; (2) the time required for runoff to reach the point of measurements from the most distant point in the area being considered.

When duration or time of accumulation is evaluated for an area, the following relationships to intensity are established:

(1) if the contributing area for a stream is smaller than the areal extent of the storm, a higher average intensity will pre-

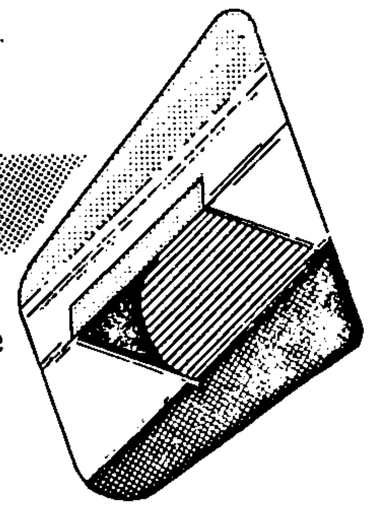


vail, and the time required for accumulation of runoff in the drainage area will be shorter; and (2) if the contributing area for a stream is larger than the areal extent of the storm, a lower average intensity will accompany a longer time of accumulation. This points up the relativity of the intensity and the duration of storms. Yarnell (Yarnell, D. L. - "Rainfall Intensity-Frequency Data" - U.S. Dept. of Agriculture Miscellaneous Publication 204. 1935.) performed exhaustive studies of storm patterns, frequencies, intensities of rainfall and times of accumulation of runoff. From this research, Yarnell concluded that there was a definite relationship between intensity, duration and storm patterns for various areas of the country and further, he was able to derive a standard expression relating intensity and duration. This equation contains two constants which apply to given areas in which storm patterns generally assume somewhat fixed characteristics. Storm frequencies are predictable based upon mathematical probability and Yarnell found that adjusting the constants would also reflect frequency of reoccurrence of given magnitudes of storms. The net result of Yarnell's endeavors is that equations can be developed which express intensity-duration relationships for given storm frequencies in any area where adequate local records are available. Since this provides a means of relating time to protection



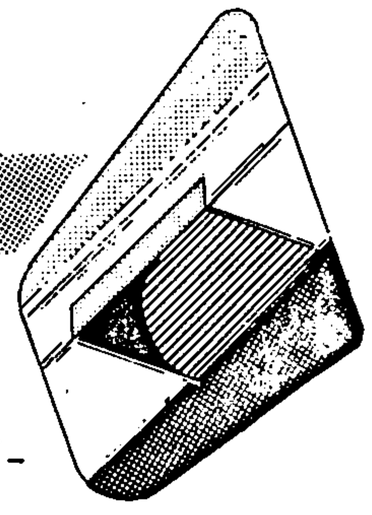
afforded, it is of great value in considerations involving ammortization and financing.

Intensity-duration-frequency relationships were developed by obtaining 24 hour rainfall records from the U. S. Weather Bureau Station at the Albuquerque Airport and plotting rates of recurrence of storms of given magnitudes. Data taken from the resulting chart were then applied to standard formulae which Yarnell developed for the southwestern United States, and intensity-duration-frequency equations for this area were derived. These relationships are set forth graphically in Appendix A. Three charts were prepared. These are all predicated upon the same basic methods and formulae, but each set of curves was constructed using different physical data. Chart 1 is based upon the latest aggregate of information from the U, S. Weather Bureau at the Albuquerque Airport and is the chart prepared for the purposes of this study. Chart 1A was prepared in 1954 for use in connection with the development of the master plan for the Sandia Conservancy District using data from Los Lunas, Bernalillo and Albuquerque. Chart 1B is plotted using Yarnell's 1935 standard expressions for the entire southwestern United States. Variations indicated by these three presentations displays the necessity for constant revision based upon the latest aggregate of data.



Table, charts and formulae used for estimating runoff are also presented in Appendix A. Chart 2 contains peak runoff-area curves for various frequencies of storms. These curves are based upon fully developed basic contributing areas one-half mile square with an average gradient of 1.414%, a runoff coefficient of 0.45, and a time of accumulation of 20 minutes. The chart is predicated upon development which will yield maximum peak rates of flow. It must be judiciously applied or modified to reflect actual development conditions. Times of accumulation vary widely with the shape, average gradient and pattern of flow within an area. Runoff coefficients require modification to reflect variations in surface characteristics. Thus, this graph is valuable for making preliminary evaluations, but it should never be used for design purposes.

Peak runoff for specific areas may be calculated by any of several methods. The Corps of Engineers' curve for obtaining maximum probable rates of runoff appears on Chart 3 of Appendix A. This graph was derived by transposing the Las Cruces, New Mexico storm of 1935 to the Albuquerque area and gives the maximum probable peak rate of flow for areas of various sizes. For the purposes of this report, calculations of peak rates of runoff exercise the "Rational Formula" which is the product of area,

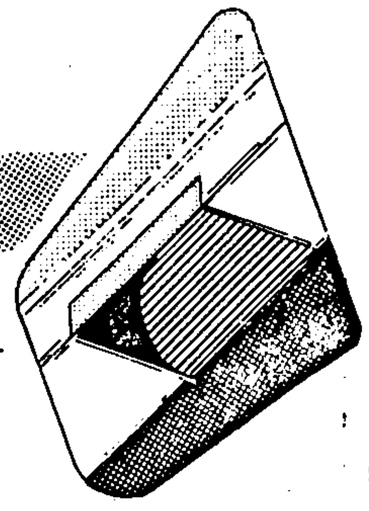


average intensity and runoff coefficient.

DERIVATION OF BASIC CRITERIA - HYDRAULICS OF DRAINAGE FACILITIES: Cross-section design, gradient and surface composition govern water carrying capacities of streets. Manning's formula, described in Appendix A, is used to compute these capacities. Table IV of Appendix A gives capacities of streets with various widths, cross sections, and gradients. The amounts of flood water which may be transported by a street must be limited by the capacity of its least efficient reach.

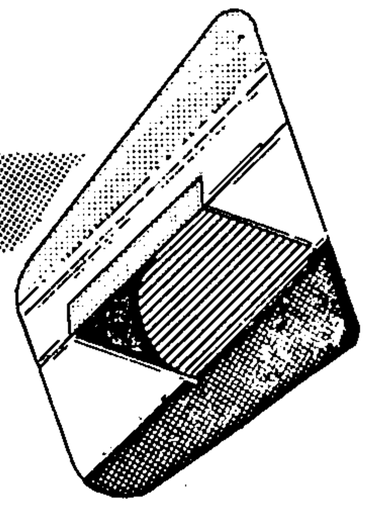
Storm sewers are gravity systems and are sized by using Manning's Formula to compute capacities. Table V in Appendix A shows average inlet capacities. Table VI, Appendix A gives carrying capacities of concrete and corrugated metal pipe laid on various slopes.

Manning's formula is also used for designing drainage channel capacities. Table VII in Appendix A indicates carrying capacities of channels with different slopes, surface treatment and cross sections. This table generally will apply to ditches for highland areas. Table VIII in Appendix A sets forth carrying capacities of ditches with low to moderate slopes, and is divided into two parts as follows: (1) lines and unlined channels with varying widths and fixed depths; and (2) a standard cross section for lined



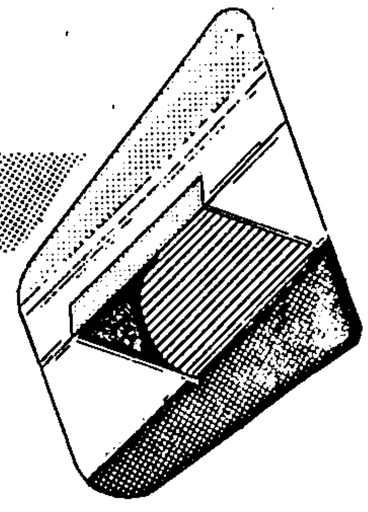
and unlined ditches with varying depths. This table may be used to evaluate ditch requirements in lowland areas. Table IX in Appendix A contains basic data that will be helpful in analyzing channel requirements for major flood control facilities. No energy-momentum considerations are reflected in any of these tables, since these evaluations require refined data and detailed design analyses. A brief treatise of the basic concepts applicable to energy and momentum of flows is presented in Appendix A.

Entrance and discharge conditions at drainage structures are of primary significance. Treatment of upstream and downstream portions of the channel in the proximity of the structure is required to provide favorable entrance and discharge conditions and to protect the structure, the channel and the street. Hydraulic drop occurs naturally when a conduit discharges and its tendency to produce scouring requires careful downstream treatment. Placing a riprap blanket on the channel sides and floor immediately below the concrete slope paving and apron serves to reduce turbulence and prevent excessive scouring. Changing channel gradient is occasionally necessary in order to build structures where natural channels are shallow. This must be accomplished by transitioning slopes in the channel in a manner that will prevent serious alterations of flow conditions. Manning's formula applies to flows in the



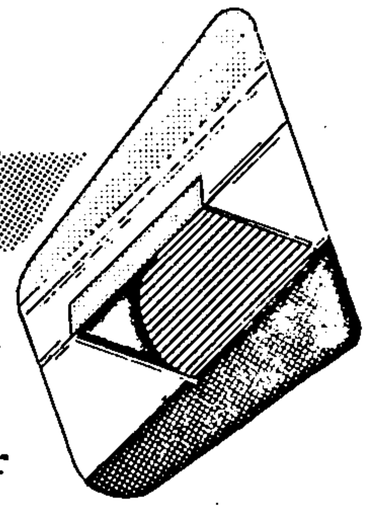
conduit, although capacity is usually controlled by entrance or discharge conditions. Complete channel control with the conduit passing design flows without submergence is the best condition. If design flows through the channel submerge the inlet of the conduit (entrance controlled flow), U. S. Bureau of Public Roads Standards are used to determine quantities of discharge. Entrance conditions of a structure are improved by increasing the area of the inlet by cutting the conduit on the skew formed by side slopes of street embankments. This also produces a more pleasing appearance. Flow conditions within the channel both upstream and downstream as well as within the conduit must be evaluated in order to provide proper controls.

Capacities of ponding areas are measured in acre-feet. The volume that can be contained in a pond is defined as the storage available below the lowest invert of installations that discharge into it. In lowland areas where the ground water level is high, depth of storage is limited. Large surface areas tend to evaporate rapidly. The average evaporation rates measured for this area during the months of July, August and September are 10.69 inches, 8.34 inches and 7.22 inches respectively. Rates of infiltration vary considerably for different soils but most surface material in this area has fairly high permeability. Tests indicate that rates



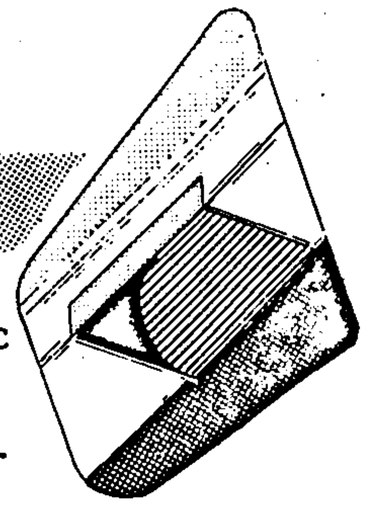
of infiltration are directly proportional to static head created by the depth of the pond. Thus, increasing depths causes higher rates of infiltration. It has also been observed that freshly excavated material has higher permeability than undisturbed surface material. This is due to the decrease in porosity caused by fine materials deposited by wind or water. In order to maintain high rates of infiltration, it is necessary to keep these fines from closing the natural pores of the soil. When sedimentation is limited, this can be overcome by furrowing. Large amounts of fine sediment must be removed in order to restore porosity. Little data on rates of infiltration are available; however, the State Engineer's Office has determined that about 30% of water used for irrigation in valley areas will return to underground storage. To evaluate the depletion of storage in a ponding area, evaporation and infiltration must both be considered. Evaporation is enhanced by increasing the surface area of the pond. Infiltration is increased proportionally with the depth of storage as long as high permeability of the soil in contact with the water is maintained. Detail permeability tests of the soil in potential ponding areas must precede selection of locations.

Sedimentation has pronounced effects on hydraulic capacities and characteristics of installations which handle storm runoff. The Corps of Engineers' Report contains estimates of the amount of an-

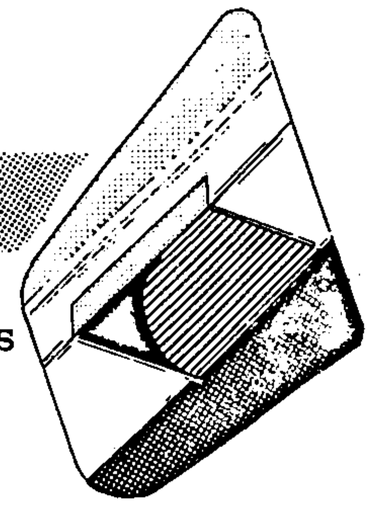


nual sedimentation to be expected for several of the major drainage areas. These estimates are reiterated in the master plan for Sandia Conservancy District as follows: Bear Canyon, 7.0 acre-feet; Vineyard Arroyo, 1.4 acre-feet; Hahn Arroyo, 4.0 acre-feet; Embudo Arroyo, 20.0 acre-feet; Campus Wash, 4.4 acre-feet; and Tijeras Arroyo, 26 acre-feet. This information has value in determining the quantity of sedimentation probable in lowland areas at the terminus of each arroyo. However, some problems are encountered at intermittent points in individual carriers such as streets, drainage structures, and storm sewer systems. If these facilities have relatively high gradients, sediment deposition is not critical. However, when low gradients persist for any distance, sedimentation impairs hydraulic function. The quantity of sediment contained in runoff from fully developed areas is relative small. Storm water which collects in undeveloped or partially developed areas generally accumulates substantial amounts of sediment. This is transported to points where low velocities permit deposition. It appears that velocities under 2.0 feet per second result in deposition at the rate of approximately 0.10 acre feet per year per square mile of undeveloped area.

CLASSIFICATION OF TYPICAL SUB - AREAS: In order to utilize criteria which has been derived, it is necessary to classify



and define typical areas for application. Development, topographic features and the presence of existing facilities are the factors affecting area classification. Developed, undeveloped and large planned or platted areas are the classifications pertaining to status of development. Developed areas include those which have an average population density that warrants consideration as a metropolitan area, while undeveloped areas have scattered or no population. Those areas such as the planned portion of the Elena Gallegos Grant and platted subdivisions east of Juan Tabo Boulevard fall into the classification of large planned or platted areas. Topographic features affecting drainage characteristics are as follows: (1) areas having natural gradient which render them easily drained; and (2) areas where existing low slopes create ponding or very slow movement of overland flows. Since most highland areas have natural gradient and most valley areas have little slope, the terms "Heights" and "Valley" will be used to describe these respective areas. Existing facilities may be present in some areas herein-before defined and these usually require some special consideration. Areas subject to natural ponding, flooding from extraneous runoff, and containing artificial barriers such as local irrigation and drainage systems require special treatment. Using these classifications, relationships with regard to drainage characteristics were de-

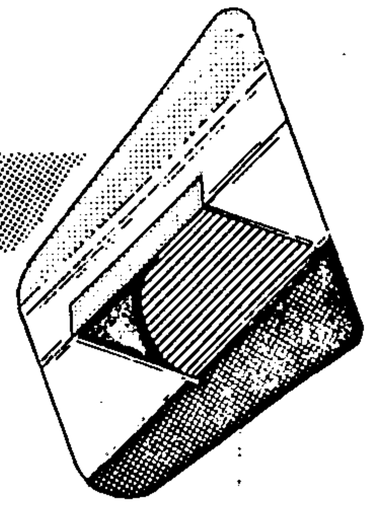


veloped and the master plan area was divided into categories as follows:

1. Undeveloped "Heights" areas.
2. Developed "Heights" areas.
3. Undeveloped "Valley" areas.
4. Developed "Valley" areas.
5. Large planned or platted "Heights" areas.
6. Areas requiring special treatment.

Establishing these categories provides a basis for treating the master plan area. Using developed criteria, a method for analyzing each of these has been devised.

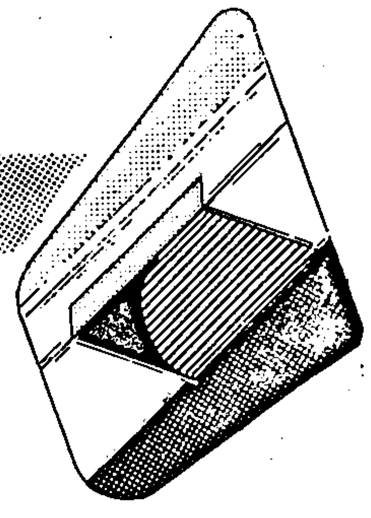
APPLICATION OF BASIC CRITERIA - GENERAL: Location, orientation and engineering design of streets are of primary significance to effect an economical plan for handling storm runoff. Streets located and oriented to receive and transport drainage without extensive physical alteration to the surrounding area will render the best service. In some situations, this recourse to pure function cannot be accomplished and remain consistent with good planning procedures. However, in most instances, economic factors will dictate that a functional approach be used. Controlling the distribution of flows is also extremely important. Natural topographic features generally tend to accumulate large quantities of



runoff at specific locations. To prevent this natural accumulation, some alterations are required. Distributing flows into several available facilities will often preclude excessive accumulation, and will also provide the most economical overall treatment. These requirements are easily met if a preliminary drainage plat is prepared. This plat consists of a reproduction of the preliminary land plan with topography and drainage data superimposed upon it. The following data are required: (1) areas from both inside and outside the planned subdivision that contribute runoff to each street; (2) the routing of flows within the subdivision; and (3) cumulative totals of areas contributing runoff to each street intersection. This allows a thorough study of storm drainage to be made during preliminary planning stages, and facilities required within or near the development can be easily determined.

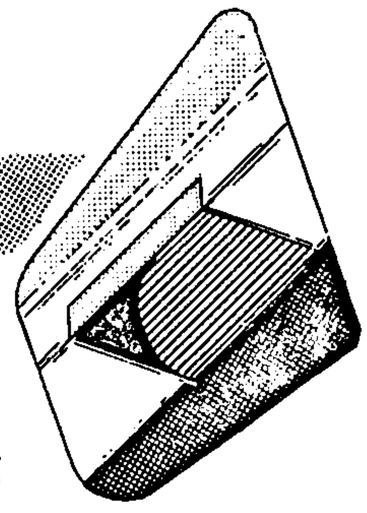
Providing necessary rights-of-way and easements are mandatory in implementing a successful and economical plan for drainage. Careful planning must be exercised to determine the best alignment and location of a facility to insure good overall function without adversely affecting land use. The exclusion of a right-of-way or easement necessary for any major drainage requirement seriously impairs the overall function of the system.

Although one of the primary functions of an easement is to



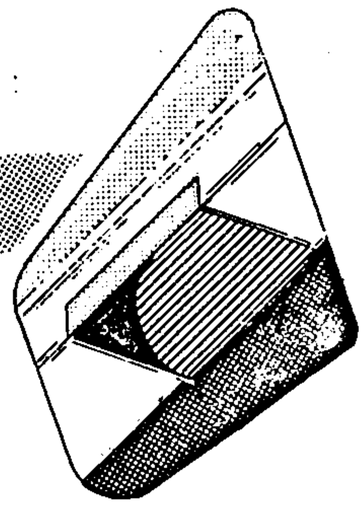
permit ease of maintenance for facilities that are situated within it, development that occurs around these easements is generally such that no ready access is available except at street intersections. Heavy equipment access is necessary at intervals not to exceed one mile. This can be provided by widening easements at intersections with arterial streets.

Basic criteria which has been derived is applied for the purposes of planning only. No detailed design work has been performed. This master plan does not reflect any actual locations, design capacities, final alignments or other design requirements; since these depend upon final development. The plan shows approximate positions where facilities are required, approximate areas contributing to them, and methods of disposing of flows from various local installations. Although methods for handling storm runoff in the entire master plan area are indicated, it is unlikely that these exact applications will suffice, because many variations are possible during planning and development. The plan is, therefore, schematic in order to provide a maximum of flexibility necessary to develop the best land use. Each drainage problem usually has several solutions which permit a choice of the facility to be used. Only refined considerations employed at the time of design will reveal the best solution and the choice of the proper



facility.

Streets and storm sewers generally handle flows resulting from relatively small contributing areas and channels usually accommodate drainage accumulated from larger areas. Therefore, even though no specific installation is indicated, many requirements are fixed. Drainage areas applied to facilities indicated on the maps are natural contributing areas along with those areas expected to contribute after development. Generally; these areas are approximately equal. However, in some instances, they differ considerably. This indicates that upstream diversion from or into the facility is proposed. These conditions must be carefully analyzed before design work can proceed. The upstream treatment indicated will produce the most economical results. If the proposed upstream treatment has not been completed, designs must be predicated on using the largest contributing area indicated. This is not practical, since it will ultimately result in overdesign. Several diversions are indicated on the maps. If these diversions are not in effect before downstream construction is initiated, substantial cost increases can be anticipated. While several ponding areas are indicated and defined, it should be emphasized that ponding is not the only method for handling storm runoff at these points. This method is feasible and should be seriously considered. Pond-



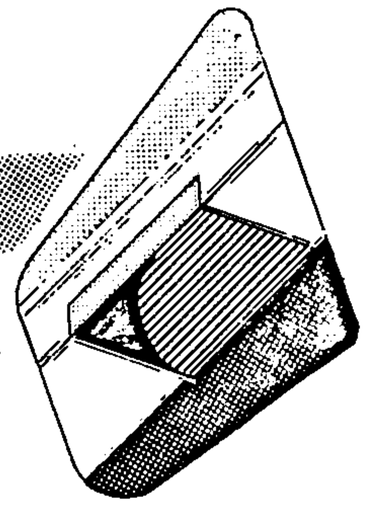
ing, unless it can be incorporated into a multiple use scheme, is not always the most practical solution, even though it may be the most economical. As suggested hereinbefore, such facilities have been successfully combined with recreational development. This type of multiple use may have merit in this area.

APPLICATION OF BASIC CRITERIA - DRAINAGE FACILITIES:

GENERAL: Defining the appropriate function of each facility used to handle storm runoff is pertinent to implementing an efficient system. Streets, storm sewers, drainage channels and ponding areas have individual characteristics which render each of them adaptable to specific uses within the complex.

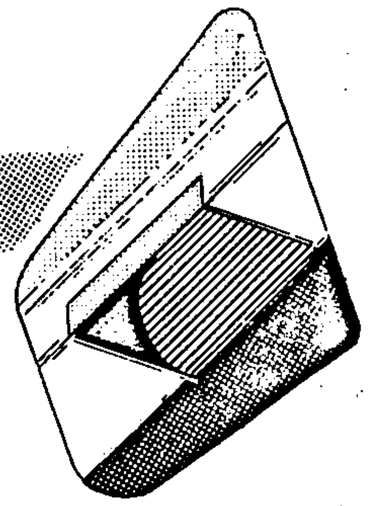
APPLICATION OF BASIC CRITERIA - DRAINAGE FACILITIES:

STREETS: Streets are the primary collectors for storm runoff in developed areas and economic evaluations indicate that they must also be used as carriers. Investigation indicates that where appreciable natural gradients exist, about 85% of the runoff from a fully developed 160 acre parcel can be safely carried in streets that are properly designed and oriented. In an area with an average gradient of approximately 1.5% and with streets oriented to have a gradient of about 1.0%, a general rule may be adopted. Runoff accompanying a 25 year frequency storm in an 80 acre developed area can be safely handled on an arterial street designed using present cross section standards. On the same basis, a residential

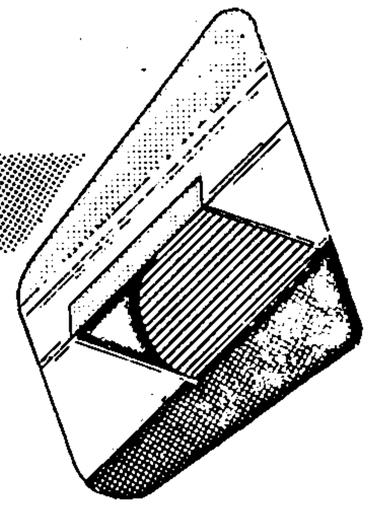


or collector street can carry runoff from 40 acres. This criteria coincides with established planning and traffic policies of the City. Runoff and hydraulic capacity of streets are fairly balanced for the lower range of gradients up to 1.5%; but, for higher slopes, street capacities exceed rates of runoff. Therefore, each situation must be investigated at the time of design. Evaluations of street carrying capacities are also required for each condition, so that the area served and/or the cross section design can be adjusted to meet requirements.

Residential streets together with adjacent front yards can be designed to carry the 100 year frequency flood by fixing the finished floor elevation of residences with respect to the top of curb. This provides maximum protection for minimum cost. Table X in Appendix A contains required heights of finished floors above top of curb for various street slopes to accomplish this result. Buildings facing the upgrade of a "T" intersection must be given individual consideration and treatment, since allowances must be made to overcome conditions created by the momentum of oncoming flows. Similar requirements for commercial frontage would be desirable. However, the absence of setback requirements and the inconvenience created by raising finished floors of business establishments above sidewalk elevations prevents this application.

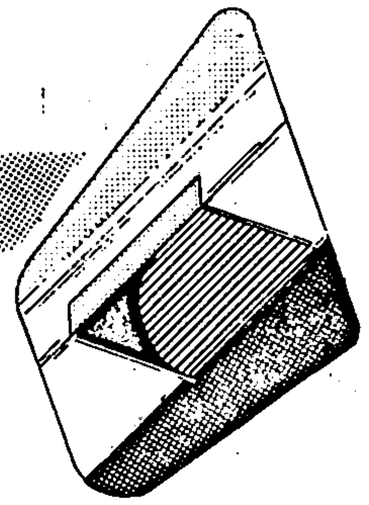


Hydraulic carrying capacities of streets can be substantially improved by modifying cross section designs. However, this may produce some inconvenience and reduce driving safety during storms, and will make interception of flows with standard storm sewer inlets more difficult.



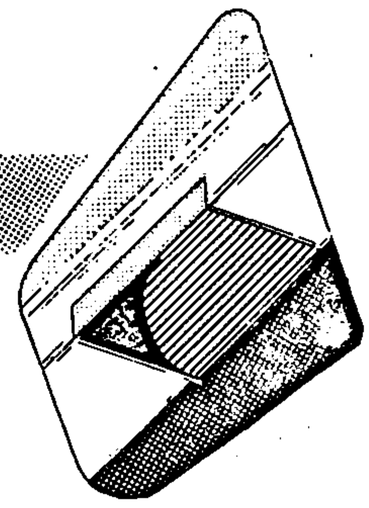
APPLICATION OF BASIC CRITERIA - DRAINAGE FACILITIES:

STORM SEWERS: Storm sewers are the primary systems required to relieve excessive flows in streets. The relationship between street carrying capacities and requirements for storm sewer systems is as follows: (1) streets will be designed to carry runoff from the 25 year frequency storm wherever possible; (2) storm sewers will be designed to intercept and transport that excess of 25 year runoff which the street system cannot handle; (3) runoff will not be allowed to accumulate from areas greater than one square mile (640 acres) in street-storm sewer systems; and (4) when runoff from an area of one square mile has been accumulated, it will be transported to major carrier facilities in interceptor storm sewers. Available points of discharge for storm sewer systems fix configurations of areas served by combinations of streets and storm sewers. This criteria establishes the following: (1) the primary purpose of storm sewer systems is to supplement street capacities in handling 25 year runoff; and (2) storm sewer systems will collect, transport and discharge 25 year runoff when accumulations in streets become unmanageable. Chart XI in Appendix A illustrates a grid type development of a one square mile area developed to produce maximum runoff. This chart also shows one method of distributing drainage flows between streets



and storm sewers. Actual subdivision layout together with flow orientations fixes requirements. Chart XI is primarily significant in evaluating undeveloped highland areas where comprehensive overall planning can be accomplished. The concept can be adapted, with modifications, to certain developed highland areas, but existing drainage patterns render it difficult and often infeasible to apply. Thus, present criteria, which provides that storm sewer systems intercept and carry 10 year storm runoff when flows become unmanageable in streets, must be continued in developed areas.

An entirely different concept is applicable to areas on the valley floor. Generally, streets and storm sewers will serve very small local areas, since low natural gradients do not permit the accumulation of large quantities of runoff. Areas approximately 3,000 feet long by 500 feet wide which have 0.1% minimum gradients in the 3,000 foot direction are easily drained when they are adjacent to a facility where they can be discharged. Economic considerations preclude the use of long storm sewer lines. Absence of any substantial natural slope requires lines of any length to be built at considerable depths and the proximity of ground water in these areas necessitates expensive dewatering during construction. The depth at the point of discharge also requires pumping in order



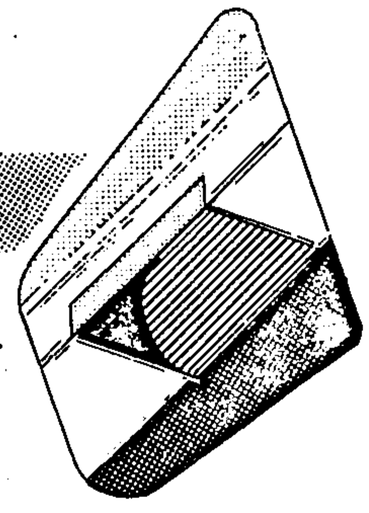
to lift flows into the river or other carrier facilities.

Storm sewers handling only local runoff provide the most economical system. Economic considerations make it imperative that the minimum possible use of these systems be made. If function is measured against construction cost, extensive systems cannot be justified. However, no other facilities are available for transporting large flows in some existing developed areas, thus this use is occasionally unavoidable.

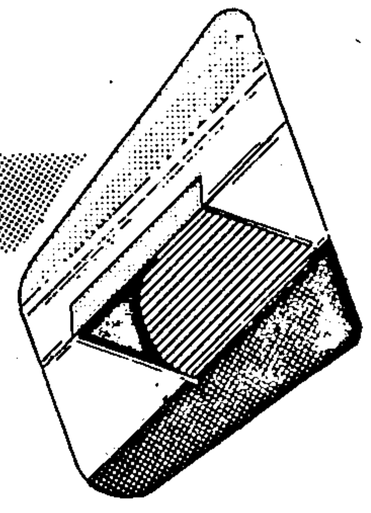
Storm sewer conduits must be designed to preclude high rates of exfiltration even when systems may become pressurized as a result of intercepting runoff in excess of that which can be carried by gravity flow. Since standard inlets will intercept varying quantities of flow under different circumstances, it is anticipated that pressurizing will be more common when storm sewers are used to supplement streets. This requirement becomes more significant as street paving standards are upgraded, since washouts in the subgrade result in serious surface damages. Therefore, conduit joints designed to withstand pressure are mandatory.

APPLICATION OF BASIC CRITERIA - DRAINAGE FACILITIES:

DRAINAGE CHANNELS: Drainage channels receive, transport and ultimately dispose of flows from streets and storm sewers. Economic and functional evaluations indicate that these vehicles and their appurtenances render the best overall service when they

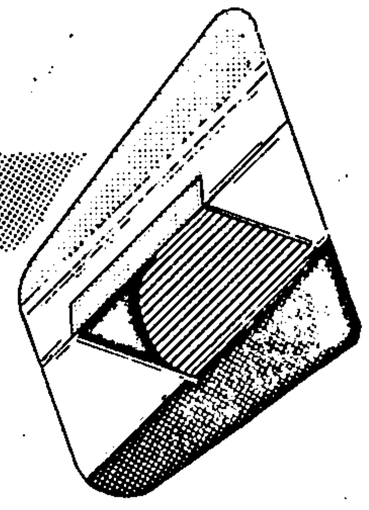


are designed to handle 100 year storm runoff. Differences between right-of-way requirements and construction costs for 25 and 100 year designs do not justify foregoing the protection afforded by the larger requirement. However, one major qualification is mandatory to justify this criterion. At intersections of streets and drainage channels, streets must be designed to discharge their full carrying capacity into the channel. If the structure at the intersection has an overflow or dip section, this requirement is automatically fulfilled. However, any other condition requires special treatment. Increasing storm sewer capacities at these points will accomplish this, however, this cannot be justified economically. This is best accomplished by providing a rollover section in streets to dam flows and overflow them into channels. Outlets to drainage channels are designed to handle 25 year runoff while channels are designed for the 100 year storm. Thus, in storms larger than the 25 year frequency, channel capacities are reached only when other facilities are overflowing or functioning at higher than optimum rates. Therefore, excess runoff will reach channels only by flowing uncontrolled through developments, unless precautions are taken to prevent it. Also, when streets are capable of transporting flows larger than 25 year runoff, it is not realistic to intercept only that portion of the flow at drainage channels, since these accum-



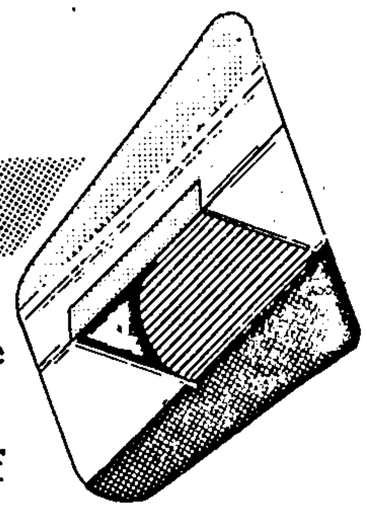
ulations of drainage will cause difficulties downstream. Many streets in the highlands are capable of carrying runoff in excess of that generated by the 25 year storm. Another effective method can be employed to channelize large quantities of runoff. Streets which cross a number of channels may be designed to receive, direct, carry and discharge a fixed quantity of runoff into a channel nearest to its point of accumulation. Existing north-south arterials in the developed area of the east highlands indicate how this type of treatment might have been applied. Problems created for properties adjacent to streets to be treated in this manner, must be considered prior to design. All streets must direct flows to channels via the shortest possible route. This is done in order to obtain their best overall function by reducing possibilities of unwarranted accumulations of runoff.

Drainage channel design is often governed by considerations other than those which are purely functional. Esthetics and specific characteristics and requirements regarding their use in particular areas often dictates design elements. Channels located in highland areas are usually designed for shallow flows, with cross sections selected to avoid impairing or detracting from land use and appearance. Paving these channels will not be common, although requirements in certain developments may render this treatment advisable. In contrast, low slopes in lowland areas require



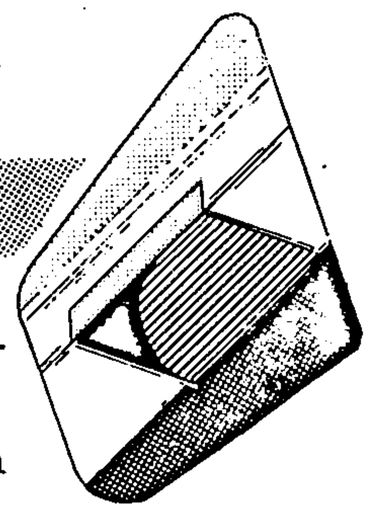
larger and deeper ditches to service equivalent areas. Lining these to improve carrying capacities is usually indicated. Channels devised to intercept runoff from the highlands before it reaches the valley floor are major design and construction projects. They require extensive easements of considerable width. Only comprehensive overall planning will minimize severance damages and rerouting of utilities. This type of ditch must be lined to obtain the best possible carrying capacity along with facilitating maintenance.

Minimum depths should be maintained in channels except where capacities require excessive widths. Those reaches which are critical to entrance and discharge requirements for structures require independent consideration. Shallow depths provides a measure of safety for children who use these facilities as playgrounds. Also, maintaining side slopes will be less difficult. Design depths are fixed when energy and momentum of flows is a primary consideration. In areas with average gradients of 1% and over, relatively shallow channels are most satisfactory. Where very low slopes of 0.1% and under prevail, substantial depths are required. Low gradients generally occur in areas on the valley floor where ground water levels prohibit deep channels. A study of the Middle Rio Grande Conservancy District drain complex reveals that channel depths in these areas are limited to about twelve feet. The usual



seasonal variation in ground water level indicates that this entire depth would generally be available for transporting storm runoff during the July through September period. However, some flows due to exfiltration of ground water, will occur in these ditches during winter and spring months.

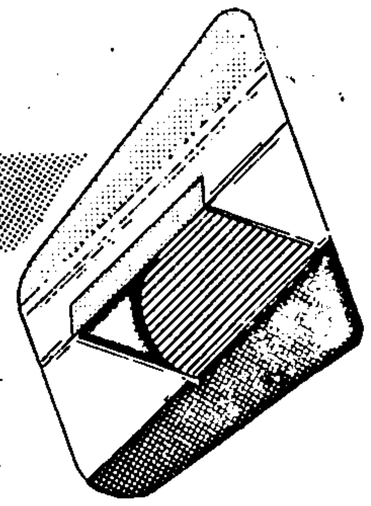
Designing channels requires evaluating the effects of energy and momentum of flows. Abrupt changes in the gradient of channel floors creates two problems as follows: (1) sudden decreases in slope produces hydraulic jump (a sharp rise in water level) which can result in overflow; and (2) sharp increases in grade will introduce hydraulic drop (a rapid fall in water level) which induces scouring. Relatively large transitions in gradient must be accomplished in several increments which individually will not induce appreciable changes in flow conditions. Abrupt variation in channel cross section also causes undesirable effects. Hydraulic drop results when abrupt changes from narrow to wide sections occur, and conversely, sharp transitions from wide to narrow channels produces hydraulic jump. Obstructions to normal flows can also result in jump. Changes in channel alignment produces problems created by flow momentum. Gentle horizontal transitions coupled with increasing downstream bank height and protection usually provides effective control. Momentum of flows is of primary signifi-



cance at channel confluence, since combining flows creates a possibility of producing an oscillatory wave motion downstream. When feasible, channels should be oriented as nearly parallel as possible before flows are combined. The channel generally having the largest flow prior to convergence should be oriented as nearly as possible to coincide with the alignment of the resultant flow. When large flows are encountered, increased bank heights at confluence and for a distance downstream are usually required to prevent overflowing. Design considerations must also include evaluations of energy since it also has a considerable effect in all of the foregoing occurrences.

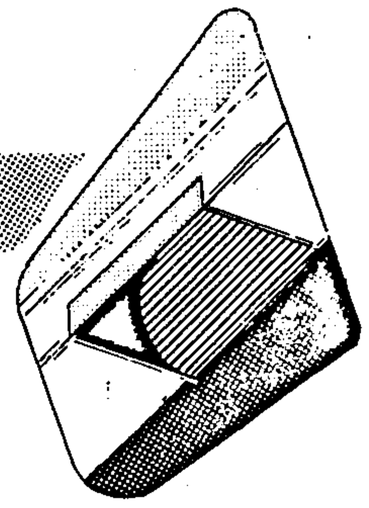
Developers of private land, through which channels pass, should be given some latitude in selecting channel alignments to permit maximum beneficial land use. However, all foregoing considerations must be evaluated for any alignment. Natural drainage courses generally meander and channelizing them to follow straight lines and gentle transitions will produce the best results. Locating, orienting and designing channels where rapid changes in gradient, abrupt transitions in alignment and difficult conditions of confluence with other carriers will occur must be avoided.

Major diversion channels are required to intercept flows from facilities that cannot be discharged directly into the river. These

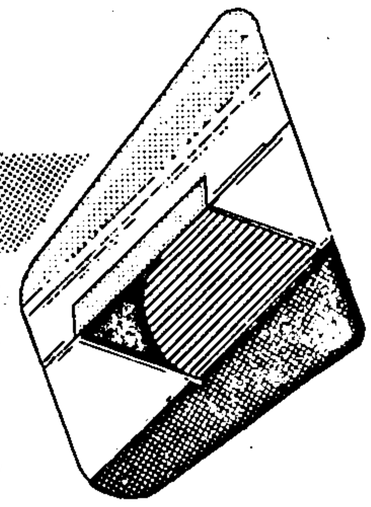


carriers will transport runoff through and above areas which are not normally subject to these flows. Thus, they must be designed to intercept and carry runoff from the maximum probable storm to afford the additional protection required. The same technical considerations set forth for other channels apply to these carriers. However, considerations regarding energy and momentum must be given a priority in design work due to their unusual capacity and interception requirements. These carriers cannot be financed in the same manner as channels which intercept normal flows from developments through which they pass, since they are community facilities requiring overall participation.

The Middle Rio Grande Conservancy District's (MRGCD) drain ditches are indicated for use in both the north and south lowland areas. These were chosen for two reasons: (1) because no new rights-of-way would be required; and (2) because they can serve immediate needs. Negotiations with the MRGCD must be predicated upon the assurance that this use of these facilities will not interfere with or impair the primary function for which they are intended. Extreme caution must be exercised in this regard. Each facility discharging into these drains must be designed so that there is no possibility of its acting to deter the intended function of the ditch. Apparent advantages are offset somewhat by the fact



that the first improvements required would not necessarily be in areas where service is effected, because maximum flows would occur downstream. The City may encounter some difficulty financing a large number of these projects. In some instances, existing rights-of-way may be inadequate to house channels which will be required, since depths are limited by ground water levels. Lining channels, procuring additional rights-of-way to provide for increased widths, or a combination of these methods may be required in some situations. Discharging these ditches into the river is invariably a problem. It is not usually feasible to transport drainage to points where they discharge naturally; thus, large pump stations are indicated. Locating these pump stations where drains converge near the river will eliminate extensive piping systems. Extremely large pumps are required to discharge peak flows. It is anticipated that the most effective service will result when pumps are installed in batteries along the ditch, since this permits progressive development of pumping facilities concurrent with area development. Using low slope ditches will also permit improvements to be made as development occurs instead of requiring complete new facilities to serve local needs when there is no assurance that the entire area will ever develop.

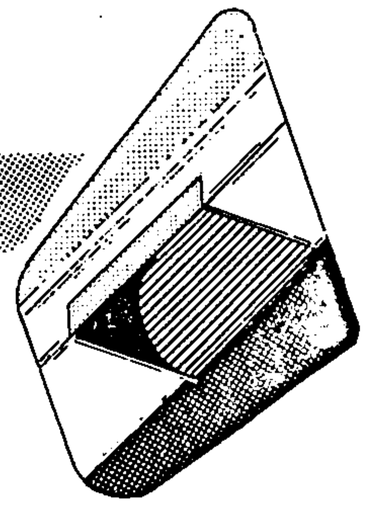


APPLICATION OF BASIC CRITERIA - DRAINAGE FACILITIES:

DRAINAGE STRUCTURES: Drainage structures must be designed to pass the 100 year runoff to remain consistent with channel requirements. Since streets and storm sewers are designed for 25 year runoff, conduits of structures must handle this quantity of flow and the remainder can overflow. If no overflow section is possible, the entire flow must pass through the conduit.

When flows are partially passed through overflow sections, which are dips in streets above structures, advantages in function together with savings in construction costs are effected. This type of construction is not usually possible in highland areas, because steep slopes render natural drainage courses relatively shallow. Thus, excessive excavation of the channel is required to obtain the necessary depth for both a conduit and an overflow section. In many instances channels are so shallow that rollover sections are required to situate structures without raising the entire street above the elevation of surrounding terrain.

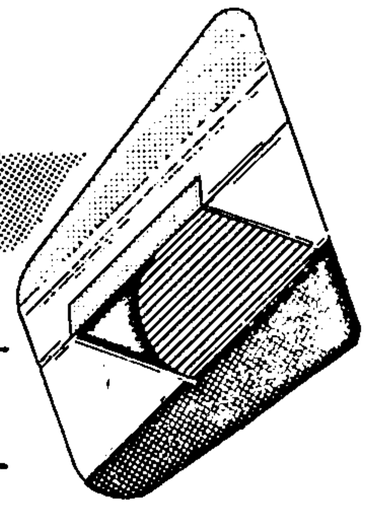
The orientation of a channel where it intersects a street requires careful consideration. Normal crossings usually produce the best overall conditions. When channels are skewed across a street, the increased length adds substantially to construction costs and inlet and outlet treatment become more complicated. When



overflow sections are provided for skewed structures, the riding quality of the overlying street is often impaired. This condition is created by the angle that the overflow trough makes with the direction of travel.

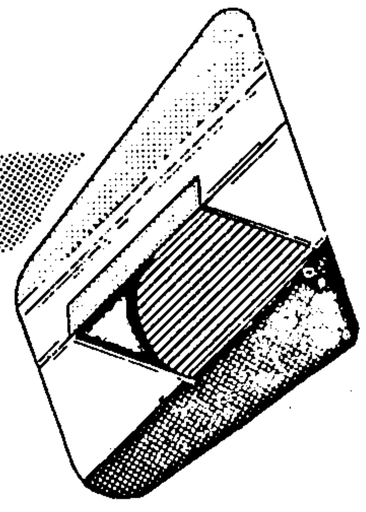
APPLICATION OF BASIC CRITERIA - DRAINAGE FACILITIES:

PONDING AREAS: Ponding is indicated only when economic advantages far outweigh other considerations. If overflow into another facility can be accomplished by building gradient through increased depths of ponding, the physical area required can be limited. If this is not possible, the size of the ponding areas is fixed by requirements to accommodate maximum probable runoff. Temporary ponds equipped with pumping facilities to provide controlled discharge produce the best results. This precludes the possibility of water standing for long periods of time and pumping and piping requirements are minimized. The capacity of any overflow facility must handle maximum probable runoff to insure proper function if this storm occurs when the pond is full. Discharge pumping facilities capable of draining the full capacity of the pond in a maximum of 72 hours are required in order to prevent possible introduction of health hazards. If discharge pumping is accomplished directly into the river and not into another channel which must function during a storm, pumping can begin simultaneously



with ponding, and ponding area requirements can be reduced accordingly. Otherwise, pumping cannot begin until the other installation has fulfilled its primary purposes and the pond must accommodate maximum probable runoff.

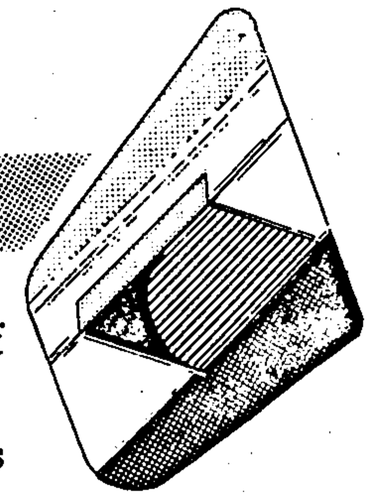
Ponding is usually indicated in areas where little or no natural gradient exists. This creates problems in determining the most effective methods of construction. The surface of the pond must be maintained below the lowest invert of incoming flows to prevent backing water into facilities which discharge into it. In valley areas, channels must be constructed at maximum depths which ground water will permit in order to obtain maximum capacity and still receive discharges from streets and storm sewers. Difficulties arising in effecting the discharge of these channels into ponding areas are apparent. Excavating ponding areas below the ground water level defeats their primary purpose. Thus, the only economical solution requires widening incoming channels and reducing their depth. This requirement precludes the use of MRGCD drains in this reach since their primary function is impaired. Low natural gradients require a considerable reach of a channel to be treated in order to accomplish this transition, thus this portion of channel will not be available to receive branch flows. Depths of ponding are extremely limited in the valley, since the 12 foot maxi-



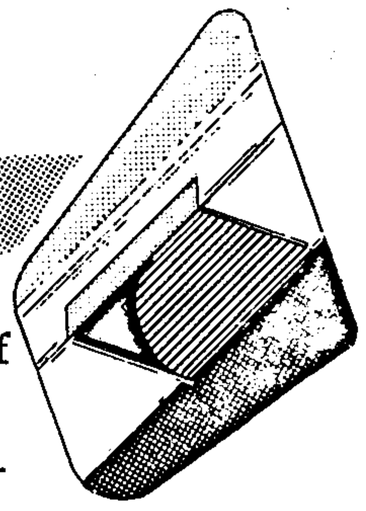
mum depth established for channels also applies to ponds. This increases area requirements. Overflow systems in valley areas are impossible, because inflow requirements do not permit building the necessary artifical gradient. Excess excavation can also create problems. It is anticipated that excess earth could be used for embankment around the pond to separate the basin from the surrounding development. In areas outside the valley, establishing depths is generally arbitrary. Where invert elevations of incoming flows are controlled, excess excavation can be used advantageously to increase depths of ponding and thereby create conditions necessary to provide overflow systems.

APPLICATION OF BASIC CRITERIA - RAIN FALL AND RUNOFF: The total quantity of rain that falls depends upon the intensity, duration and areal extent of the storm. Rainfall volume is not usually of primary significance. Rainfall in excess of that which is absorbed, ponded and evaporated (runoff) is the important consideration. Intensity, duration and their relationship are important in making runoff computations. Both the total quantity and peak rate of runoff are significant when effects of storms in this area are considered.

Total quantities and peak rates of runoff were each discussed in detail in the illustrative example presented in the section on



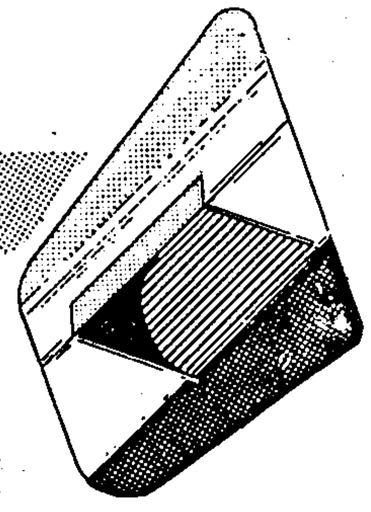
Derivation of Basic Criteria. Computations of total quantities of runoff in this area by the method set forth is not practical. Storms do not usually have a large areal extent and individual drainage areas relating to any particular ephemeral stream are generally small. This method would require the establishment of a large number of measuring stations in order to collect information necessary to prepare accurate isohyetal maps. Thus, the total quantity of runoff must be ascertained by making physical measurements of streams which deliver it. Collecting this data permits estimates of total runoff to be made for various magnitudes of storms by establishing relationships between actual and computed information. Making estimates of peak rates of runoff are similarly complicated. The most effective method for making these computations results from employing data derived from Yarnell's premise. Times of accumulation can be estimated with reasonable accuracy, and, when more measuring stations are established, these times can be computed by formulae derived from information made available from accurate measurements. Thus, total quantities of runoff are derived from making actual stream measurements and peak rates of runoff are computed by employing intensity values obtained from the local version of formulae derived from Yarnell's standard equations. Total runoff is expressed in



acre-inches or acre-feet. Computations of peak rates of runoff yield results in acre-inches per hour which, for all practical purposes, is equal to cubic feet per second.

When detailed measurements of individual stream flow are made, a graph of these flows can be constructed. This graph, known as hydrograph, is a plot of measured flows against time and it makes it possible to derive both peak rates and total quantities of flow. Unit hydrographs which relate a given quantity of runoff to the length of time which a stream flows, may also be developed for individual streams. These permit calculations of both peak rate and total discharge from the drainage area for any unit of runoff.

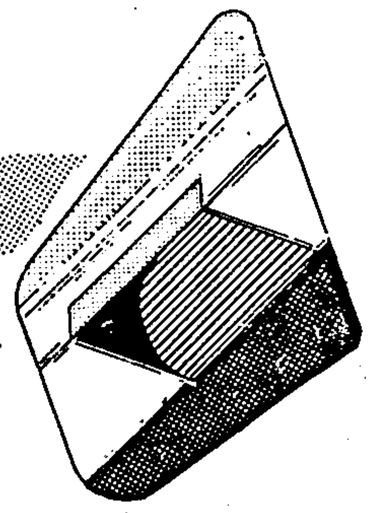
Data which will be most frequently employed for making rainfall and runoff computations are as follows: (1) the intensity-duration-frequency chart (Chart 1, Appendix A); (2) the "Rational Formula" described in Appendix A; (3) the peak flow-area curve (Chart 2, Appendix A); and (4) the maximum probable curve (Chart 3, Appendix A). A coefficient of runoff of 0.45 was derived as described below for application to undeveloped areas when the "Rational Formula" is used for computing peak rates of runoff. In comparison, a coefficient of 0.30 was developed for the Sandia Conservancy District Master Plan for the Hoffmantown Subdivision



in the northeast Highlands of Albuquerque and 0.35 is commonly used for design purposes within metropolitan Albuquerque. Each area must be evaluated individually for design purposes.

Drainage areas applicable to runoff computations were measured on U. S. Geological Survey Quadrangle Maps with scales of 1:24,000 and 1:250,000. Intensities were derived from Chart 1 of Appendix A. A ratio of excess rainfall to total rainfall (0.45) was developed especially for this study. It is based upon a one square mile area developed in a manner to yield the highest possible runoff and remain consistent with present zoning ordinances and planning policies. Physical areas of various types of surfaces in existing developments having similar zone classifications were measured on aerial photographs. Standard runoff coefficients were applied to these surface areas and an area-mean runoff coefficient was developed. Table XI of Appendix A depicts the zoning pattern conceived to produce this coefficient. It is not probable that such development will ever prevail; therefore, a lower coefficient will generally apply. However, since this order of development is possible, it is almost mandatory to use it in considering undeveloped areas.

This section sets forth the complex requirements for analyses of rainfall and runoff. Technical specialists are required to make

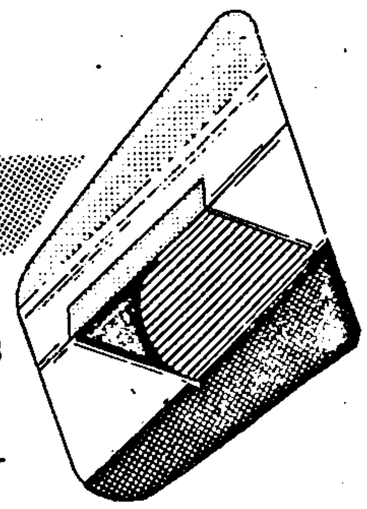


evaluations and computations in this field in order to prevent misinterpretations.

Criteria developed with regard to rainfall and runoff is adaptable for runoff computations for areas indicated on the maps. The contributing area applicable to each facility is indicated on maps rather than showing quantities of flow. This permits runoff computations to be made for any frequency storm. The variation in requirements for each type of installation together with the impracticability of designating the particular facility to be used renders information on quantities of flow difficult to adapt.

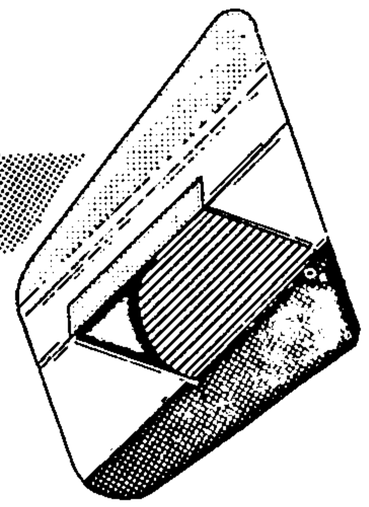
APPLICATION OF BASIC CRITERIA - CATEGORIES OF AREAS:

UNDEVELOPED "HEIGHTS" AREAS: Undeveloped "Heights" areas were first analyzed using natural drainage courses and their natural contributing areas. After this was completed, large areas were gridded into one-half mile squares (160 acres). Each of these incremental areas was then applied to the natural drainage course which was most accessible according to the dictates of natural topography. Therefore, this method indicates areas contributing prior to development along with areas expected to contribute after one possible method of development is imposed. Since the grid system of development is being rapidly abandoned in order to consider all phases of land planning and land use applicable to specific areas,



some comprehensive analyses were made in this regard. This study revealed that in most instances topography and other purely functional attributes will govern land planning. Therefore, the grid system method which was employed provides a workable means of applying given areas regardless of the configuration of the final plan. In order to substantiate this, areas indicated in the Paradise Hills and Snow Vista Master Plans were analyzed in the foregoing manner. A close examination of these areas as depicted on the maps indicates that the method is both applicable and workable. This comparison is also valuable, because it illustrates the application of the master plan with regard to future development.

The fact that natural drainage courses are the only carrier facilities indicated in these undeveloped "Heights" areas does not mean that they are the only vehicles required to transport storm runoff. Chart XI in Appendix A is indicative of the extent of development required in order to provide service for the interiors of individual areas. Where acreage figures are indicated on a natural drainage course, streets and usually some storm sewers are required to deliver flows accumulated within this area. Chart XI indicates only a schematic method of providing local service. Actual requirements will depend upon orientation, alignment, location and design of streets within the area. Before the completion of final



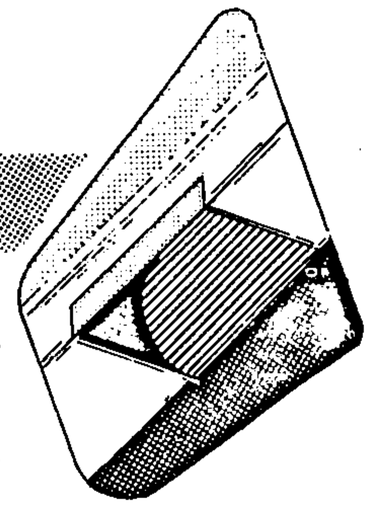
land plans, it is impossible to foresee actual conditions. Therefore, the general approach shown on Chart XI is used to indicate general requirements within an area thereby preserving the flexibility of the plan.

Purely arbitrary methods were employed to select the natural drainage courses which are indicated. However, these choices were predicated upon rendering service to relatively large areas. The proximity of arroyos in many areas offers some latitude for making the final selection of the one which is to be used. If the overall intent of the plan is not violated, developers should be allowed to participate in choosing the final location in order to effect the best land use.

The grid method of development has one advantage in handling storm drainage that will not usually be found in other configurations. The provision requiring all streets to be completely drained of 25 year runoff from a maximum contributing area of 640 acres establishes that the largest storm sewer line in a grid type of development will generally carry flows accumulated on two 160 acre tracts. This is indicated on Chart XI, and is an ideal condition which cannot normally be expected to occur in any other type of development.

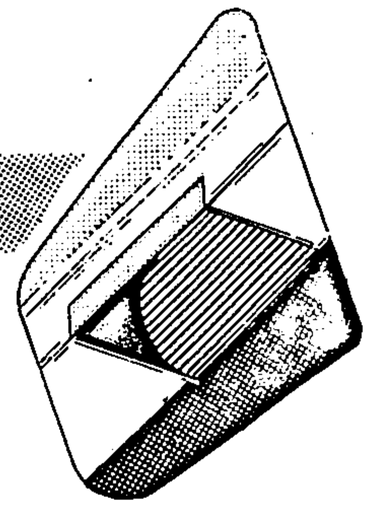
APPLICATION OF BASIC CRITERIA - CATEGORIES OF AREAS:

DEVELOPED "HEIGHTS" AREAS: Developed "Heights" areas



were analyzed by using a combination of collected and computed data. Natural drainage areas contributing runoff to sections of the community were evaluated. However, the vast complex of streets which collect and transport runoff dictated the use of a large amount of information collected from observation. The entire developed area has been studied for several years and most areas where drainage problems exist have been identified.

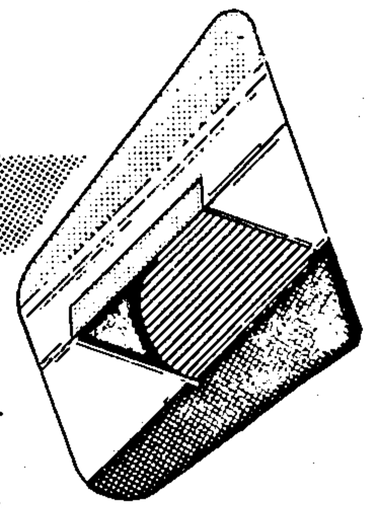
Criteria applied to different facilities in developed "Heights" areas will vary considerably. It is obvious that a storm sewer designed to transport runoff from the 10 year storm cannot be extended using any other criteria. New systems constructed in areas where drainage installations have already been built should be designed based upon the same criteria used for existing systems, unless some modifications can be made that will increase the protection afforded by the existing facilities. Major carriers, other than channels, must usually be designed based upon flows received from branches rather than on a frequency basis. However, in some instances, provisions for prohibiting overflow from a definite frequency storm is desirable. If branch flows equal or exceed this runoff, the provision is automatic. However, if incoming lateral lines deliver some lesser quantity of flow, then additional control and intercepting facilities must be provided in conjunction with the major carrier.



Criteria to be used for each situation can be selected after policy for requirements is established.

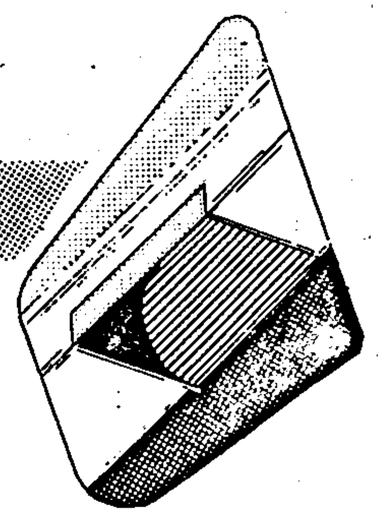
Requirements in these areas vary from simple extensions of local systems to providing major carrier facilities. These modifications and additions necessary to improve the function of the existing system are shown on the maps. The Master Plan indicates overall requirements, but does not assign any priorities for construction. In developing priorities, the following can be considered: (1) construct all local facilities possible where major carriers exist to intercept these branch flows; (2) construct the necessary carrier facilities required to enlarge the overall system; and (3) construct additional branch systems as carrier facilities are provided to accommodate their discharge. Is it probably not possible to adhere strictly to this policy, but some predetermined logical sequence is essential. Only limited extensions of existing systems and few new installations can be accomplished before some provisions is made to enlarge capacities of existing primary carriers and to provide new primary facilities that are necessary. Thus, this work must be given a high priority.

To conform with previously developed criteria, design for any facility within this area is undesirable. The fact that overall local drainage patterns can be altered considerably by making rela-



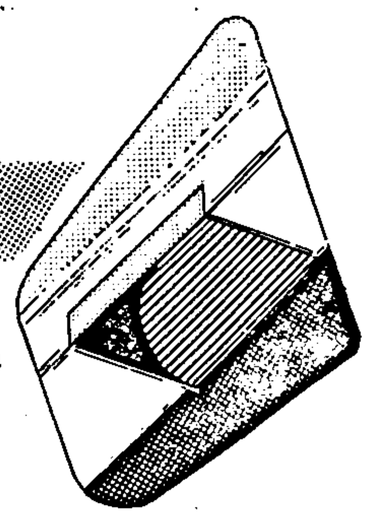
tively few modifications in the orientation of flows on individual streets is of particular significance. Serious consideration should be given to re-routing runoff presently transported by certain streets in order to effect better distribution and disposal of flows. New street paving projects should be carefully evaluated with regard to the accumulation and discharge of storm drainage. Design engineers must be authorized to decide the manner in which flows are to be received, intercepted and directed to effect the best and most economical treatment. Other important considerations are also evident. Runoff from Sandia Base creates some problems in the southeast quadrant of the developed highland area. Most of this drainage can be readily disposed of in Tijeras Arroyo, thereby minimizing these problems. This will require negotiations with various federal agencies.

Abandoning the Sandia Conservancy District Plan as it was originally conceived has created several problems in the developed "Heights" area. The East Diversion Channel would have effectively protected the northeast quadrant of this area by precluding runoff that had its origin in the mountains to the east. While the District was still in effect, considerable design and construction work was done for channels and drainage structures in this quadrant of the City. These installations are now under-sized for present require-



ments. Although the Embudo Channel is presently being built in conjunction with the east-west freeway, its function is not entirely in keeping with that proposed in the Sandia Conservancy District Plan. Policies of the Bureau of Public Roads, the agency in charge of establishing criteria and approving design on interstate highways, are not always compatible with urban development. In many instances, the Embudo ditch has been designed to receive and transport overland sheetflow when it would have been more advantageous to accumulate these flows into carriers within the developed area and to discharge them into the ditch at specific points. Since this is not in keeping with highway design policy, only existing conditions were treated. This design effectively prevents introducing new channel flows into the ditch. These modifications require a complete re-evaluation of all facilities originally intended to terminate in this channel.

Even when financing becomes available for building some facilities, other work required in conjunction with them often delays the project. This fact emphasizes the need for long range planning. The San Mateo Storm Sewer Project is a typical example. This installation has top priority as a drainage facility, but the street widening that must be done in conjunction with it to effect an economical overall development is not ready to proceed. No project

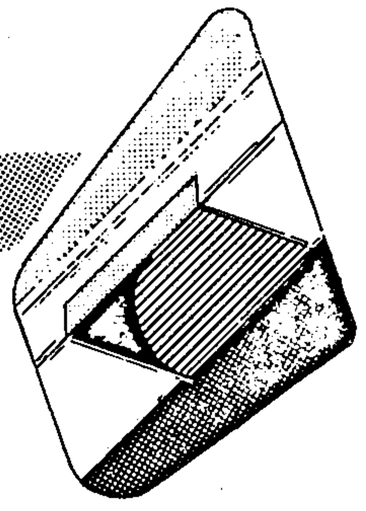


of any magnitude can be initiated, designed and constructed with expediency without comprehensive long range planning.

APPLICATION OF BASIC CRITERIA - CATEGORIES OF AREAS:

"VALLEY" AREAS: Major problems are encountered in both categories of "Valley" areas because runoff is difficult to accumulate due to low gradients. In many instances, slopes required for drainage facilities can be obtained only by excavating or filling. Excavation is limited by the proximity of ground water and filling requires an extensive operation because all surrounding terrain must also be raised to prevent unnecessary severance damage. If existing low gradients are not modified, drainage areas contributing to streets and storm sewers must be small in order to control overflows. Otherwise, installations with very large cross sectional areas are required, and this is usually uneconomical. Low and non-existent gradient together with a relatively high water table are problems inherent with all "Valley" lands. These factors will dictate elements of proposed treatments.

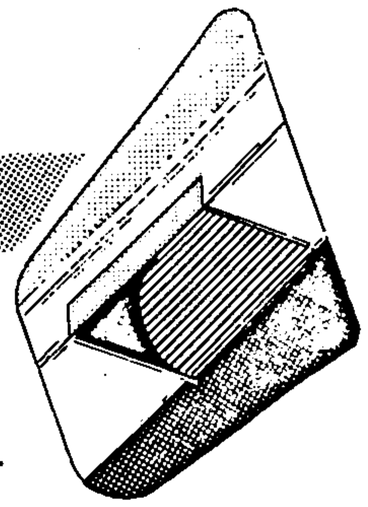
In both categories of "Valley" areas, economics dictate the use of channels. Since MRGCD drains are existing facilities which have minimum flows relating to their primary function during the July through September period together with being adaptable to the intermittent and scattered development characteristic in "Valley"



areas, they should be utilized whenever possible. After negotiating with the MRGCD for employing these facilities, detailed surveys should be made to determine specific uses. Any future platting in "Valley" areas should be designed to make the best possible use of these ditches for drainage purposes. Undoubtedly, manipulations will be necessary to provide service for some existing developed areas, but it is probable that most areas can be drained. When situations arise where MRGCD drains cannot be used, then similar facilities designed specifically to handle storm runoff must be provided. This entails a number of problems, but the extent of construction required to effect discharge into the river constitutes the major difficulty. If this type of installation becomes necessary, locating them in median areas of major collector and/or arterial streets should be considered. Advantages inherent in adopting this use are obvious, but crossings required to provide access to side streets and business establishments can develop into a serious and expensive problem. It is concluded that a system of low slope ditches which provide for discharging local drainage facilities is indicated.

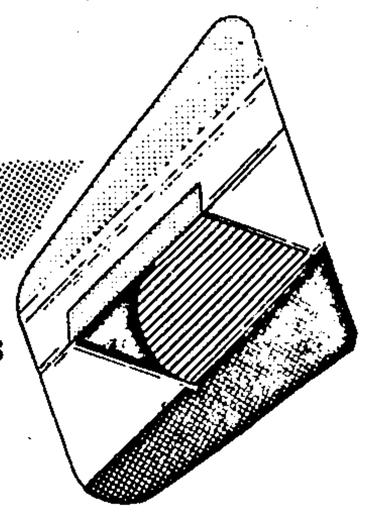
APPLICATION OF BASIC CRITERIA - CATEGORIES OF AREAS:

UNDEVELOPED "VALLEY" AREAS: Undeveloped "Valley" areas are treated by using ponding, storm sewers and low slope



channels. In all instances, areas were analyzed using each of these methods as the primary facility. Costs and feasibility were considered for each situation and the most practical solution was adopted.

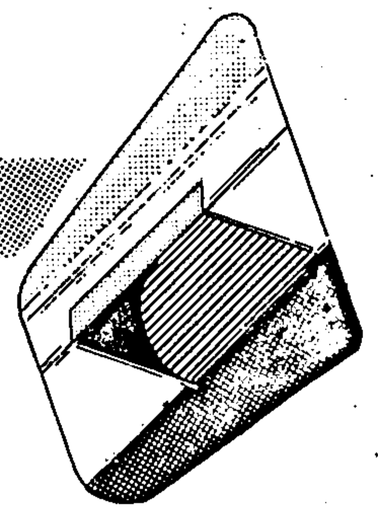
Using storm sewers as the primary facility was eliminated for several reasons. A trunk system of piping requires north-south trunk laterals to be installed to receive flows from local installations. Major trunk lines which would intercept flows from these laterals must be oriented in an east-west direction to effect their discharge into the river. Natural gradients from north to south vary from 0.04% to 0.10% and extensive storm sewer lines laid to conform with these low slopes cannot be economically justified. If additional gradient is built into these trunk laterals, depths are excessive at points where east-west trunk lines intercept them. Through access for the east-west trunk from its inception to its discharge into the river is mandatory. This requirement automatically fixes areas to be served by north-south laterals, since existing major east-west streets define this access. This provides little or no latitude for achieving economical limits of areas to be served by lateral trunk lines. Major east-west trunk lines traverse areas that are, for all practical purposes, flat; and, in many instances, profile elevations will fall below the datum plane of the river bed.



These lines range from 8,000 to 11,000 feet in length. Unless large portions of them are built below the ground water level, little or no slope can be attained. Even when lines at depths of 30 to 40 feet are contemplated, the quantities of flow dictate the use of very large piping. Under any circumstances, it is necessary to provide pumping facilities to discharge them into the river. Costs for constructing this type of system in fully developed areas are extremely high. When areas to be served are only partially developed, costs are prohibitive. In order to provide service for any small individual area it would be necessary to construct discharge pumping facilities along with portions of major and lateral trunk lines. This renders treatment of this type for partially developed areas infeasible.

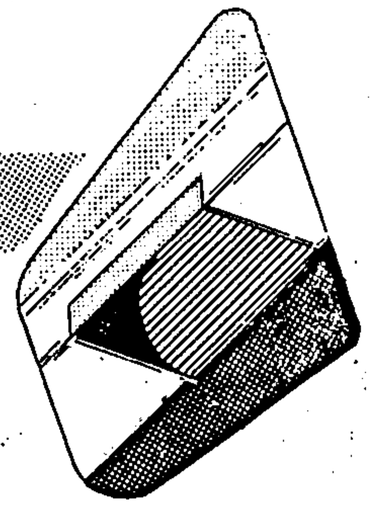
Since storm sewers are not feasible as major carriers, ponding and low slope channels remain for consideration. Each of these facilities are indicated to illustrate their function. A large ponding area is shown in an undeveloped "Valley" area north of the east-west freeway (I-40), and low slope ditches were used throughout the south "Valley". These uses could feasibly be interchanged or the ponding area could be eliminated. The treatment indicated is the most economical.

In undeveloped "Valley" areas, employing MRGCD drains is simple. However, methods of development do become somewhat



fixed, because all drainage must be directed toward these ditches. Runoff can be delivered to ditches by streets, storm sewers or both. Without requiring a storm sewer, the average area that can be served by a standard 32 foot street with a minimum gradient of 0.1% is 3,000 feet long by 500 feet wide. This criteria was adopted because this slope can be obtained in areas with no natural gradient without seriously impairing the use of adjacent property. Excavation three feet deep would result at the drain and this would allow a depth of flow of nine feet in a ditch twelve feet deep. The area served can be increased by adding storm sewer, but the depth of flow in the drain is further limited if this is done. Where existing streets do not terminate at the drains, storm sewer easements will be required. Street runoff directed into surface pipe will maintain the highest possible elevation at points of discharge into channels. This also affords considerable savings in construction costs along with maintaining the maximum efficiency of the ditch.

Ponding is indicated in the north "Valley" because it is not feasible to pump peak flows directly into the river. It is also impractical to effect overland transportation of flows to points where they discharge naturally into the river. Either of the latter methods is possible, but each requires extensive and costly construction. Ponding is not inexpensive; however, it is more economical than



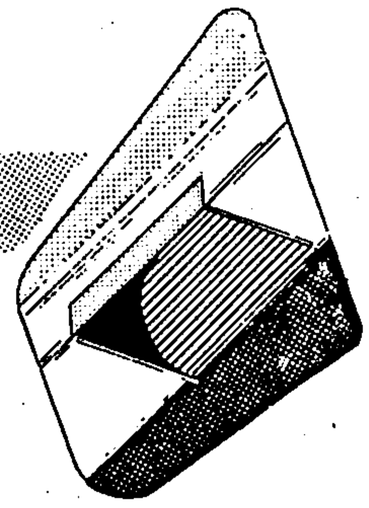
using either of the other methods.

APPLICATION OF BASIC CRITERIA - CATEGORIES OF AREAS:

. DEVELOPED "VALLEY" AREAS: Developed "Valley" areas can be classified as: (1) metropolitan areas where considerable storm sewer work has already been done; and (2) areas outside the City where little or no drainage work has been accomplished. Each of these classifications must be treated in a different manner.

Some existing storm sewer systems in developed "Valley" areas require modifications and additions in order for them to perform properly. These requirements are shown on the maps along with similar requirements for the developed "Heights" area. The completion of the Barelas Pump Station has made it possible to install a more complete system in the downtown metropolitan area, and many problems presently encountered in this part of the City can now be solved. Some use has already been made of the system of MRGCD drains existing in these areas. If these channels can be used to a larger extent to intercept local runoff, the entire system will be more effective.

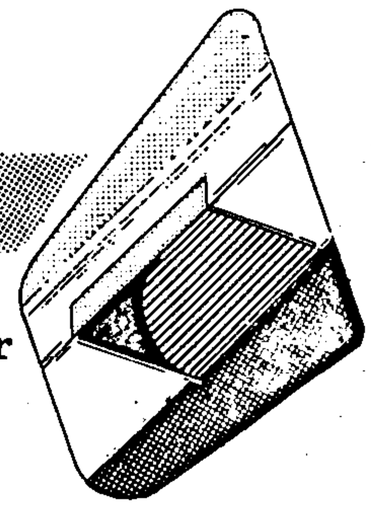
. Developed "Valley" areas outside the City are treated similar to undeveloped "Valley" areas except that present patterns of development must be recognized. Therefore, more extensive systems of local storm sewers will be required. Fortunately, the orienta-



tion of most streets existing within these areas permits access to the drains, but the fact that some streets do not extend through to the ditches necessitates procuring easements to provide required access. Another predominant factor to be recognized is the limitation in depths of flow in ditches in order to permit effectively discharging storm sewer lines. A 3,000 foot branch storm sewer line will be at least seven feet deep at its entrance to the twelve foot deep drainage ditch. Therefore, design water depths in these ditches must be no greater than seven feet if the storm sewer is to function properly. This may require some drains to be lined and/or to be increased in width in the early stages of development.

APPLICATION OF BASIC CRITERIA - CATEGORIES OF AREAS:

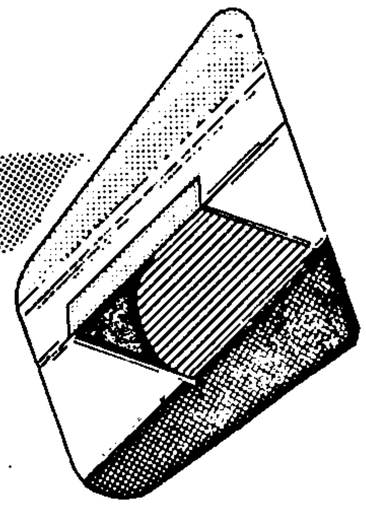
LARGE PLANNED OR PLATTED "HEIGHTS" AREAS: The category of large planned or platted areas includes large undeveloped parcels east of Juan Tabo Boulevard, the Elena Gallegos Grant from Eubank Boulevard to the North Diversion Channel, Albuquerque Estates, Paradise Hills and Snow Vista. Juan Tabo Boulevard to the east boundary of the City is an area that is almost entirely platted while the other subdivisions described are merely master planned, although some small segments of Snow Vista and Paradise Hills have been platted. Albuquerque Estates, areas east of Juan Tabo Boulevard and those within the Elena Gallegos Grant are



treated based upon the configuration shown on plats or master plans. Areas included in the master plan of Paradise Hills and Snow Vista were analyzed as undeveloped "Heights" areas.

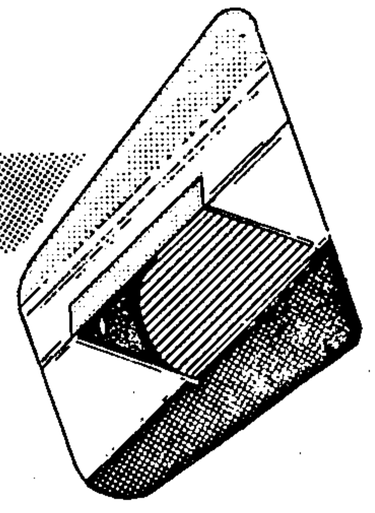
Areas east of Juan Tabo Boulevard were analyzed and treated as though they were developed areas. Careful control of street drainage will be necessary in order to implement this plan. Latitudes offered by control of street drainage alone render the treatment of these subdivision flexible. The fact that runoff can be routed in any one of several different ways constitutes the major difference between this area and developed areas.

Master plan configuration was followed in treating the Elena Gallegos Grant. No drainage easements were employed other than those indicated on the master plan. This method was used to display the vast requirements for storm sewer trunk lines over and above those which are necessary when a number of drainage channels are provided. Fewer drainage structures at intersections of arterial streets and channels are necessary, but the required increase in length and size of storm sewer systems far outweigh this economy. When final platting is undertaken in this area, a through evaluation of comparative costs should be made in order to arrive at the most effective method for final development. Drainage facilities for Albuquerque Estates were based on actual platted

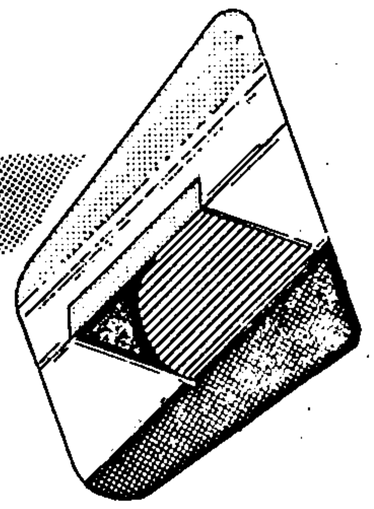


streets, but drainage channels are proposed to provide discharge facilities so that storm sewer lengths and sizes can be limited. This type of treatment will apply to the Elena Gallegos Grant if more channels are provided. It also illustrates more detailed methods and procedures applicable to undeveloped "Heights" areas.

If requirements for primary facilities in the Elena Gallegos Grant Master Planare increased, storm sewer lengths and capacities can be substantially reduced. It is probable that this would appreciably alter the land use pattern proposed. Analysis of the Albuquerque Estates Subdivision was made by gridding it into one-half mile squares. This is the exact method employed for undeveloped "Heights" area. Computations indicate that each residential street within the area is capable of carrying runoff developed in the 25 year storm for a distance of one mile or more before being subject to overflow. Therefore, it is logical to intercept and discharge drainage at one mile intervals. Actual calculations indicate that some streets in the easternmost reaches would carry runoff for distances up to two miles. However, in order to preserve continuity, streets were drained at one mile intervals. Disregarding continuity, there is another major reason for adopting this one mile interval. This pertains to drainage control at the intersections of east-west and north-south streets. East-west streets are ori-



ented almost perpendicular to contour lines and north-south streets have low slopes. The natural tendency is for runoff to continue west at these intersections unless prohibited from doing so. The most effective method of turning runoff onto north-south streets is to "ditch" them, so that flows on east-west streets cannot pass the intersection. This cannot be accomplished in limited distances without impairing riding qualities on east-west streets. If this method is not used, then all runoff must be intercepted on east-west streets where velocities are high, collection is difficult and storm sewer requirements are more extensive. This treatment also renders some difficulties in developing adjacent properties. However, it is absolutely necessary to intercept runoff at these intersections in order to avoid the accumulation of unmanageable quantities. Unless decisive design methods are employed, intersections will turn low and average flows, but high flows will pass them. This results in complete changes in applicable contributing areas at various points for different intensities of rainfall. If arterial streets could be oriented at 45° from the direction of the average gradient, they would all have approximately the same slope. Such alignment should be adopted wherever possible. However, the north-south, east-west grid will usually prevail since land is sectionalized in those directions. The analysis and treatment indicated

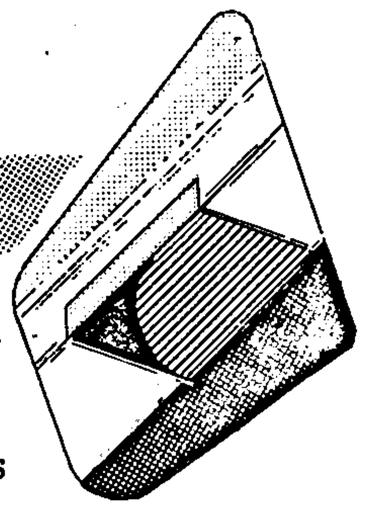


for Albuquerque Estates displays those considerations which must be applied to undeveloped "Heights" areas and other master planned "Heights" areas. This indicates those requirements which are fixed and the variation in methods that are possible.

Snow Vista and Paradise Hills were first analyzed in accord with their master plan. These results indicate that they can be treated as undeveloped "Heights" areas for planning purposes. This illustrates the manner in which proposed land master plans should be examined to determine drainage requirements.

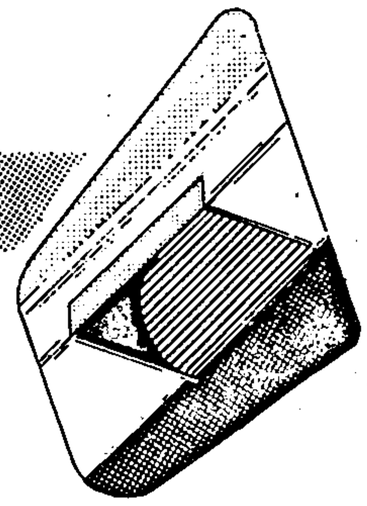
APPLICATION OF BASIC CRITERIA - CATEGORIES OF AREAS:

AREAS REQUIRING SPECIAL TREATMENT: Areas requiring special treatment are those which necessitate extraordinary installations before they can be treated by ordinary methods. These extraordinary installations will generally consist of major diversion channels or ponding areas. All "Valley" areas must be protected from runoff originating in highland areas before provisions for handling local runoff can be effective. Diversion channels and ponding areas are the primary facilities used for this purpose. Ponding may also apply to some highland areas where topography does not permit either drainage directly toward the river or the construction of channels.



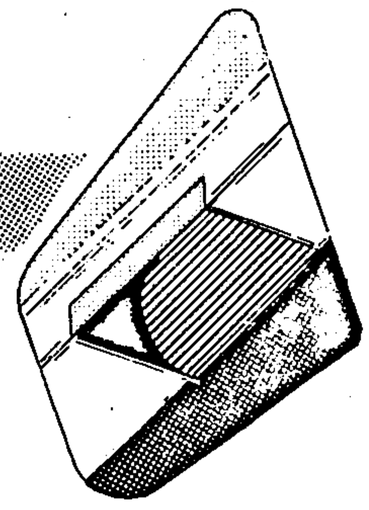
Five major diversion channels are delineated on the maps. They are necessary to prevent large and unmanageable quantities of runoff from flowing uncontrolled into "Valley" areas. These facilities are the North and South Diversion Channels proposed by the Corps of Engineers to protect "Valley" areas east of the river, a channel to be constructed adjacent to the east-west freeway (I-40) west of the river, a channel south of the South Diversion Channel and east of the river and a channel which begins south of U. S. Highway 66 and which terminates north of the Isleta Pueblo Grant Line. The latter channel is the major installation west of the river and is designated to protect "Valley" areas south of U. S. Highway 66.

The North and South Diversion Channels are shown as they were proposed by the Corps of Engineers. It is anticipated that the location, extent and detail for intercepting runoff originally indicated for these facilities will be subject to modification due to changes which have been effected in both the character of land through which they pass and arroyos which they intercept. The North Diversion Channel will probably remain in approximately the same location and it will intercept runoff from approximately the same contributing area. However, points of interception as well as quantities of flow being delivered from specific arroyos have altered considerably due to large scale development and plan-



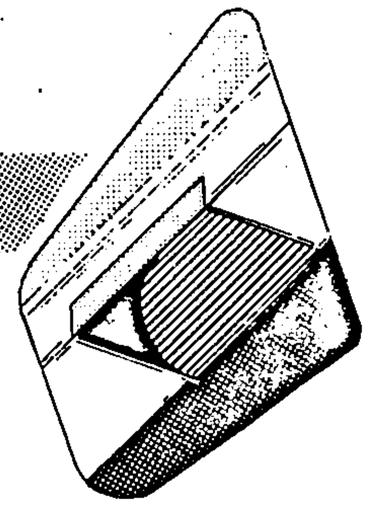
ning which has occurred during the last eight years. The South Diversion Channel will probably be modified considerably because of developments in the area through which it passes. Some pronounced relocation and realignment of this channel may be required. It may be necessary to move its point of beginning as far south as Miles Road. If it is shortened, some additional treatment will be required in the reach between Miles Road and its former point of origin. Some modifications may also be required at its confluence with Tijeras Arroyo. These channels still represent the most feasible and economical method of effecting flood control for the "Valley" areas below them.

The proposed channel paralleling the east-west freeway (I-40) on the west side of the river will eliminate many existing problems for the southwest "Valley" area. In order to justify building this ditch in conjunction with the Freeway, the Bureau of Public Roads must be convinced that its cost will be equal to or less than the cost of providing structures to pass runoff under the highway. This qualification may require design for less than maximum probable runoff. If it can be constructed as a part of the Freeway, the City will benefit tremendously. If this cannot be accomplished, the City should make the necessary provisions to construct the channel with other funds at some future time. Existing gradients parallel to the



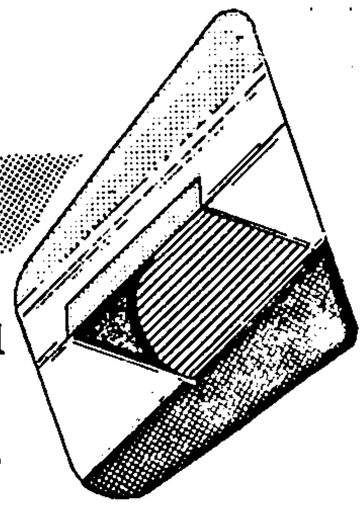
freeway are ideal for channel construction. The alignment of the freeway further enhances this location. This channel will also serve an alternate purpose. Excavation in excess of thirty feet is planned for the freeway at its intersection with Coors Road. Examining topography in areas both north and south of this point reveals that flat slopes and many natural ponding areas exist. This channel provides a means for draining these areas, thereby making some additional protection possible for developments to the east.

A major diversion channel located west of the river and south of U. S. Highway 66 is necessary to relieve inundation in the southwest "Valley". No effective local treatment can be provided for the communities of Atrisco and Parajito, until a major diversion facility is constructed. The alignment and location of the present Arenal Main Canal provides an excellent location for this channel. However, prevailing low slopes coupled with obvious difficulties that would result if this canal was taken out of service for irrigation purposes render its use improbable. A location further west permits existing slopes to be better utilized. The area between this location and the Arenal Main Canal can be served by local drainage systems which would discharge into the Isleta Drain and natural gradients make this possible without requiring extraordinarily large facilities. Crossing the flat "Valley" floor with this diversion chan-



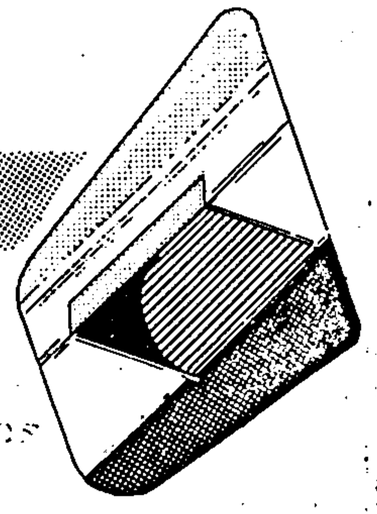
nel will require starting the construction on a fill in order to obtain artificial gradient. Along with creating severance and utility problems, major difficulties will arise in intercepting incoming branch flows. Lifting flows of the magnitudes indicated for incoming facilities into the ditch requires large pumping installations. This method may not be economically feasible. An alternate method of elevating all incoming flows to provide for gravity confluence would increase severance and utility problems. It is contemplated that the angular crossing of the valley floor indicated on the maps will provide fewer problems than any other alignment. By taking advantage of some of the low natural gradient that does exist, the amount of artifical elevation which must be created is decreased.

The ditch to the south of the South Diversion Channel follows the alignment of the Barr Canal. Eliminating the Barr Canal will not create those difficulties which would result from taking the Arenal Main Canal out of service. The area it serves as an irrigation canal is identical to that it would protect as a diversion channel. Thus, when development of the area requires flood control, the agricultural use would be abandoned. Many of the same basic problems discussed hereinbefore apply to constructing this facility when it traverses the valley floor and at points where interception and discharge are effected.



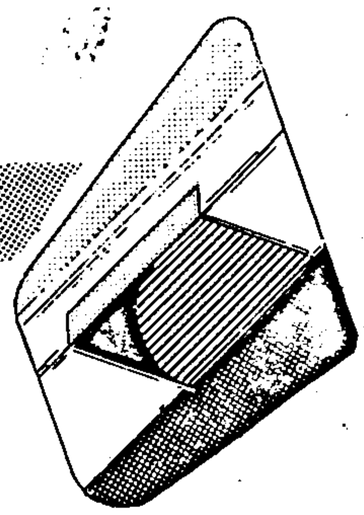
Extensive topographical and route surveys will be required to determine the best alignment and location for all of these facilities. No attempt should be made to locate them before these surveys are conducted.

Ponding of runoff in some undeveloped "Heights" areas bears careful consideration. These ponds would ultimately be drained by pumping. This treatment is indicated in the area west of the Military Reservation and north of the Isleta Pueblo Grant. Each of the three ponding areas shown was treated in a different manner in order to indicate possible variations. The southeasternmost pond has a natural overflow into Hell's Canyon which lies to the south within the Isleta Grant. This provision permits the size of the pond to be limited to handle runoff from any desired frequency of storm. Any runoff accumulated in a small storm or that runoff which is not overflowed in larger storms is then drained by pumping. The southwesternmost pond collects runoff from its contributing area and its content is then pumped into the central pond in order to limit piping requirements. The central pond collects runoff from its contributing area and retains it until the storm is over. This pond must have a capacity that will accommodate the maximum probable storm runoff from its contributing area plus the difference between the maximum probable runoff to and the design capacity of the south-



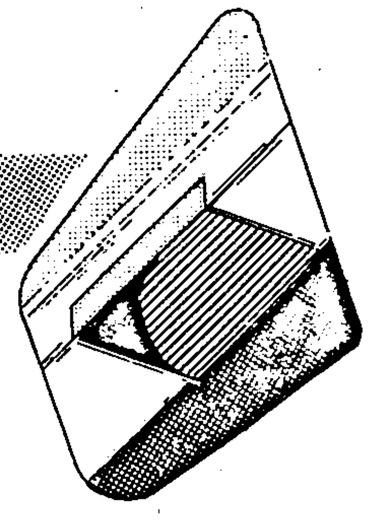
westernmost pond. The pumping facility on the southwest pond must be designed to pump this difference in anticipated runoff. After the storm has subsided, pumping from the central pond and draining of the southeasternmost pond can begin. The entire area is interlaced with facilities designed to route flows into the ponds. Pecularities of topography in this area render this to be the most advantageous treatment. However, other methods could be employed. Several similar areas exist in undeveloped "Heights" areas west of the river and north of U. S. Highway 66. Other feasible methods were employed to drain these in order to indicate possible variations for treating this type of area. However, when development occurs in these areas, it may be more advantageous to create some ponding areas instead of installing facilities that are indicated.

INTERRELATING ALL PHASES OF THE PLAN: The overall plan is composed by relating individual treatments rendered in each of the several categories. The foregoing text sets forth the variety of methods possible for treating various conditions. All problems are identified and a solution is proposed for each. It is impractical to propose actual design in this master plan. As indicated in the text of this report, simple modifications within a developed area or employment of different methods in an undeveloped area



can appreciably alter final design conditions. Locations where treatment is necessary are the primary objectives. The design engineer, after extensively researching and surveying areas to be treated, is equipped to recommend the extent and type of facility that will provide the best and most economical service.

The Master Plan of Drainage consists of maps, text and appendices. The maps identify natural drainage areas and areas that presently or may in the future contribute runoff to facilities handling storm drainage. Methods employed and solutions indicated provide a schematic plan. Actual locations, alignments and types of facilities are not fixed. Design was avoided to maintain the maximum possible flexibility for accomplishing final planning. The text provides an historical sketch, the development and application of criteria and detailed discussion of the methods possible to analyze and solve problems related to handling storm runoff. Cost estimates are presented following the text. All of this information gives planners and designers a basis for making final decisions. Appendices are both technical and non-technical. Charts, graphs, tables, discussion and the development of various data and criteria constitute the technical portion. Maps and comparative analyses make up the non-technical presentation. The overall presentation provides a comprehensive analysis of all physical and ap-

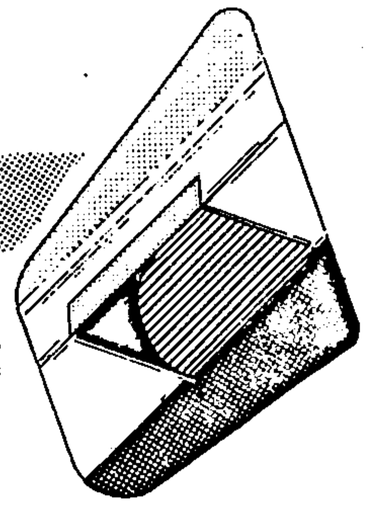


plied information and data which establish the basis for handling storm runoff. The use of this plan will enable planners and designers to visualize the entire problem with all of its ramifications. Therefore, planning and designing to handle storm runoff for future developments within the treated area can be more easily accomplished.

COSTS AND FINANCING

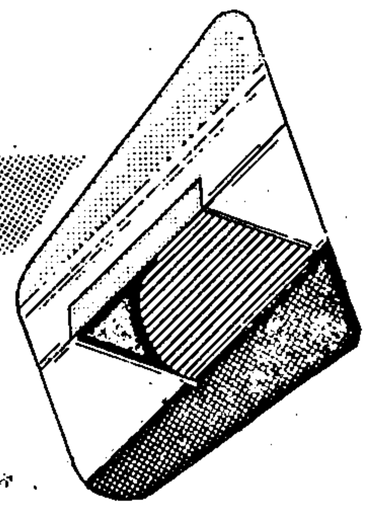
ESTIMATES: Estimates of cost for constructing required drainage facilities in the various portions of the master plan area follow this text. These estimates are based upon - 1961-1962 bid and construction prices.

Estimates include costs for facilities and appurtenances required to serve areas indicated on the maps. No costs for drainage channel construction or rights-of-way are included except those for major diversion ditches. This conforms with present City policy which states that costs of rights-of-way and construction for these carriers shall be the responsibility of the owner or developer of the land through which they pass. Costs of structures which may be required at intersections of residential and collector streets with drainage channels are not included. Estimates contain only those costs for structures required at half-mile intervals on drainage channels. Major carrier facilities, such as major diversion ditches and the San Mateo Storm Sewer are shown as individual estimates.



No costs for any portion of these projects is reflected in area estimates.

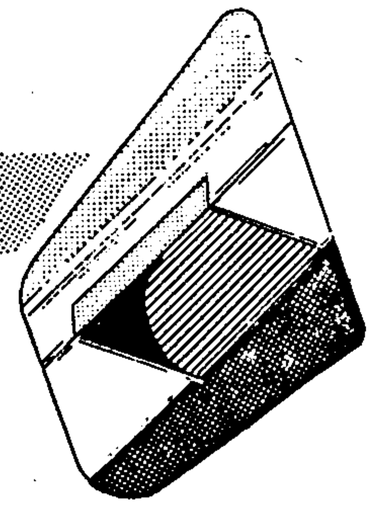
Estimates for undeveloped areas are extremely difficult to prepare, because there is no indication of the type of development which will occur. Costs for drainage facilities in these areas were based upon average requirements when a grid-system type of development is applied. Main carriers in these areas were designated as drainage channels. Any variations from this method will increase costs for providing facilities. Chart XI of Appendix A indicates average requirements. Chart XI-B shows, in detail, the estimate prepared for this scheme. This application was checked in a 69 square mile area in the northwest corner of the master plan area. This demonstrated that the method used for estimating was feasible, although costs may be somewhat lower if all facilities are designed to permit their most efficient function. An average of these two applications may be more realistic. However, interpretation in any specific area would have been more difficult. Also, it is not realistic to assume that pure function with regard to drainage can be allowed to dictate in all plans for land development. Therefore, the higher estimates indicated will probably more closely approximate actual requirements. These estimates must be applied with discretion, since areas with high average gradients will



require smaller expenditures than areas with lower average slopes. Thus, any application to small areas must be adjusted accordingly.

Estimates for construction required in developed "Heights" areas are more realistic since the configuration of development is fixed. Estimates in these areas were prepared based upon actual requirements with existing conditions unaltered. Previous discussions have indicated how modifications at some street intersections can completely change existing drainage patterns. If these alterations were made, estimates would vary considerably. Drainage patterns in large planned areas depend largely upon street grading design, thus many variations are also possible in these areas.

Costs in "Valley" areas are also subject to considerable variation due to the extent of facilities required to provide local services. However, it is probable that many channel improvements will begin downstream from areas which are served, since no substantial accumulation of runoff is possible in small local areas. If total development of the area were instantaneous, this would not be a factor. However, with scattered development, requirements for downstream improvements will take precedence. Branch storm sewer lines in developed "Valley" areas are based upon the average length required to serve areas 500 feet in width, unless developed streets oriented toward the channels are closer

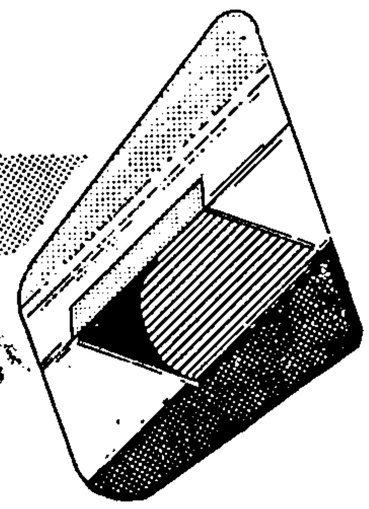


together. Criteria applied for undeveloped "Valley" areas was followed closely, thereby requiring no extensive branch systems. Streets carry and discharge runoff from the 500 foot wide area directly into ditches. Thus, the major requirements in these areas are ditch improvements.

Special treatments, such as ponding and pumping facilities and major diversion installations, are covered by individual estimates. Estimates for costs for rights-of-way, easements and land purchases for these facilities are included. It is anticipated that these estimates will vary considerably from the final costs. Only detailed design will provide sufficient information for preparing accurate estimates.

The estimates prepared are not intended for construction purposes. They will be extremely valuable in determining bond issue requirements and responsibilities inherited when areas are annexed. Cost estimates of this nature must not be applied to small areas, since they have been developed from schematic and average conditions.

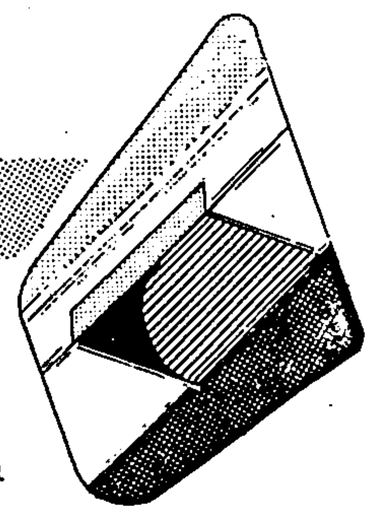
FINANCING: Financing facilities to handle storm runoff is the major problem encountered in implementing any drainage plan. Most of the existing system of storm sewers has been financed by general obligation bonds authorized by property owners in the community. This



method is undoubtedly the most equitable that can be employed. However, the cost and nature of some individual requirements make it impossible to use this as the sole method of financing. Therefore, it is imperative that some alternate method or methods be devised to finance extraordinary improvements.

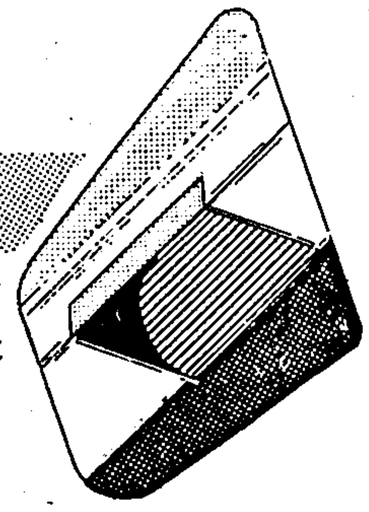
Financing through special assessment districts has been considered. However, recent legal opinions renders this method inappropriate. Strict interpretations of "benefit clauses" within existing statutes prevents financing by this method. Unless storm runoff can be confined within the area where it falls, equitable distribution of benefits is almost impossible. Some areas considered in this study fall into this category, but no general application can be made.

Attempting to finance the entire program with existing ad valorem property taxes creates several problems. The first difficulty arises from the assessed valuation established for properties in this County. The average valuation in the County is about 16% of full value. Since all limitations on bonding refer to assessed valuations, considerable restriction is imposed on bonding capacities. A major benefit can be accured if assessed valuations are raised. It is a common belief that this would automatically raise taxes. However, since general obligation bonds are voted in spe-



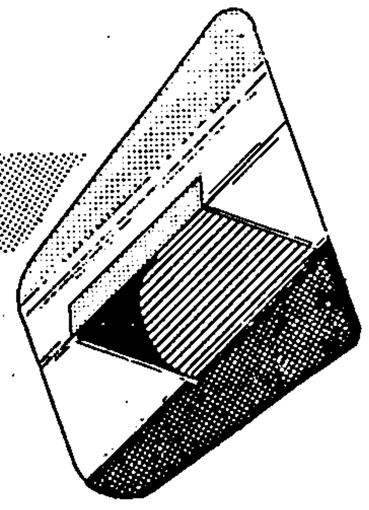
cific amounts, the tax on these amounts could not increase. The only existing provision by which taxes could be raised is through the 20 mil levy. However, this provision is a limitation, not a requirement. Thus, lowering the rate of this levy would produce revenues identical to those presently derived. There is no constitutional limitation on the value of bonds that can be authorized for sewer and water construction, but there is a 4% limitation on indebtedness for other improvements. There are a number of other vital community services which must be financed under this 4% limitation. A few of these are rights-of-way and improvements for streets and construction of libraries, parks and public buildings. These requirements alone preclude using these funds for any other purposes, unless its potential can be increased by raising assessed valuations. A Constitutional Amendment is required to raise this limitation. This is subject to a vote of the people and most recent attempts to amend the Constitution have been defeated at the polls. Therefore, financing drainage facilities through general obligation bonding is effective only to provide local services and some other method is required to finance the construction of major community installations.

Modifications to existing statutes pertaining to special assessment districts were considered. However, the basic philosphy of



this type of legislation must be founded upon the concept of "cost assessed for benefits received". This effectively precludes financing by special assessments.

New legislation recognizing problems encountered in existing statutes and constitutional provisions appears to be the only alternative remaining. This legislation must develop a method of financing without requiring difficult or impossible modifications to existing laws. The creation of a new political subdivision, which would include all of that portion of Bernalillo County treated in this plan, has considerable merit. Although it would physically overlap other political subdivisions, a new authority which would have no bonded indebtedness would be created. The property owning electorate of this new subdivision could approve bond issues for the construction of open channels without exceeding constitutional limitations or forestalling other necessary public works. No benefits would be assessed, since the projects financed would be classified as general improvements. This new legislation would permit financing several flood control projects which have been delayed for several years due to a lack of funds with which to proceed. The construction of these and other major installations for handling storm runoff would further the overall program tremendously. This proposal is undoubtedly the most feasible solution to the problem of



financing.

RECOMMENDATIONS

Implementing this plan requires that certain policies be adopted. It is recommended that these policies include the following:

1. Chart I in Appendix A shall be used for determining rainfall intensities. This chart should be revised every 10 years to reflect additional collected data.

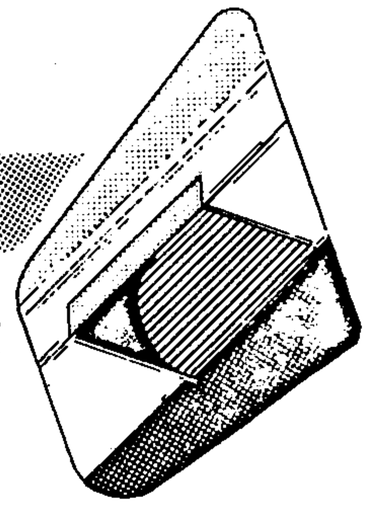
2. Chart III in Appendix A shall be used to determine maximum probable runoff expected for this area.

3. Streets shall be designed with a minimum carrying capacity equal to the runoff expected from their contributing area for the 25 year storm. Storm sewers shall be provided when required to insure this degree of protection.

4. Streets shall be designed so that total possible flows are discharged into drainage channels where they intersect. Runoff carried by streets shall be directed to drainage channels via the shortest possible route.

5. Residential streets and adjacent buildings shall be designed so that the street combined with the setback area will carry 100 year storm runoff from its contributing area.

6. Branch storm sewer systems shall be designed to intercept and transport storm runoff in excess of that carried by the



streets in the 25 year storm, and shall completely drain streets of 25 year storm runoff at points to be determined from design considerations.

7. Drainage channels, other than major diversion installations, shall be constructed to carry runoff resulting from the 100 year storm.

8. Runoff accumulated within any new subdivision shall be routed toward the nearest drainage easement.

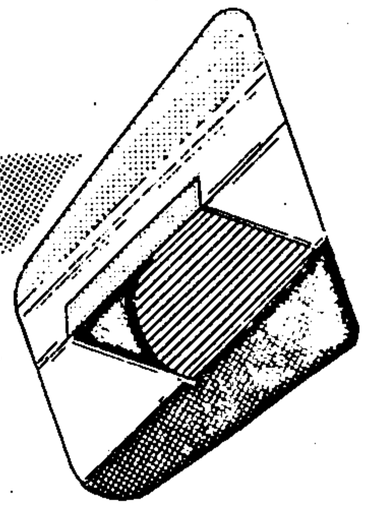
9. Drainage structures shall be designed to pass runoff from the 100 year storm and their conduits shall be capable of handling a minimum of 25 year runoff.

10. The City Engineer shall establish criteria and approve designs for channel construction and alignment as well as street design. Particular care must be exercised in setting street grades in conjunction with drainage structures.

11. The City Engineer shall establish criteria for and approve all design and profiles relating to drainage channels.

12. Major diversion channels and ponding areas shall be designed to accommodate maximum probable runoff as established by the Corps of Engineers.

13. Plans for handling storm runoff in each proposed subdivision shall be required as a part of the preliminary submittal for



approval by the Planning Commission. The City Engineer shall act as the advisory agent for the Commission with regard to these plans.

14. The City Engineer shall establish uniform criteria for analyzing storm runoff and carrying capacities of facilities (Table XII in Appendix A sets forth suggestions in this regard).

15. The City Engineer shall evaluate the feasibility and efficiency of all proposed rights-of-way and easements to be used for drainage purposes, and shall advise the Planning Commission and City Commission with regard to their effectiveness in fulfilling the purpose for which they are intended.

16. Negotiations with the Middle Rio Grande Conservancy District shall be opened in order to establish the extent to which their facilities can be used for handling storm runoff.

17. The Corps of Engineers plan for the North and South Diversion Channels, with required modifications, shall be adopted.

18. The mechanics required to create a new political subdivision for the purpose of financing drainage works which cannot be otherwise financed shall be initiated.

19. This Master Plan of Drainage for the City of Albuquerque, New Mexico and environs shall be adopted as the official planning document for drainage control.

THE MASTER PLAN OF DRAINAGE

CITY OF ALBUQUERQUE, NEW MEXICO

AND ENVIRONS

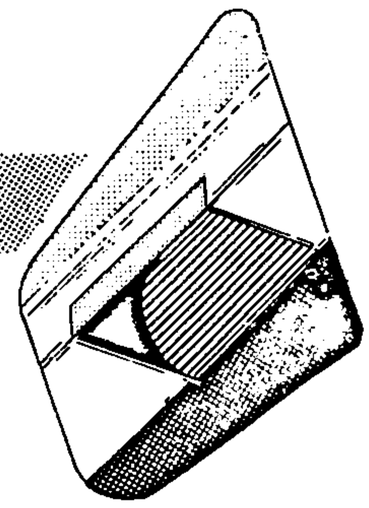
CONSTRUCTION, MAINTENANCE AND OPERATION

COST ESTIMATES

PREFACE

The estimates of costs for construction and annual maintenance and operation for primary drainage facilities shown on the following sheets were prepared to conform with criteria established in the preceding text.

Sheet 1 is a recapitulation by area of the estimated construction costs based upon 1961-62 bid and construction prices along with the anticipated average annual maintenance costs. Chart XIV in Appendix A is a base map with an overlay indicating area designations. Sheets 2 to 14, inclusive, tabulate the breakdown of construction costs of the various facilities within each area.



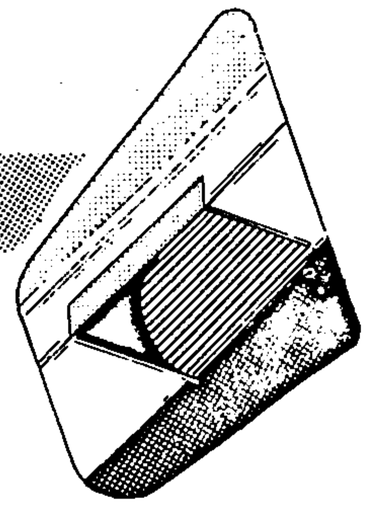
MASTER PLAN OF DRAINAGE

FOR

ALBUQUERQUE, NEW MEXICO
AND
ENVIRONS

RECAPITULATION:

Area	Avg. Annual Operation and Maintenance Costs	Construction First Cost
Northeast Area 1 (N.E. 1)	$ 3,050.00	$ 225,000.00
Northeast Area 2 (N.E. 2)	13,900.00	3,196,900.00
Northeast Area 3 (N.E. 3)	12,650.00	2,647,260.00
Northeast Area 4 (N.E. 4)	7,950.00	2,048,920.00
Northeast Area 5 (N.E. 5)	8,500.00	1,495,920.00
Northeast Area 6 (N.E. 6)	19,300.00	3,473,000.00
Northeast Area 7 (N.E. 7)	65,000.00	14,394,000.00
East Central Area 1 (E.C. 1)	5,000.00	2,788,000.00
East Central Area 2 (E.C. 2)	28,000.00	3,100,000.00
East Central Area 3 (E.C. 3)	220,500.00	9,932,000.00
Southeast Area 1 (S.E. 1)	4,800.00	318,200.00
Southeast Area 2 (S.E. 2)	178,000.00	8,952,000.00
Southeast Area 3 (S.E. 3)	20,900.00	5,562,440.00
Southeast Area 4 (S.E. 4)	34,600.00	2,661,220.00
Northwest Area 1 (N.W. 1)	32,200.00	8,380,200.00
Northwest Area 2 (N.W. 2)	59,500.00	11,849,110.00
Northwest Area 3 (N.W. 3)	31,100.00	8,389,830.00
Southwest Area 1 (S.W. 1)	227,100.00	14,504,800.00
Southwest Area 2 (S.W. 2)	114,400.00	10,317,580.00
Southwest Area 3 (S.W. 3)	25,700.00	6,911,430.00
Southwest Area 4 (S.W. 4)	31,900.00	9,057,760.00
Interstate 40 Diversion Ditch (West of the River)	6,500.00	1,901,250.00
Southwest Diversion Ditch	31,500.00	7,953,010.00
Southeast Diversion Ditch	16,000.00	2,610,000.00
North Diversion Ditch (Local participation only)	27,500.00	6,680,000.00
South Diversion Ditch (Local participation only)	7,700.00	2,312,000.00
San Mateo Interceptor Storm Sewer	2,800.00	2,530,000.00
TOTALS	$1,236,050.00	$154,191,830.00



CONSTRUCTION COST

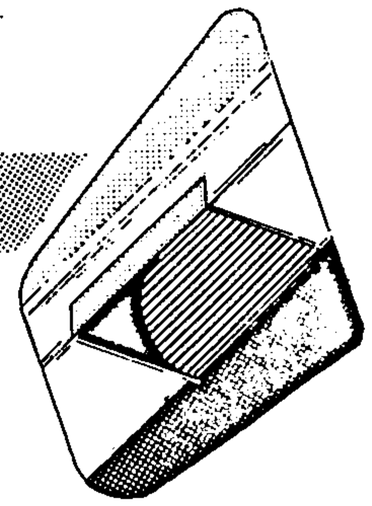
ESTIMATE BREAKDOWNS

Northeast Area 1 (NE 1): Elena Gallegos Grant Mountain Area

Street Crossing Structures	$225,000.00
Total	$225,000.00

Northeast Area 2 (NE 2): Elena Gallegos Grant Eubank Blvd. to Mountains

Inlets	$190,772.00
Manholes	95,119.00
Concrete Junction Boxes	42,750.00
15" through 36" piping	675,878.00
42" through 60" piping	769,179.00
66" and larger piping	153,045.00
Outlet structures for piping	80,157.00
Street crossing structures	840,000.00
Lined diversion ditch (including excavation)	276,000.00
Right-of-way for diversion ditch	74,000.00
Total	$3,196,900.00



CONSTRUCTION COST

ESTIMATE BREAKDOWNS

Northeast Area 3 (NE 3): Platted Area Albuquerque Estates

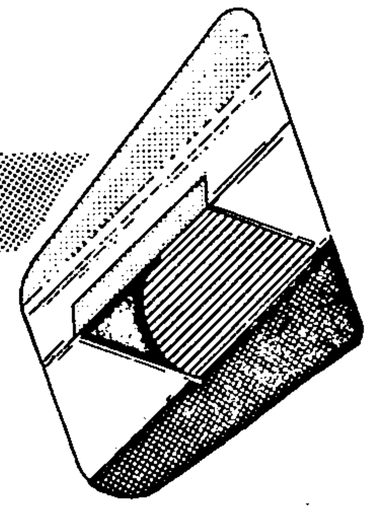
The estimate that follows is based upon intercepting runoff on high slope east-west streets.

Inlets	$348,740.00
Manholes	78,400.00
Concrete junction boxes	10,000.00
15" through 36" piping	367,250.00
42" through 60" piping	340,450.00
66" and larger piping	711,170.00
Outlet structures for piping	131,250.00
Street crossing structures	660,000.00
Total	$2,647,260.00

Alternate Northeast Area 3 (NE 3): Platted Area Albuquerque Estates

The estimate that follows is based upon grading east-west streets so that drainage is turned onto north-south streets at one mile intervals. Runoff is then intercepted on these low slope north-south streets.

Inlets	$282,000.00
Manholes	62,800.00
Concrete junction boxes	10,000.00
15" through 36" piping	53,625.00
42" through 60" piping	238,700.00
66" and larger piping	711,170.00
Outlet structures for piping	131,250.00
Street crossing structures	660,000.00
Total	$2,149,545.00



CONSTRUCTION COST

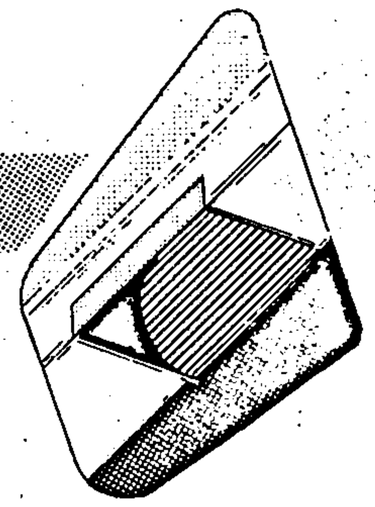
ESTIMATE BREAKDOWNS

Northeast Area 4 (NE 4): Master Planned Area
Elena Gallegos Grant

Inlets	$ 200,220.00
Manholes	46,400.00
Concrete junction boxes	10,000.00
15" through 36" piping	129,440.00
42" through 60" piping	691,020.00
66" and larger piping	409,340.00
Piping outlet structures	142,500.00
Street crossing structures	420,000.00
Total	$2,048,920.00

Northeast Area 5 (NE 5): Elena Gallegos Grant
Eubank to North Diversion Ditch

Inlets	$ 102,275.00
Manholes	50,994.00
Concrete junction boxes	22,919.00
15" through 36" piping	362,346.00
42" through 60" piping	412,366.00
66" and larger piping	82,049.00
Piping outlet structures	42,971.00
Street Crossing structures	420,000.00
Total	$1,495,920.00



CONSTRUCTION COST

ESTIMATE BREAKDOWNS

Northeast Area 6 (NE 6): Alignment Tulane St., Yale Blvd., and Comanche Rd.

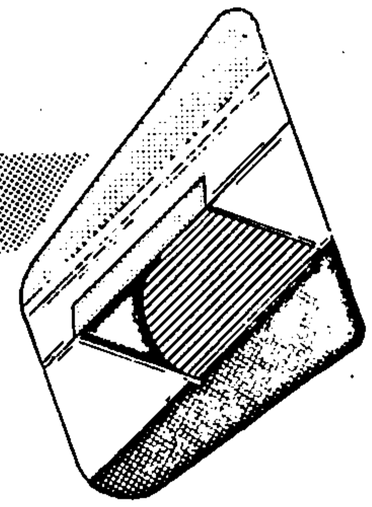
Alameda Quadrangle

1. Carlisle Blvd. - Montgomery north to arroyo	$ 92,000.00
2. Washington Blvd. - Montgomery north to arroyo	160,000.00
3. Juan Tabo Blvd. - Montgomery south to arroyo	120,000.00
4. Albuquerque Main Canal Alignment	
Excavation	43,000.00
Structures	240,000.00
Ditch Linings	358,000.00
Right-of-Way	150,000.00
Miscellaneous Storm Sewer Extensions	165,000.00
5. Alameda Ditch Alignment	
Excavation	25,000.00
Structures	640,000.00
Ditch Lining	945,000.00
Miscellaneous Storm Sewer Extensions	435,000.00
6. Arroyo Crossings	100,000.00
Total	$3,473,000.00

Northeast Area 7 (NE 7): Alignment Yale Blvd., Comanche Rd. to the River.

Los Griegos Quadrangle

1. Albuquerque Main Canal Alignment	
Excavation	$ 136,000.00
Structures	630,000.00
Ditch Linings	1,138,000.00
Right-of-Way	482,000.00
Minor Storm Sewer Extensions	525,000.00
2. Alameda Ditch Alignment	
Excavation	384,000.00
Structures	1,040,000.00
Ditch Linings	1,560,000.00
Minor Storm Sewer Extensions	720,000.00



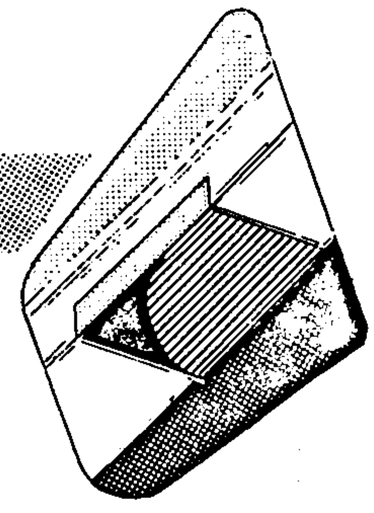
Los Griegos Quadrangle - continued

3. Los Griegos Lake	
Excavation	$ 750,000.00
Right-of-Way	3,600,000.00
Well	60,000.00
4. Los Griegos Pump Station	
Pump Station	500,000.00
Main Line Storm Sewer	500,000.00
5. Duranes Lateral	
Excavation	21,000.00
Structures	175,000.00
Ditch Linings	455,000.00
Miscellaneous Storm Sewer Extensions	210,000.00
6. Griegos Ditch	
Excavation	30,000.00
Structures	100,000.00
Ditch Linings	410,000.00
Miscellaneous Storm Sewer Extensions	75,000.00
7. Los Poblanos Ditch	
Excavation	40,000.00
Structures	150,000.00
Ditch Linings	553,000.00
Miscellaneous Storm Sewer Extensions	150,000.00
Total	$14,394,000.00

East Central Area 1 (E.C. 1): Alignment Gibson Avenue and Comanche Rd. from Forest boundary to Panorama Blvd.

Tijeras Quadrangle

1. Panorama Blvd.	
a. North of first arroyo north of Candelaria	$ 80,000.00
b. North of Piedra Lisa Canyon arroyo	100,000.00
c. South of Piedra Lisa Canyon arroyo	120,000.00
d. Freeway to north of Lomas Blvd., NE	380,000.00



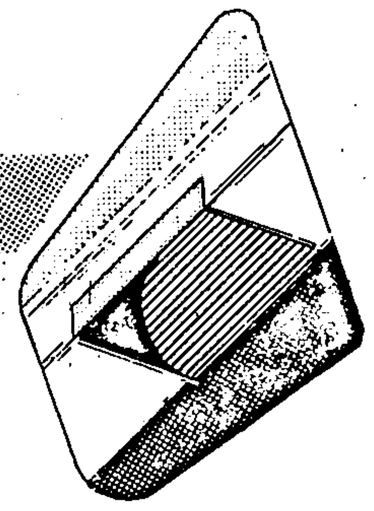
Tijeras Quadrangle - Continued

2. Monte Largo Alignment and Extension	
a. Storm Sewer	$ 175,000.00
b. Ditch	
Right-of-Way	180,000.00
Excavation	335,000.00
Structures	450,000.00
Ditch Lining	618,000.00
3. Arroyo Crossings	350,000.00
Total	$2,788,000.00

East Central Area 2 (E.C. 2): Alignment Gibson Avenue and Comanche Rd. from Panorama Blvd. to Yale Blvd.

Albuquerque - East Quadrangle

1. Wyoming Blvd. - Utah, Lomas, Wyoming Copper	$ 226,000.00
2. Juan Tabo Blvd. - Lomas to Haines	265,000.00
3. Chelwood Blvd.	
a. Lomas - North	225,000.00
b. North fork of Embudo (Sec. 10, north of Menaul)	254,000.00
c. North fork of Embudo (North of Candelaria)	153,000.00
4. Eubank Blvd.	
a. Extension - Copper east to Juan Tabo	240,000.00
b. Embudo - North of Menaul	160,000.00
c. Ditch, SE - Central to Tijeras Arroyo	770,000.00
5. Luthy Drive - Indian School to Embudo	100,000.00
6. Louisiana Blvd. - Menaul, Pius X, Freeway	272,000.00
7. Arroyo Structures	435,000.00
Total	$3,100,000.00



East Central Area 3 (E.C. 3): Alignment of Yale Blvd., Tijeras Arroyo, The River and the Alignment of Comanche Road

Albuquerque West Quadrangle

1. Metropolitan Area bounded essentially on the north by Menaul Blvd., on the east by the Santa Fe Railroad Tracks, on the south by Barelas Road and on the west by Riverside Drain, U. S. 66 Bridge, Central Avenue, West, Rio Grande to 12th Street, N. W.

$2,300,000.00

2. I-025 Cut-off area is essentially that area east of the North-South Freeway called Interstate Route 25 and further bounded on the south by Miles Road and a portion of Kirtland Air Force Base Military Reservation, on the east the contiguous Campus Arroyo watershed, and on the north the Encino Medial Arts Square and the contiguous Campus Arroyo watershed.

Storm Sewer	$ 970,000.00
Right-of-Way	46,000.00
Excavation	34,000.00
Ditch Lining	250,000.00

3. Duranes Lateral Alignment - South of Coronado Freeway

Excavation	$ 25,000.00
Structures	100,000.00

4. Alameda Drain - South of Coronado Freeway

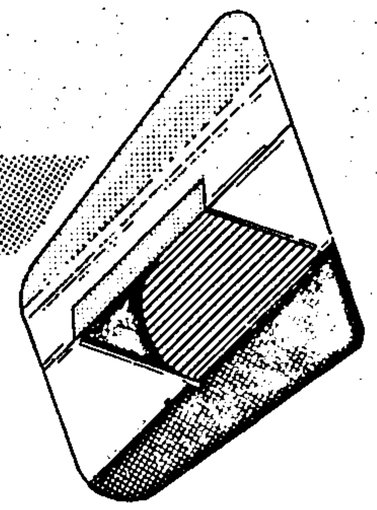
Excavation	$ 25,000.00
Structures	87,000.00

5. Alameda Drain - Coronado Freeway North to Mathew Blvd.

Excavation	$ 55,000.00
Structures	80,000.00

6. Griegos Drain - Mathew to North Quad Line

Excavation	$ 13,000.00
Structures	50,000.00
Ditch Lining	195,000.00



Albuquerque West Quadrangle - Continued

7. Main Alameda

Excavation	$ 270,000.00
Structures	400,000.00
Ditch Lining	1,105,000.00
Right-of-Way	165,000.00
Minor Storm Sewer Extensions	510,000.00

8. San Jose Ditch

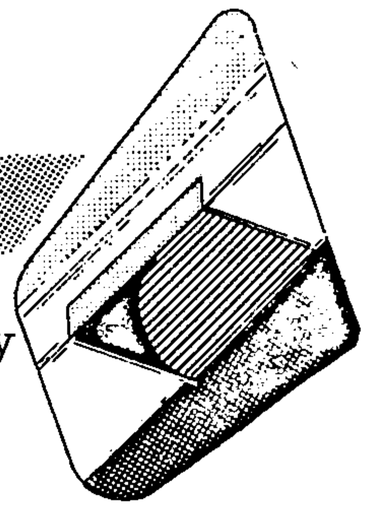
Excavation	$ 254,000.00
Structures	700,000.00
Ditch Linings	1,300,000.00
Right-of-Way	144,000.00
Minor Storm Sewer Extensions	30,000.00

9. Barr Ditch

Excavation	$ 21,000.00
Structures	90,000.00
Ditch Lining	455,000.00
Right-of-Way	48,000.00
Minor Storm Sewer Extensions	210,000.00
Total	$9,932,000.00

Southeast Area 1 (S.E. 1): Forest Boundary, Military Reservation, Eubank Blvd., and Interstate Highway 40.

Inlets	$ 20,000.00
Manholes	10,000.00
15" through 36" piping	65,500.00
Piping outlet structures	8,200.00
Street crossing structures	214,500.00
Total	$ 318,200.00



Southeast Area 2 (S.E. 2): Military Reservation, Isleta Pueblo Grant Boundary, Alignment University Blvd., Tijeras Arroyo

Right-of-Way	$4,800,000.00
Excavation	1,292,500.00
Forced Lines (piping)	759,500.00
Pumping Facilities	1,700,000.00
Street Crossing Structures	400,000.00
Total	$8,952,000.00

Southeast Area 3 (S.E. 3): Alignment University Blvd., Isleta Pueblo Grant Boundary Line, A.T. & S.F. Railway Main Line, Tijeras Arroyo

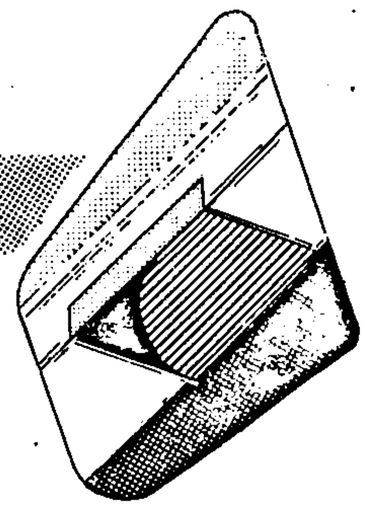
Inlets	400,426.00
Manholes	199,652.00
Concrete junction boxes	89,731.00
15" through 36" piping	1,418,651.00
42" through 60" piping	1,614,490.00
66" and larger piping	321,238.00
Piping outlet structures	168,252.00
Street crossing structures	1,350,000.00
Total	$5,562,440.00

Southeast Area 4 (S.E. 4): A.T. &S.F. Railway Main Line, Isleta Pueblo Grant Line, River and Tijeras Arroyo

Complete Storm Sewer Systems	$ 397,800.00
Street Crossing Structures	270,000.00
M.R.G.C.D. Drain Treatment	1,553,420.00
Pumping Facilities	440,000.00
Total	$2,661,220.00

Northwest Area 1 (N.W. 1): The River, Town of Alameda Grant Line, Drainage Divide between the Rio Puerco and Rio Grande Watersheds and the Bernalillo-Sandoval County Line

Inlets	$ 587,934.00
Manholes	293,144.00
Concrete Junction Boxes	131,750.00
15" through 36" piping	2,114,168.00



Northwest Area 1 (N. W. 1) - Continued

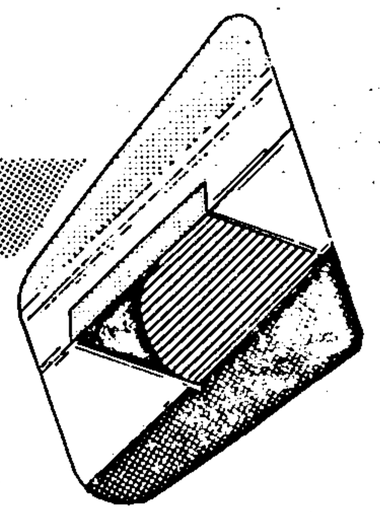
42" through 60" piping	$2,370,512.00
66" and larger piping	471,660.00
Piping outlet structures	247,032.00
Street Crossing structures	2,005,000.00
M. R. G. C. D. Drain Treatment	159,000.00
Total	$8,380,200.00

Northwest Area 2 (N. W. 2): The River, Atrisco Grant Line, Drainage Divide Between the Rio Puerco and Rio Grande Watersheds, and the Town of Alameda Grant Line.

Inlets	$ 729,200.00
Manholes	363,579.00
Concrete junction boxes	163,406.00
15" through 36" piping	2,791,453.00
42" through 60" piping	2,940,087.00
66" and larger piping	584,995.00
Piping outlet structures	306,390.00
Street crossing structures	2,660,000.00
M. R. G. C. D. Drain Treatment	1,060,000.00
Pumping Facilities	250,000.00
Total	$11,849,110.00

Northwest Area 3 (N. W. 3): The River, Interstate Highway 40 and U. S. Highway 66, the Drainage Divide between the Rio Puerco and Rio Grande Watersheds and the Atrisco Grant Line.

Inlets	$ 606,454.00
Manholes	302,378.00
Concrete junction boxes	135,900.00
15" through 36" piping	2,148,579.00
36" through 60" piping	2,445,181.00
66" and larger piping	486,522.00
Piping outlet structures	254,816.00
Street crossing structures	2,010,000.00
Total	$8,389,830.00



Southwest Area 1 (S. W. 1): The River, Barcelona Road, Coors Road and Interstate Highway 40.

Complete storm sewer systems	$7,628,400.00
Street crossing structures	840,000.00
M.R.G.C.D. Drain Treatment	4,666,400.00
Pumping Facilities	1,370,000.00
Total	$14,504,800.00

Southwest Area 2 (S. W. 2): The River, Southwest Diversion Ditch, Coors Road and Barcelona Road.

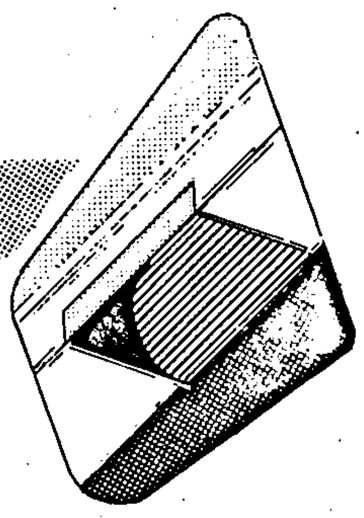
Complete storm sewer systems	$1,388,400.00
Street crossing structures	560,000.00
M.R.G.C.D. Drain Treatment	6,981,180.00
Pumping Facilities	1,388,000.00
Total	$10,317,580.00

Southwest Area 3 (S. W. 3): Southwest Diversion Ditch, Isleta Pueblo Grant Line, The Drainage Divide between The Rio Puerco and Rio Grande Watersheds, and the Atrisco Grant Line.

Inlets	$ 497,290.00
Manholes	247,948.00
Concrete junction boxes	111,442.00
15" through 36" piping	1,761,825.00
42" through 60" piping	2,005,040.00
66" and larger piping	398,940.00
Piping outlet structures	208,945.00
Street crossing structures	1,680,000.00
Total	$6,911,430.00

Southwest Area 4 (S. W. 4): Coors Road, Atrisco Grant Line, The Drainage Divide between the Rio Puerco and Rio Grande Watersheds, Interstate Highway 40 and U.S. Highway 66.

Inlets	$ 652,836.00
Manholes	325,504.00
Concrete junction boxes	146,294.00
15" through 36" piping	2,312,904.00
42" through 60" piping	2,632,190.00



Southwest Area 4 (S. W. 4) - Continued

66" and larger piping	$ 523,732.00
Piping outlet structures	274,300.00
Street crossing structures	2,190,000.00
Total	$9,057,760.00

Interstate 40 Diversion Ditch: Parallel to Interstate Highway 40 west of the River.

Right-of-Way	$ 450,000.00
Excavation	193,500.00
Concrete, rebar and subgrade preparation	1,257,750.00
Total	$1,901,250.00

Southwest Diversion Ditch: U. S. Highway 66 to the River at the Isleta Pueblo Grant Line.

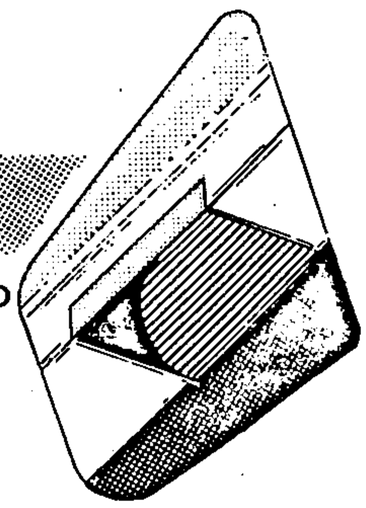
Right-of-Way	$1,290,000.00
Excavation	765,410.00
Concrete, rebar and subgrade preparation	4,877,600.00
Bridges	1,020,000.00
Total	$7,953,010.00

Southeast Diversion Ditch: Tijeras Arroyo to the River at the Isleta Pueblo Grant Line.

Right-of-Way	$ 510,000.00
Excavation	216,000.00
Concrete, rebar and subgrade preparation	1,404,000.00
Bridges	480,000.00
Total	$2,610,000.00

North Diversion Ditch: Indian School Road to River at Alameda

Local participation costs only	$6,680,000.00



South Diversion Ditch: Central Avenue to River at Tijeras Arroyo

Local participation costs only $2,312,000.00

San Mateo Interceptor Storm Sewer: Gibson Avenue to the Embudo Arroyo

$2,530,000.00

MASTER PLAN
OF
DRAINAGE
CITY OF ALBUQUERQUE, NEW MEXICO
AND ENVIRONS
1963

CHARTS, TABLES AND MAPS

GORDON HERKENHOFF AND ASSOCIATES
CONSULTING ENGINEERS
ALBUQUERQUE, NEW MEXICO

INDEX

APPENDIX A

MASTER PLAN OF DRAINAGE

CITY OF ALBUQUERQUE, NEW MEXICO

AND ENVIRONS

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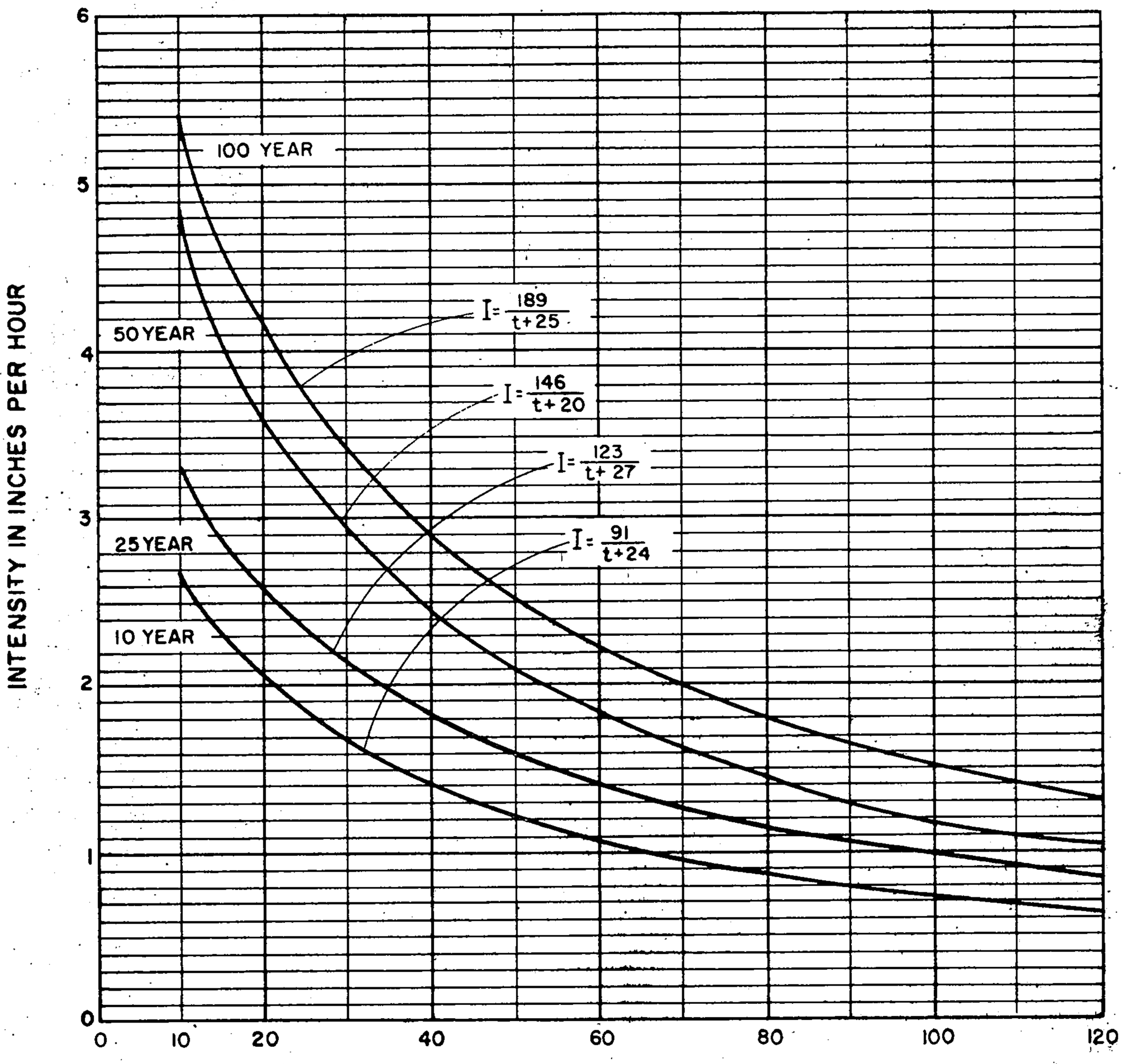
TABLE XII	Suggested Uniform Criteria For Analyzing Storm Runoff and Carrying Capacities of Facilities
CHART XIII	Map Delineating Areas Covered By Estimates

Meteorological Phenomena Relating to the Creation of Rainfall

Energy and Momentum of Gravity Flows

Technical Development Of Intensity-Duration-Frequency Relationships



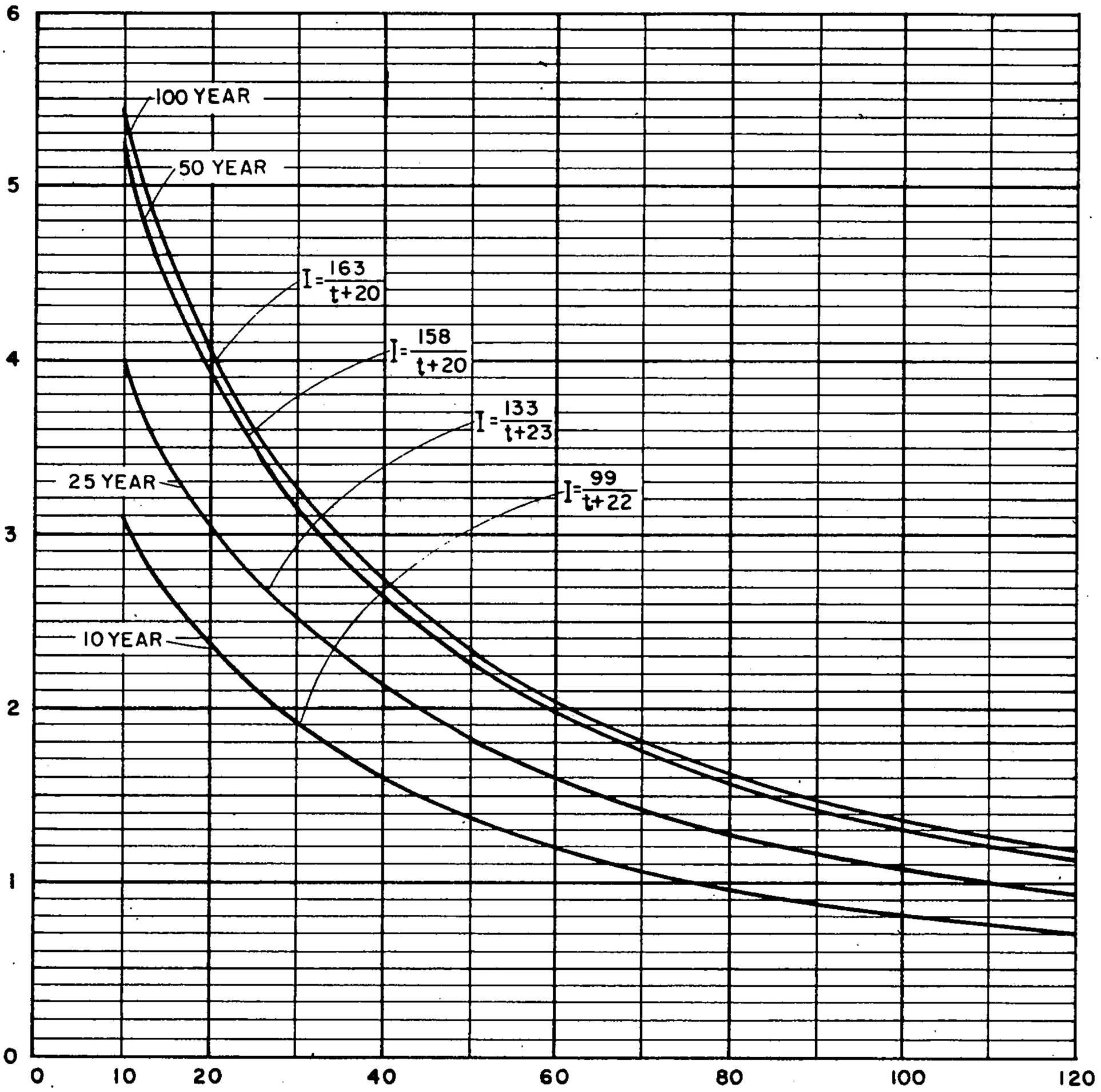
MASTER PLAN OF DRAINAGE
CITY OF ALBUQUERQUE - NEW MEXICO
AND ENVIRONS

INTENSITY DURATION
FREQUENCY CURVES

(ALBUQUERQUE AREA - 1961)

GORDON HERKENHOFF & ASSOC
CONSULTING ENGINEERS
ALBUQUERQUE, NEW MEXICO

CHART
1

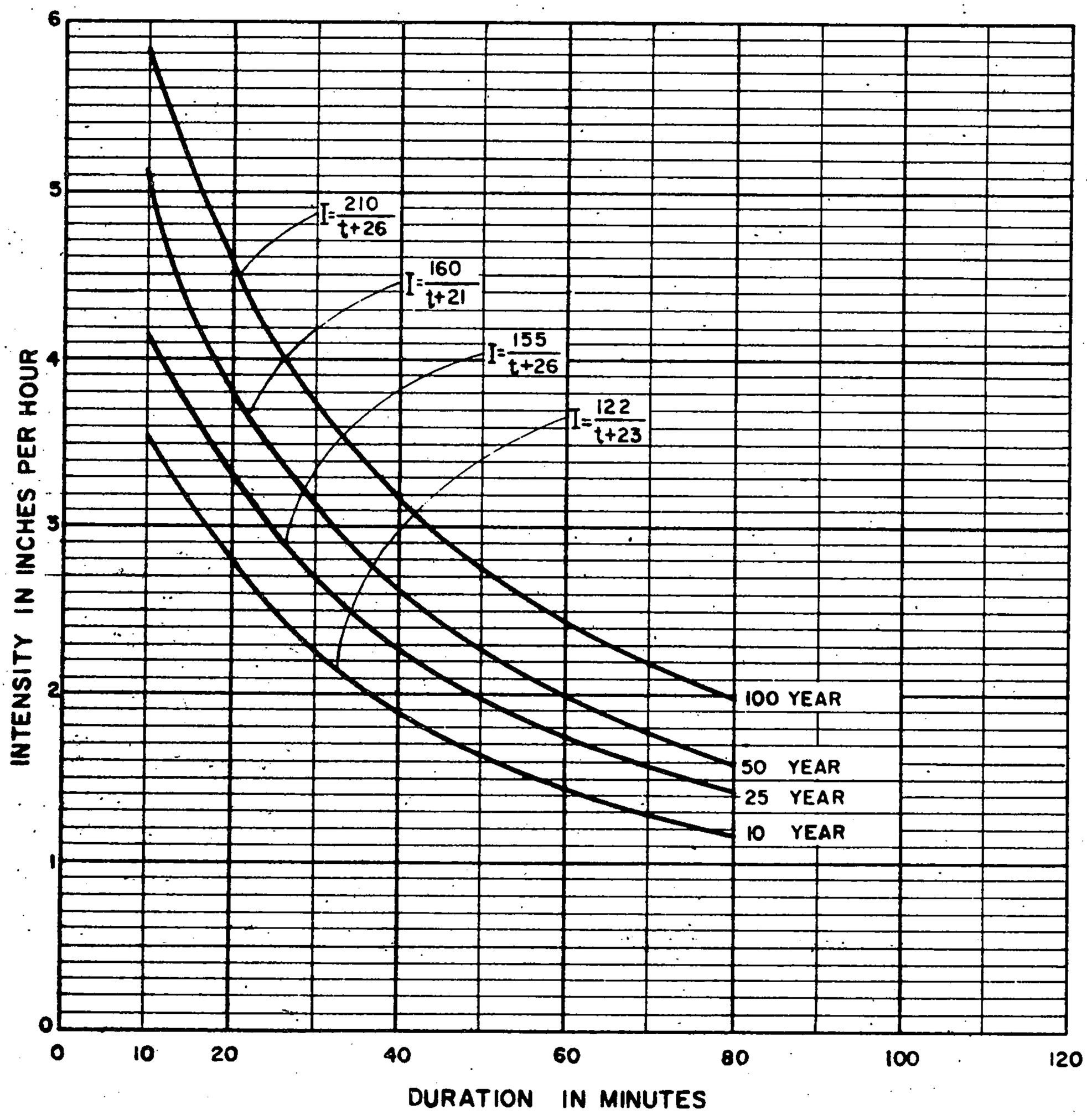


MASTER PLAN OF DRAINAGE
CITY OF ALBUQUERQUE - NEW MEXICO
AND ENVIRONS

INTENSITY DURATION
FREQUENCY CURVES
SANDIA CONSERVANCY DISTRICT
(ALBUQUERQUE, AREA-1954)

GORDON HERKENHOFF & ASSOC
CONSULTING ENGINEERS
ALBUQUERQUE, NEW MEXICO

CHART
1A

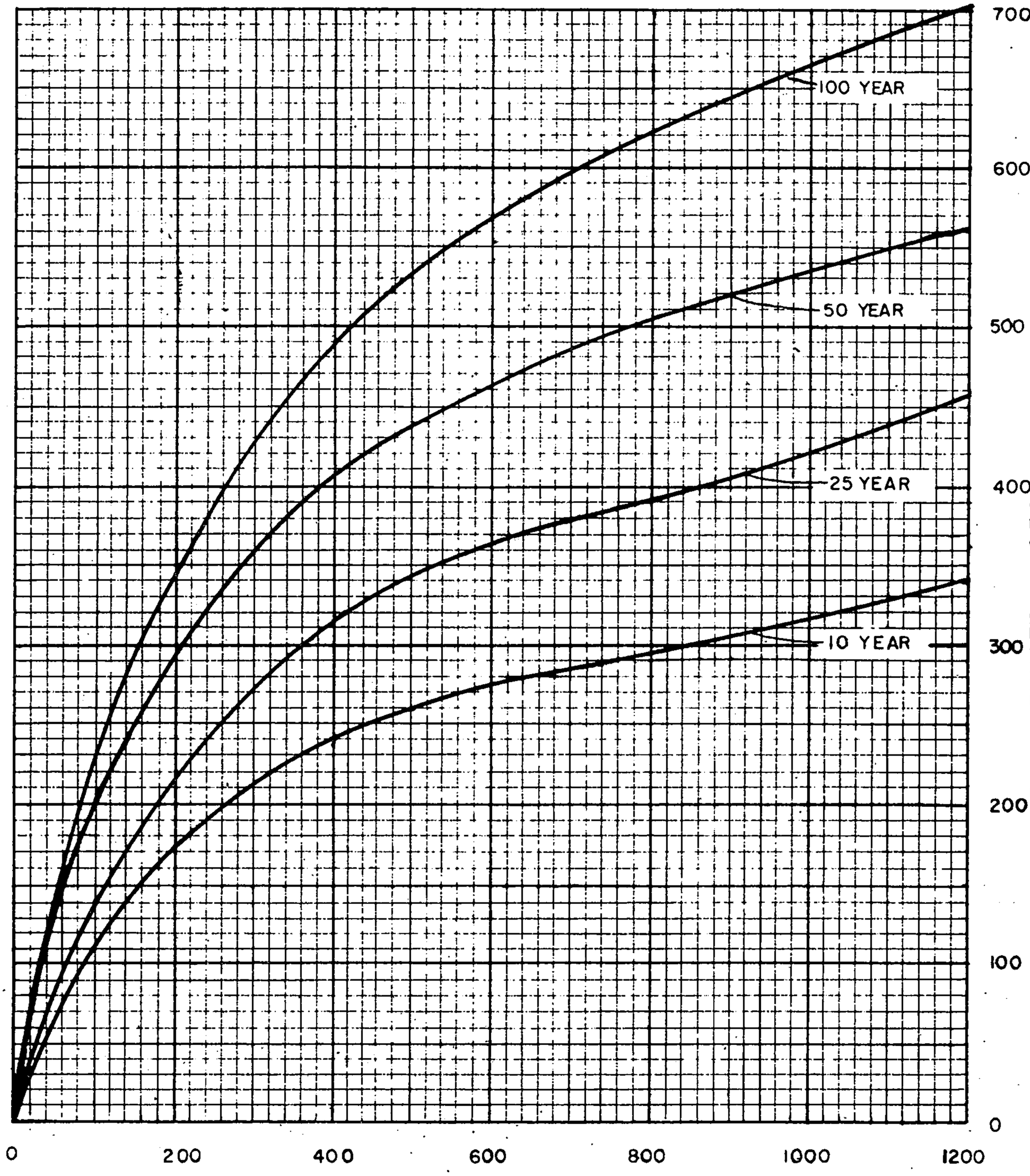


MASTER PLAN OF DRAINAGE
CITY OF ALBUQUERQUE – NEW MEXICO
AND ENVIRONS

INTENSITY DURATION FREQUENCY CURVES
SOUTHWESTERN U.S. – 1935
(STANDARD YARNELL)

GORDON HERKENHOFF & ASSOC
CONSULTING ENGINEERS
ALBUQUERQUE, NEW MEXICO

CHART
1B



THE PEAK RUNOFF ASSUMES A STRAIGHT LINE RELATIONSHIP WITH RESPECT TO AREA AFTER 1200 ACRES. THE PEAK RUNOFF FOR ANY AREA OVER 1200 ACRES MAY BE COMPUTED BY MULTIPLYING THE AREA BY THE FOLLOWING CONSTANTS FOR EACH FREQUENCY: 10YR = 0.28; 25YR. = 0.38; 50YR. = 0.47; 100YR. = 0.58

RUNOFF COEFFICIENT, R = 0.45 FOR THIS FAMILY OF CURVES.

INTENSITIES ARE BASED UPON THE INTENSITY DURATION CURVES ALBUQUERQUE AREA.

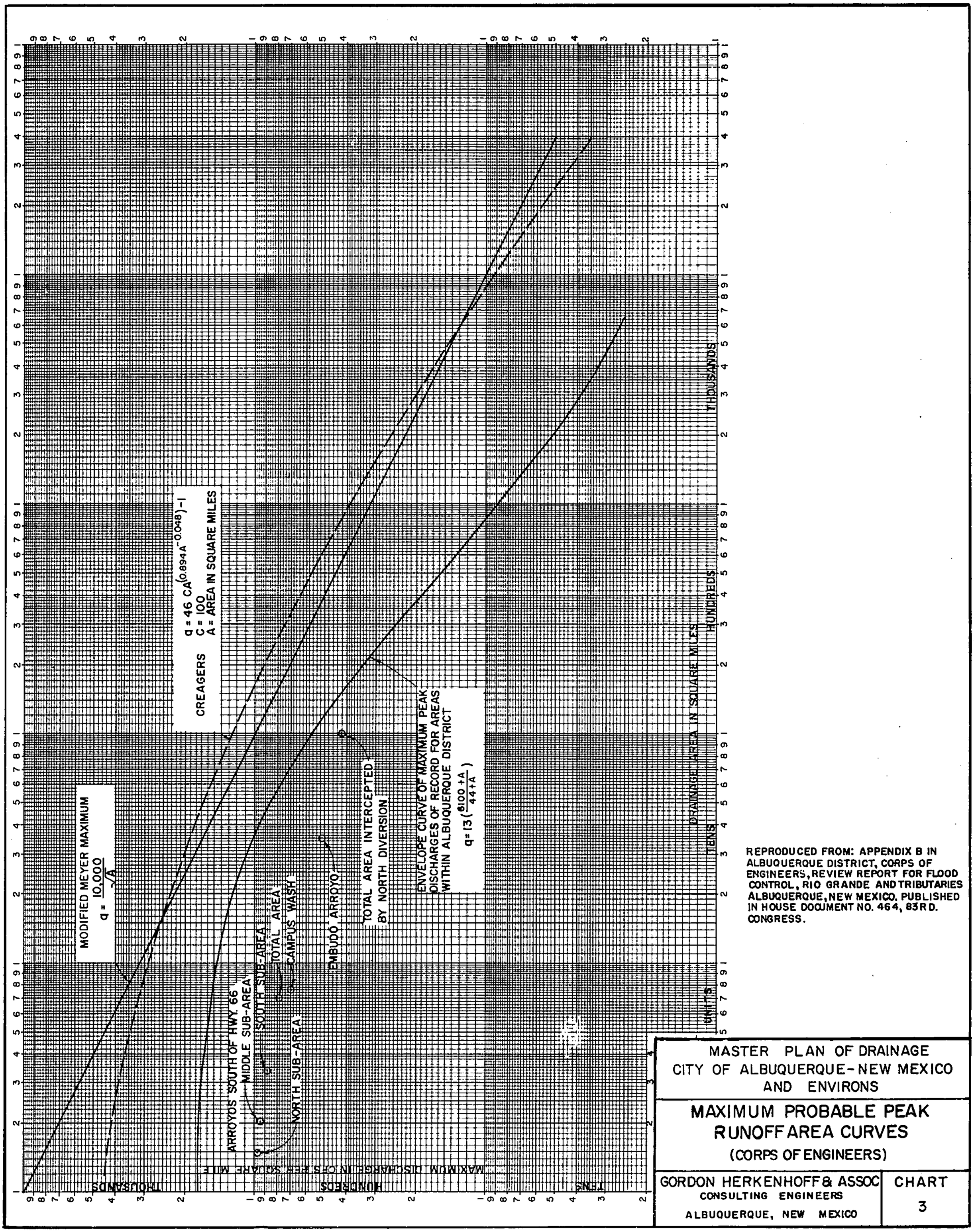
MASTER PLAN OF DRAINAGE
CITY OF ALBUQUERQUE – NEW MEXICO
AND ENVIRONS

PEAK RUNOFF · AREA FREQUENCY CURVES

ALBUQUERQUE AREA

GORDON HERKENHOFF & ASSOC.
CONSULTING ENGINEERS
ALBUQUERQUE, NEW MEXICO

CHART 2



REPRODUCED FROM: APPENDIX B IN ALBUQUERQUE DISTRICT, CORPS OF ENGINEERS, REVIEW REPORT FOR FLOOD CONTROL, RIO GRANDE AND TRIBUTARIES ALBUQUERQUE, NEW MEXICO. PUBLISHED IN HOUSE DOCUMENT NO. 464, 83RD. CONGRESS.

MASTER PLAN OF DRAINAGE
CITY OF ALBUQUERQUE - NEW MEXICO
AND ENVIRONS

MAXIMUM PROBABLE PEAK RUNOFF AREA CURVES
(CORPS OF ENGINEERS)

GORDON HERKENHOFF & ASSOC
CONSULTING ENGINEERS
ALBUQUERQUE, NEW MEXICO

CHART 3

TABLE IV

STREET CARRYING CAPACITIES
(CFS)

Slope	32' Street 6" Crown at center line or 6" sidehill with no crown	32' Street Flat Cross-section	32' Street 6" Inverted crown at center line	32' Street with 8" sidehill with no crown	32' Street with 12" sidehill with no crown	40' Street 6" Crown at center-line or 6" sidehill with no crown	44' Street 6" Crown at center-line or 6" sidehill with no crown	48' Street 6" Crown at center-line or 6" sidehill with no crown
0.0025	37	78	133	25	17	47	51	56
0.0050	53	111	189	36	23	66	72	79
0.0075	65	136	231	44	29	81	89	97
0.0100	75	156	267	51	33	93	102	112
0.0125	84	175	299	57	37	104	115	125
0.0150	92	193	328	62	41	114	126	138
0.0175	99	207	352	67	44	123	135	148
0.0200	105	221	376	71	47	131	144	158
0.0225	112	235	400	76	50	139	154	168
0.0250	118	247	422	80	52	147	162	177
0.0275	124	260	444	84	55	155	170	186
0.0300	129	271	462	88	57	161	177	193
0.0325	134	282	480	91	59	167	184	202
0.0350	139	293	499	95	62	174	191	209
0.0375	145	304	518	98	64	180	199	217
0.0400	149	313	534	101	66	186	205	224
0.0425	154	322	550	104	68	192	211	231
0.0450	158	332	566	107	70	197	217	237
0.0475	163	341	582	110	72	203	223	244
0.0500	167	350	598	113	74	208	230	251

Coefficient of Roughness = n = 0.015 for all streets.
Carrying Capacity Given in Cubic Feet per Second (C.F.S.)
Arterial Design (Stage & Final) has a +2% Slope from the Top of Curb to the top of Median Curb.
Curb Height = 8"

Table IV - Street Carrying Capacities (CFS)
Page 2

Slope	66' & 86' Arterial Stage Construction	66' Arterial Final Construction	86' Arterial Final Construction
0.0025	56	53	54
0.0050	79	74	76
0.0075	97	91	93
0.0100	112	105	108
0.0125	125	118	121
0.0150	138	130	133
0.0175	148	139	142
0.0200	158	148	152
0.0225	168	158	162
0.0250	177	166	170
0.0275	186	175	179
0.0300	193	182	186
0.0325	202	189	194
0.0350	209	197	202
0.0375	217	204	209
0.0400	224	210	215
0.0425	231	217	222
0.0450	237	223	228
0.0475	244	229	235
0.0500	251	236	241

Coefficient of Roughness = n = 0.015 for all streets.
Carrying Capacity given in Cubic Feet per Second (C.F.S.)
Arterial design (Stage & Final) has a +2% slope from the top of curb to the top of median curb
Curb Height = 8"

TABLE V

AVERAGE INTERCEPTING CAPACITY OF STANDARD INLETS

Type of Inlet	Average Intercepting Capacity in CFS
C	5
Double C	10
D	2.5
Double D	5
E	2.5
F	1

TABLE VI

FLOW IN PIPES (CFS)

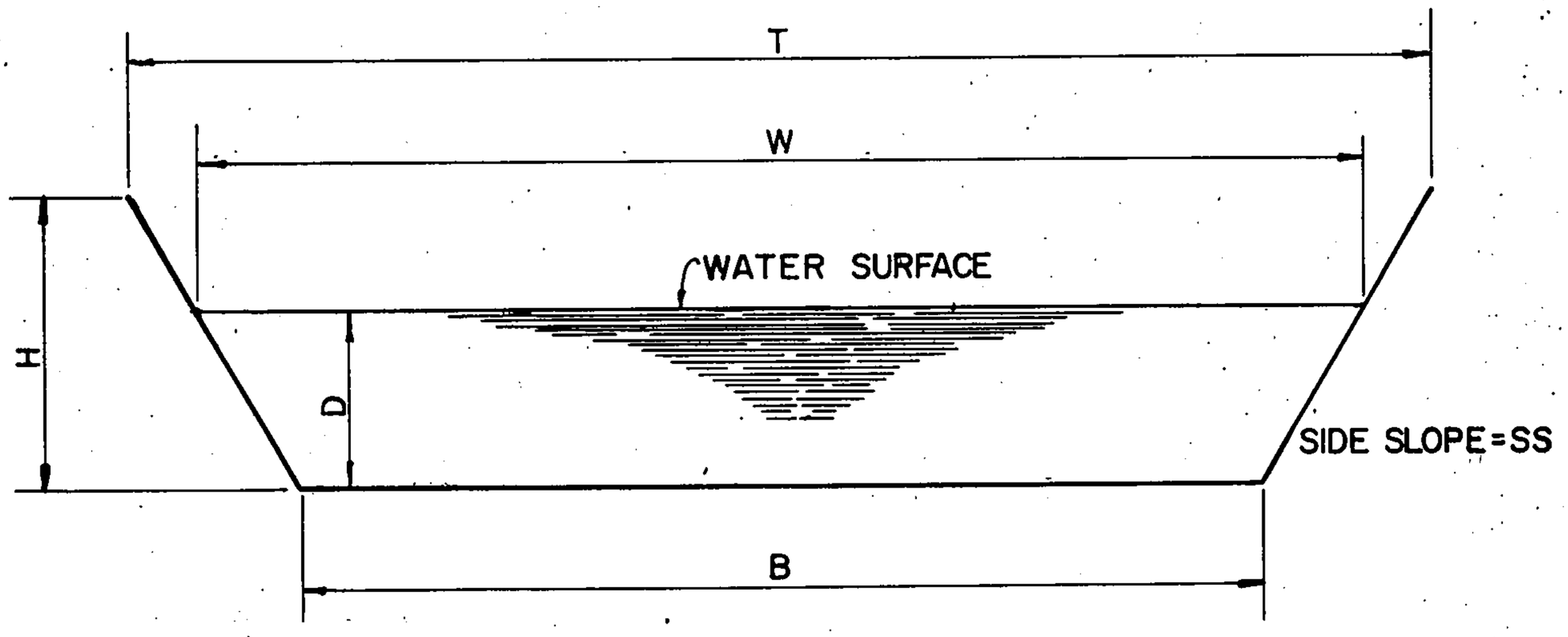
Slope		12" Pipe A=0.785	15" Pipe A=1.227	18" Pipe A=1.767	21" Pipe A=2.405	24" Pipe A=3.142	27" Pipe A=3.976	30" Pipe A=4.909	36" Pipe A=7.069	42" Pipe A=9.621	48" Pipe A=12.566
0.001	Q_c	1	2	3	5	7	10	13	21	32	45
	Q_m	1	1	2	3	4	6	8	13	20	28
0.003	Q_c	2	4	6	9	12	17	22	37	55	79
	Q_m	1	2	4	6	7	11	14	23	34	49
0.005	Q_c	2	5	7	11	16	22	29	47	72	101
	Q_m	1	3	4	7	10	14	18	29	45	63
0.007	Q_c	3	5	8	13	19	27	34	56	85	119
	Q_m	2	3	5	8	12	17	21	35	53	74
0.009	Q_c	3	6	9	15	21	30	39	63	96	135
	Q_m	2	4	6	9	13	19	24	39	60	84
0.010	Q_c	4	6	11	16	23	31	41	67	101	144
	Q_m	2	4	7	10	14	19	25	41	62	89
0.015	Q_c	4	8	13	19	28	38	50	82	123	176
	Q_m	3	5	8	12	17	23	31	50	76	109
0.020	Q_c	5	9	15	22	32	44	58	94	142	203
	Q_m	3	6	9	14	20	27	36	58	88	126
0.025	Q_c	6	10	17	25	36	49	65	106	159	228
	Q_m	3	6	10	16	22	30	40	65	98	141
0.030	Q_c	6	11	18	27	39	54	71	116	175	249
	Q_m	4	7	11	17	24	33	43	71	107	154
0.035	Q_c	7	12	20	30	42	58	77	125	189	269
	Q_m	4	8	12	18	26	36	47	77	116	166
0.040	Q_c	7	13	21	32	45	62	82	133	201	287
	Q_m	4	8	13	20	28	38	51	83	125	178
0.045	Q_c	8	14	22	34	48	66	87	142	213	306
	Q_m	5	8	14	21	30	40	53	87	132	189
0.050	Q_c	8	14	24	35	51	69	92	149	226	322
	Q_m	5	9	15	22	31	43	57	92	139	199

Q_c = Flow in Cubic Feet Per Second (CFS) Concrete Pipe (n=0.013)
Q_m = Flow in CFS in Corrugated Metal Pipe (n=0.021)
A = Area in Square Feet
Note: All quantities shown are based upon pipes flowing full - gravity flow

TABLE VI - FLOW IN PIPES (CFS)
Page 2

Slope		54" Pipe A=15.904	60" Pipe A=19.635	66" Pipe A=23.758	72" Pipe A=28.274	78" Pipe A=33.183	84" Pipe A=38.485	90" Pipe A=44.179	96" Pipe A=50.266	108" Pipe A=63.617	120" Pipe A=78.54
0.001	Q_c	62	82	106	134	166	202	243	288	395	523
	Q_m	39	51	66	83	103	125	150	178	244	324
0.003	Q_c	108	143	184	232	287	350	421	499	684	906
	Q_m	67	89	114	144	178	217	261	309	424	561
0.005	Q_c	139	183	237	300	372	452	543	644	883	1169
	Q_m	86	113	147	186	231	280	336	399	547	724
0.007	Q_c	164	217	281	355	440	535	643	763	1045	1382
	Q_m	102	134	174	220	273	331	398	473	648	856
0.009	Q_c	186	246	318	402	498	606	730	864	1183	1570
	Q_m	115	152	197	249	308	376	452	535	734	972
0.010	Q_c	197	260	336	423	524	639	768	912	1249	1654
	Q_m	122	161	208	262	325	395	476	565	773	1024
0.015	Q_c	242	319	411	518	641	782	940	1117	1530	2025
	Q_m	149	197	255	321	398	484	584	691	946	1256
0.020	Q_c	278	368	475	598	740	904	1088	1290	1765	2340
	Q_m	172	228	294	370	460	559	674	799	1093	1449
0.025	Q_c	311	411	531	668	827	1009	1212	1440	1975	2618
	Q_m	193	254	329	414	513	624	751	891	1220	1618
0.030	Q_c	341	451	582	733	907	1105	1330	1580	2162	2864
	Q_m	211	279	360	454	562	684	824	978	1339	1773
0.035	Q_c	368	486	628	792	980	1193	1435	1708	2335	3097
	Q_m	228	301	389	490	607	738	890	1053	1443	1917
0.040	Q_c	393	521	671	847	1048	1278	1536	1824	2497	3307
	Q_m	243	322	416	524	649	791	951	1129	1546	2047
0.045	Q_c	418	552	713	898	1111	1355	1630	1934	2650	3507
	Q_m	259	342	442	556	690	837	1010	1199	1640	2173
0.050	Q_c	440	581	750	945	1170	1428	1715	2040	2790	3700
	Q_m	273	360	465	585	725	884	1062	1260	1727	2289

Q_c = Flow in Cubic Feet Per Second (CFS) Concrete Pipe (n=0.013)
Q_m = Flow in CFS in Corrugated Metal Pipe (n=0.021)
A = Area in Square Feet
Note: All quantities shown are based upon pipes flowing full - gravity flow



TYPICAL CHANNEL SECTION

MASTER PLAN OF DRAINAGE
CITY OF ALBUQUERQUE - NEW MEXICO
AND ENVIRONS

CHANNEL DEMENSION SKETCH

GORDON HERKENHOFF & ASSOC
CONSULTING ENGINEERS
ALBUQUERQUE, NEW MEXICO

CHART
VII-A

TABLE VII

CARRYING CAPACITIES OF LINED AND UNLINED CHANNELS
In CFS

	25' Easement				30' Easement			
	Earth		Gunite or Asphalt Macadam		Earth		Gunite or Asphalt Macadam	
Slope	B=10' T=15' W=13' D=0.75' H=1.25' SS=2:1 CY=0.58	B=1.6' T=15' W=11.6' D=2.5' H=3.35' SS=2:1 CY=1.02	B=10' T=15' W=13' D=0.75' H=1.25' SS=2:1 CY=0.58 P=17.61	B=1.6' T=15' W=11.6' D=2.5' H=3.35' SS=2:1 CY=1.02 P=18.6	B=10' T=20' W=18' D=2.0' H=2.5' SS=2:1 CY=1.39	B=2.12' T=20' W=15.12' D=3.25' H=4.47' SS=2:1 CY=1.43	B=10' T=20' W=18' D=2.0' H=2.5' SS=2:1 CY=1.39 P=23.36	B=2.12' T=20' W=15.12' D=3.25' H=4.47' SS=2:1 CY=1.43 P=24.12
0.0025	16	49	22	67	90	98	123	134
0.0050	23	69	31	94	128	139	175	189
0.0075	28	84	38	115	157	170	214	232
0.0100	32	97	44	133	181	196	247	268
0.0125	36	109	49	149	202	220	277	300
0.0150	39	120	54	164	222	241	304	329
0.0175	42	128	58	176	238	259	326	353
0.0200	45	137	61	188	255	277	348	377
0.0225	48	146	65	199	271	294	370	401
0.0250	51	153	69	210	285	310	390	423
0.0275	53	161	72	221	300	326	410	444
0.0300	55	168	75	230	312	339	426	463
0.0325	58	175	78	239	325	353	444	482
0.0350	60	182	82	249	338	367	462	500
0.0375	62	189	85	257	350	381	479	519
0.0400	64	195	87	266	362	392	494	535
0.0425	66	200	90	274	372	404	508	551
0.0450	68	206	92	282	383	416	523	567
0.0475	70	212	95	290	394	428	538	583
0.0500	72	218	98	297	404	439	552	599

Coefficient of Roughness = n=0.030 (Earth), n=0.022 (Gunite)

Carrying Capacity given in cubic feet per second
CY = Cubic Yards of excavation per lineal foot of channel
P= Square Feet of Paving per lineal foot of channel with one foot of blanket on top of side slope.
Note: The first column under each the earth and gunite channel presents data for that most easily constructed with standard mass excavation equipment. The second column shows data for the most hydraulically efficient section. Portland cement concrete or asphaltic concrete smooth finishes will have twice the carrying capacities designated under earth channels.

Table VII - Carrying Capacities of Lined and Unlined Channels (CFS)
Page 2

	50' Easement				60' Easement			
	Earth		Gunite or Asphalt Macadam		Earth		Gunite or Asphalt Macadam	
Slope	B=10' T=30' W=25' D=2.5' H=3.33' SS=3:1 CY=2.47	B=1.52' T=30' W=24.02' D=3.75' H=4.74' SS=3:1 CY=2.77	B=10' T=30' W=25' D=2.5' H=3.33' SS=3:1 CY=2.47 P=33.0	B=1.52' T=30' W=24.02' D=3.75' H=4.74' SS=3:1 CY=2.77 P=33.52	B=10' T=40' W=31' D=3.5' H=5.0' SS=3:1 CY=3.80	B=2.08' T=40' W=32.08' D=5.0' H=6.32' SS=3:1 CY=4.94	B=10' T=40' W=31' D=3.5' H=5.0' SS=3:1 CY=3.80 P=43.6	B=2.08' T=40' W=32.08' D=5.0' H=6.32' SS=3:1 CY=4.94 P=44.08
0.0025	154	181	211	248	304	393	416	538
0.0050	218	256	298	350	430	556	588	760
0.0075	268	314	366	430	527	681	720	931
0.0100	309	363	422	496	609	786	830	1076
0.0125	346	406	472	554	681	880	931	1202
0.0150	380	446	518	610	746	966	1021	1321
0.0175	407	478	556	654	801	1038	1097	1418
0.0200	435	512	594	700	856	1109	1170	1517
0.0225	463	544	632	741	911	1179	1246	1610
0.0250	487	573	665	781	960	1240	1310	1692
0.0275	512	601	700	821	1010	1308	1380	1786
0.0300	534	627	728	856	1050	1360	1432	1858
0.0325	556	651	760	890	1092	1412	1492	1931
0.0350	577	678	788	925	1137	1470	1552	2009
0.0375	598	703	816	960	1179	1525	1611	2084
0.0400	617	725	844	990	1218	1572	1661	2150
0.0425	635	745	867	1019	1250	1619	1710	2210
0.0450	654	768	894	1049	1288	1664	1761	2278
0.0475	673	790	920	1079	1323	1712	1811	2342
0.0500	690	811	942	1109	1361	1761	1861	2409

Coefficient of Roughness =n=0.030 (Earth), n=0.022 (Gunite)
Carrying Capacity given in cubic feet per second
CY=Cubic Yards of excavation per lineal foot of channel
P=Square Feet of paving per lineal foot of channel with one foot of blanket on top of side slope.
Note: The first column under each the earth and gunite channel presents data for that most easily constructed with standard mass excavation equipment. The second column shows data for the most hydraulically efficient section. Portland cement concrete or asphaltic concrete smooth finishes will have twice the carrying capacities designated under earth channels.

Table VII - Carrying Capacities of Lined and Unlined Channels (CFS)
Page 3

	75' Easement				100' Easement			
	Earth		Gunite or Asphalt Macadam		Earth		Gunite or Asphalt Macadam	
Slope	B=20' T=45' W=41' D=3.5' H=4.17' SS=3:1 CY=4.72	B=2.40' T=45' W=38.40' D=6.0' H=7.1' SS=3:1 CY=6.25	B=20' T=45' W=41' D=3.5' H=4.17' SS=3:1 CY=4.72 P=48.4	B=2.40' T=45' W=38.40' D=6.0' H=7.1' SS=3:1 CY=6.25 P=49.4	B=20' T=70' W=60' D=5.0' H=6.25' SS=4:1 CY=9.25	B=2.0' T=70' W=58' D=7.0' H=8.50' SS=4:1 CY=11.35	B=20' T=70' W=60' D=5.0' H=6.25' SS=4:1 CY=9.25 P=73.6	B=2.0' T=70' W=58' D=7.0' H=8.50' SS=4:1 CY=11.35 P=74.0
0.0025	490	637	670	870	1097	1222	1500	1674
0.0050	694	900	949	1230	1550	1730	2120	2368
0.0075	850	1104	1162	1510	1900	2121	2600	2900
0.0100	983	1275	1343	1744	2195	2445	3000	3343
0.0125	1098	1427	1500	1950	2458	2740	3360	3745
0.0150	1208	1567	1650	2142	2700	3010	3695	4115
0.0175	1298	1681	1773	2300	2895	3230	3960	4415
0.0200	1381	1798	1890	2458	3095	3450	4225	4715
0.0225	1469	1911	2009	2614	3290	3670	4495	5019
0.0250	1550	2014	2120	2752	3465	3870	4738	5282
0.0275	1629	2118	2227	2892	3640	4065	4977	5560
0.0300	1698	2204	2320	3015	3795	4235	5182	5780
0.0325	1765	2296	2412	3140	3950	4405	5400	6020
0.0350	1832	2383	2505	3258	4083	4575	5567	6250
0.0375	1903	2473	2602	3380	4255	4750	5817	6495
0.0400	1966	2548	2688	3480	4385	4890	6000	6685
0.0425	2020	2624	2760	3585	4510	5045	6160	6900
0.0450	2078	2700	2841	3696	4645	5190	6350	7097
0.0475	2140	2770	2926	3783	4780	5335	6545	7295
0.0500	2200	2850	3008	3898	4915	5485	6715	7498

Coefficient of Roughness = n=0.030 (Earth), n=0.022 (Gunite)
Carrying Capacity given in cubic feet per second
CY=Cubic Yards of excavation per lineal foot of channel
P=Square Feet of paving per lineal foot of channel with one foot of blanket on top of side slope.
Note: The first column under each the earth and gunite channel presents data for that most easily constructed with standard mass excavation equipment. The second column shows data for the most hydraulically efficient section. Portland cement concrete or asphaltic concrete smooth finishes will have twice the carrying capacities designated under earth channels.

TABLE VIII

LOW SLOPE DITCHES & DRAINS

TOTAL CAPACITY (CFS)

Slope	Earth SS=1½:1 B=10 W=25 H=7 T=28 D=5 A=87.5	Gunite or AM SS=1½:1 B=10 W=25 H=7 T=28 D=5 A=87.5	Earth SS=2:1 B=10 W=30 H=7 T=34 D=5 A=100.0	Gunite or AM SS=2:1 B=10 W=30 H=7 T=34 D=5 A=100.0	Earth SS=1½:1 B=12 W=30 H=7 T=33 D=6 A=126.0	Gunite or AM SS=1½:1 B=12 W=30 H=7 T=33 D=6 A=126.0	Earth SS=2:1 B=12 W=36 H=7 T=40 D=6 A=144.0	Gunite or AM SS=2:1 B=12 W=36 H=7 T=40 D=6 A=144.0	Earth SS=1½:1 B=14 W=35 H=8 T=38 D=7 A=171.5	Gunite or AM SS=1½:1 B=14 W=35 H=8 T=38 D=7 A=171.5	Earth SS=2:1 B=14 W=42 H=8 T=46 D=7 A=196.0	Gunite or AM SS=2:1 B=14 W=42 H=8 T=46 D=7 A=196.0
0.0004	185	253	212	290	301	410	344	469	456	621	521	710
0.0005	207	283	234	324	332	460	379	525	502	696	574	795
0.0006	227	310	260	356	368	504	421	576	558	763	637	871
0.0007	245	334	281	384	398	544	455	621	604	823	690	940
0.0008	262	357	301	410	425	581	486	664	644	880	736	1004
0.0009	278	379	319	434	451	615	516	703	684	931	781	1063
0.0010	293	399	335	457	475	648	543	740	719	981	821	1120
0.0020	415	566	475	648	674	917	770	1049	1020	1390	1167	1590
0.0030	508	692	583	794	826	1125	944	1285	1250	1699	1429	1945
0.0040	585	798	674	919	944	1300	1090	1488	1445	1971	1650	2250
0.0050	655	893	750	1025	1063	1452	1215	1660	1610	2200	1841	2515

Earth: N=0.030
Gunite or Asphalt Macadam: N=0.022
A= Area (Square Feet)
AM= Asphalt Macadam
Note: Portland cement concrete or asphaltic concrete with smooth finishes will have twice the carrying capacities designated under earth channels (n=0.015)

TABLE VIII - LOW SLOPE DITCHES & DRAINS
TOTAL CAPACITY(CFS)

Page 2

Slope	Earth SS=1½:1 B=16 W=40 H=9 T=43 D=8 A=224.0	Gunite or AM SS=1½:1 B=16 W=40 H=9 T=43 D=8 A=224.0	Earth SS=2:1 B=16 W=48 H=9 T=52 D=8 A=256.0	Gunite or AM SS=2:1 B=16 W=48 H=9 T=52 D=8 A=256.0	Earth SS=1½:1 B=18 W=45 H=10 T=48 D=9 A=283.5	Gunite or AM SS=1½:1 B=18 W=45 H=10 T=48 D=9 A=283.5	Earth SS=2:1 B=18 W=54 H=10 T=58 D=9 A=324.0	Gunite or AM SS=2:1 B=18 W=54 H=10 T=58 D=9 A=324.0	Earth SS=1½:1 B=20 W=50 H=11 T=53 D=10 A=350	Gunite or AM SS=1½:1 B=20 W=50 H=11 T=53 D=10 A=350	Earth SS=2:1 B=20 W=60 H=11 T=64 D=10 A=400.0	Gunite or AM SS=2:1 B=20 W=60 H=11 T=64 D=10 A=400.0
0.0004	653	890	745	1018	885	1208	1011	1380	1178	1605	1343	1831
0.0005	718	996	821	1139	975	1351	1128	1545	1295	1797	1499	2050
0.0006	798	1081	903	1235	1082	1482	1239	1693	1440	1971	1642	2250
0.0007	864	1179	976	1347	1171	1600	1335	1828	1559	2125	1779	2429
0.0008	922	1248	1052	1440	1252	1709	1426	1953	1664	2270	1900	2592
0.0009	979	1333	1119	1525	1329	1812	1512	2070	1767	2408	2015	2748
0.0010	1030	1392	1164	1606	1397	1908	1597	2180	1859	2535	2120	2892
0.0020	1460	1990	1651	2258	1982	2700	2270	3080	2635	3595	3010	4100
0.0030	1767	2440	2019	2762	2430	3310	2766	3780	3230	4400	3685	5020
0.0040	2038	2787	2329	3185	2808	3830	3185	4380	3730	5085	4260	5810
0.0050	2280	3118	2605	3563	3135	4280	3563	4880	4160	5685	4750	6480

Earth: N=0.030
Gunite or Asphalt Macadam: N=0.022
A= Area (Square Feet)
AM=Asphalt Macadam
Note: Portland cement concrete or asphaltic concrete with smooth finishes will have twice the carrying capacities designated under earth channels (n=0.015)

TABLE VIII - LOW SLOPE DITCHES & DRAINS
TOTAL CAPACITY (CFS)

Slope (S)		D=1 A=35.5 W=37	D=2 A=74.0 W=40	D=3 A=115.5 W=43	D=4 A=160.0 W=46	D=5 A=207.5 W=49	D=6 A=258.0 W=52	D=7 A=311.5 W=55	D=8 A=368.0 W=58	D=9 A=427.5 W=61	D=10 A=490.0 W=64	D=11 A=555.5 W=67	D=12 A=624.0 W=70
	Q_E	34	108	215	352	517	709	929	1184	1469	1775	2109	2485
	Q_G	46	147	293	480	705	966	1266	1615	2000	2422	2875	3390
0.0004	Q_C	68	216	430	704	1034	1418	1858	2368	2938	3550	4218	4970
	Q_E	37	121	240	394	580	796	1044	1325	1636	1974	2354	2797
	Q_G	50	165	327	537	791	1063	1424	1807	2232	2693	3211	3815
0.0005	Q_C	74	242	480	788	1160	1592	2088	2650	3272	3948	4708	5594
	Q_E	41	133	263	431	634	869	1139	1448	1798	2172	2582	3040
	Q_G	56	182	359	588	864	1185	1553	1975	2450	2962	3525	4145
0.0006	Q_C	82	266	526	862	1268	1738	2278	2896	3596	4344	5164	6080
	Q_E	45	143	285	466	685	939	1230	1565	1943	2350	2790	3290
	Q_G	61	195	388	636	933	1280	1678	2134	2650	3205	3803	4485
0.0007	Q_C	90	286	570	932	1370	1878	2460	3130	3886	4700	5580	6580
	Q_E	48	153	304	498	731	1002	1312	1671	2075	2510	2980	3515
	Q_G	66	209	414	679	998	1369	1791	2280	2830	3423	4065	4790
0.0008	Q_C	96	306	608	996	1462	2004	2624	3342	4150	5020	5960	7030
	Q_E	51	163	322	528	776	1063	1394	1774	2203	2664	3165	3730
	Q_G	70	222	439	720	1059	1450	1902	2420	3005	3635	4320	5085
0.0009	Q_C	102	326	644	1056	1552	2126	2788	3548	4406	5328	6330	7460
	Q_E	53	171	339	556	816	1119	1468	1865	2319	2800	3330	3923
	Q_G	72	233	462	759	1112	1527	2000	2543	3162	3820	4540	5360
0.0010	Q_C	106	342	678	1112	1632	2238	2936	3730	4638	5600	6660	7846
	Q_E	76	243	481	787	1158	1538	2080	2645	3290	3975	4725	5570
	Q_G	104	331	656	1072	1579	2168	2838	3610	4490	5425	6450	7600
0.0020	Q_C	152	486	962	1574	2316	3176	4160	5290	6580	7950	9450	11140
	Q_E	93	297	588	964	1418	1943	2549	3240	4020	4863	5780	6820
	Q_G	127	405	801	1312	1933	2650	3477	4420	5485	6635	7890	9300
0.0030	Q_C	186	594	1176	1928	2836	3886	5098	6480	8040	9726	11560	13640
	Q_E	107	343	681	1114	1640	2248	2946	3745	4655	5625	6690	7880
	Q_G	146	468	929	1520	2235	3065	4018	5110	6350	7670	9120	10750
0.0040	Q_C	214	686	1362	2228	3280	4496	5892	7490	9310	11250	13380	15760
	Q_E	119	383	760	1244	1830	2510	3290	4180	5195	6280	7460	8800
	Q_G	162	522	1038	1697	2497	3425	4490	5710	7080	8570	10190	12000
0.0050	Q_C	238	766	1520	2488	3660	5020	6580	8360	10390	12560	14920	17600

Q_E=Capacity Earth (CFS)
Q_G=Capacity Gunite or Asphalt Macadam (CFS)
Q_C=Capacity Concrete or Asphaltic Concrete (CFS)

Earth: N=0.30
Gunite or Asphalt Macadam: N=0.022
Concrete or Asphaltic Concrete: N=0.015

All Sections:
B=34'; T=70'; H=12'; SS=1.5:1
Easement Width=100'
A=Cross Section Area in Sq. Ft.

TABLE IX

CHANNEL REQUIREMENTS FOR MAXIMUM PROBABLE FLOWS

Area (Acres)		S=0.001		S=0.002		S=0.003		S=0.004		S=0.005	
		Earth	Gunite	Earth	Gunite	Earth	Gunite	Earth	Gunite	Earth	Gunite
	D=	15.0	13.2	13.0	11.6	12.1	10.8	11.4	10.2	10.9	9.7
3,200	W=	105.0	92.4	91.0	81.2	84.7	75.6	79.8	71.4	76.3	67.9
	D=	15.8	14.1	14.0	12.4	12.9	11.5	12.2	10.8	11.65	10.4
3,840	W=	110.6	98.7	98.0	86.8	90.3	80.5	85.4	75.6	81.55	72.8
	D=	16.6	14.8	14.6	13.0	13.6	12.0	12.8	11.45	12.35	10.95
4,480	W=	116.2	103.6	102.2	91.0	95.2	84.0	89.6	80.15	86.45	76.65
	D=	17.4	15.4	15.3	13.6	14.0	12.5	13.4	11.8	12.8	11.4
5,120	W=	121.8	107.8	107.1	95.2	98.0	87.5	93.8	82.6	89.6	79.8
	D=	18.1	16.1	15.8	14.1	14.7	13.0	13.8	12.35	13.3	11.8
5,760	W=	126.7	112.7	110.6	98.7	102.9	91.0	96.6	86.45	93.1	82.6
	D=	18.6	16.6	16.4	14.5	15.2	13.5	14.4	12.8	13.7	12.3
6,400	W=	130.2	116.2	114.8	101.5	106.4	94.5	100.8	89.6	95.9	86.1
	D=	21.0	18.8	18.4	16.5	17.0	15.3	16.1	14.0	15.5	13.85
9,600	W=	147.0	131.6	128.8	115.5	119.0	107.1	112.7	98.0	108.5	96.95
	D=	22.8	20.5	20.0	17.6	18.4	16.4	17.5	15.6	16.5	14.95
12,800	W=	159.6	143.5	140.0	123.2	128.8	114.8	122.5	109.2	115.5	104.65
	D=	24.3	21.5	21.3	18.7	19.5	17.5	18.6	16.5	17.6	15.7
16,000	W=	170.1	150.5	149.1	130.9	136.5	122.5	130.2	115.5	123.2	109.9
	D=	25.3	22.4	22.2	19.6	20.5	18.1	19.3	17.2	18.6	16.5
19,200	W=	177.1	156.8	155.4	137.2	143.5	126.7	135.1	120.4	130.2	115.5
	D=	26.0	23.1	22.8	20.4	21.2	18.7	20.0	17.6	19.2	17.1
22,400	W=	182.0	161.7	159.6	142.8	148.4	130.9	140.0	123.2	134.4	119.7
	D=	26.7	24.0	23.5	21.0	21.7	19.4	20.5	18.2	19.7	17.6
25,600	W=	186.9	168.0	164.5	147.0	151.9	135.8	143.5	127.4	137.9	123.2
	D=	27.4	24.5	24.0	21.5	22.3	19.8	21.0	18.7	20.1	18.0
28,800	W=	191.8	171.5	168.0	150.5	156.1	138.6	147.0	130.9	140.7	126.0
	D=	28.0	25.0	24.5	21.8	22.7	20.0	21.4	19.0	20.5	18.25
32,000	W=	196.0	175.0	171.5	152.6	158.9	140.0	149.8	133.0	143.5	127.75

EARTH: SS=2:1; N=0.030; B=3D

GUNITE: SS=2:1; N=0.022; B=3D

TABLE IX - CHANNEL REQUIREMENTS FOR MAXIMUM PROBABLE FLOWS
Page 2

Area (Acres)		S=0.006		S=0.007		S=0.008		S=0.009		S=0.010	
		Earth	Gunite	Earth	Gunite	Earth	Gunite	Earth	Gunite	Earth	Gunite
3,200	D=	10.6	9.45	10.35	9.2	10.15	9.0	9.85	8.8	9.7	8.65
	W=	74.2	66.15	72.45	64.4	71.05	63.0	68.95	61.6	67.9	60.55
3,840	D=	11.3	10.0	11.0	9.75	10.8	9.6	10.55	9.35	10.35	9.2
	W=	79.1	70.0	77.0	68.25	75.6	67.2	73.85	65.45	72.45	64.4
4,480	D=	11.85	10.55	11.6	10.3	11.3	10.1	11.05	9.8	10.85	9.6
	W=	82.95	73.85	81.2	72.1	79.1	70.7	77.35	68.6	75.95	67.2
5,120	D=	12.4	11.0	12.1	10.8	11.8	10.5	11.55	10.2	11.25	10.05
	W=	86.8	77.0	84.7	75.6	82.6	73.5	80.85	71.4	78.75	70.35
5,760	D=	13.0	11.5	12.55	11.2	12.25	10.95	12.0	10.65	11.8	10.5
	W=	91.0	80.5	87.85	78.4	85.75	76.65	84.0	74.55	82.6	73.5
6,400	D=	13.3	11.9	13.0	11.55	12.75	11.25	12.45	11.0	12.2	10.9
	W=	93.1	83.3	91.0	80.85	89.25	78.75	87.15	77.0	85.4	76.3
9,600	D=	15.0	13.3	14.6	13.0	14.25	12.75	14.05	12.45	13.7	12.2
	W=	105.0	93.1	102.2	91.0	99.75	89.25	98.35	87.15	95.9	85.4
12,800	D=	16.2	14.5	15.8	14.1	15.5	13.75	15.1	13.6	14.95	13.25
	W=	113.4	101.5	110.6	98.7	108.5	96.25	105.7	95.2	104.7	92.75
16,000	D=	17.3	15.45	16.95	15.0	16.5	14.6	16.2	14.2	15.8	14.1
	W=	121.1	108.15	118.7	105.0	115.5	102.2	113.4	99.4	110.6	98.7
19,200	D=	18.0	16.1	17.6	15.7	17.2	15.4	16.9	15.0	16.55	14.6
	W=	126.0	112.7	123.2	109.9	120.4	107.8	118.3	105.0	115.85	102.2
22,400	D=	18.6	16.4	18.2	16.2	17.8	15.8	17.4	15.5	17.0	15.1
	W=	130.2	114.8	127.4	113.4	124.6	110.6	121.8	108.5	119.0	105.7
25,600	D=	18.9	16.9	18.5	16.6	18.2	16.2	17.7	15.8	17.4	15.5
	W=	132.3	118.3	129.5	116.2	127.4	113.4	123.9	110.6	121.8	108.5
28,800	D=	19.5	17.5	19.0	17.0	18.5	16.6	18.3	16.2	17.75	15.9
	W=	136.5	122.5	133.0	119.0	129.5	116.2	128.1	113.4	124.3	111.3
32,000	D=	19.9	17.7	19.4	17.2	19.0	17.0	18.5	16.5	18.2	16.2
	W=	139.3	123.9	135.8	120.4	133.0	119.0	129.5	115.5	127.4	113.4

EARTH: SS=2:1; N=0.030; B=3D

GUNITE: SS=2:1; N=0.022; B=3D

TABLE IX - CHANNEL REQUIREMENTS FOR MAXIMUM PROBABLE FLOWS
Page 3

Area (Acres)		S=0.001 Earth	S=0.001 Gunite	S=0.002 Earth	S=0.002 Gunite	S=0.003 Earth	S=0.003 Gunite	S=0.004 Earth	S=0.004 Gunite	S=0.005 Earth	S=0.005 Gunite
3,200	D=	12.0	10.7	10.6	9.35	9.75	8.75	9.3	8.25	8.9	7.95
	W=	120.0	107.0	106.0	93.5	97.5	87.5	93.0	82.5	89.0	79.5
3,840	D=	12.9	11.5	11.2	10.1	10.45	9.3	9.9	8.8	9.5	8.4
	W=	129.0	115.0	112.0	101.0	104.5	93.0	99.0	88.0	95.0	84.0
4,480	D=	13.6	12.0	11.9	10.6	11.0	9.75	10.4	9.25	9.95	8.85
	W=	136.0	120.0	119.0	106.0	110.0	97.5	104.0	92.5	99.5	88.5
5,120	D=	14.1	12.5	12.3	10.95	11.45	10.2	10.8	9.6	10.4	9.25
	W=	141.0	125.0	123.0	109.5	114.5	102.0	108.0	96.0	104.0	92.5
5,760	D=	14.6	13.0	12.9	11.45	11.9	10.6	11.2	10.05	10.8	9.65
	W=	146.0	130.0	129.0	114.5	119.0	106.0	112.0	100.5	108.0	96.5
6,400	D=	15.2	13.5	13.2	11.8	12.35	11.0	11.6	10.4	11.15	10.0
	W=	152.0	135.0	132.0	118.0	123.5	110.0	116.0	104.0	111.5	100.0
9,600	D=	17.1	15.2	15.1	13.3	13.9	12.4	13.2	11.7	12.6	11.2
	W=	171.0	152.0	151.0	133.0	139.0	124.0	132.0	117.0	126.0	112.0
12,800	D=	18.3	16.5	16.2	14.5	15.0	13.4	14.2	12.6	13.6	12.15
	W=	183.0	165.0	162.0	145.0	150.0	134.0	142.0	126.0	136.0	121.5
16,000	D=	19.5	17.6	17.1	15.3	15.9	14.1	15.0	13.4	14.5	12.85
	W=	195.0	176.0	171.0	153.0	159.0	141.0	150.0	134.0	145.0	128.5
19,200	D=	20.5	18.2	17.9	16.0	16.5	14.8	15.7	14.0	15.0	13.5
	W=	205.0	182.0	179.0	160.0	165.0	148.0	157.0	140.0	150.0	135.0
22,400	D=	21.2	18.8	18.5	16.5	17.2	15.4	16.2	14.4	15.6	13.9
	W=	212.0	188.0	185.0	165.0	172.0	154.0	162.0	144.0	156.0	139.0
25,600	D=	21.7	19.3	18.9	16.9	17.5	15.6	16.7	14.8	16.0	14.2
	W=	217.0	193.0	189.0	169.0	175.0	156.0	167.0	148.0	160.0	142.0
28,800	D=	22.4	19.7	19.4	17.2	18.0	16.0	17.0	15.2	16.4	14.5
	W=	224.0	197.0	194.0	172.0	180.0	160.0	170.0	152.0	164.0	145.0
32,000	D=	22.7	20.0	20.0	17.6	18.3	16.4	17.5	15.4	16.7	14.8
	W=	227.0	200.0	200.0	176.0	183.0	164.0	175.0	154.0	167.0	148.0

EARTH (E): SS=2:1; N=0.030; B=6D

GUNITE (G): SS=2:1; N=0.022; B=6D

TABLE IX - CHANNEL REQUIREMENTS FOR MAXIMUM PROBABLE FLOWS
Page 4

Area (Acres)		S=0.006		S=0.007		S=0.008		S=0.009		S=0.010	
		Earth	Gunite	Earth	Gunite	Earth	Gunite	Earth	Gunite	Earth	Gunite
3,200	D=	8.6	7.7	8.4	7.45	8.2	7.25	7.95	7.1	7.85	6.95
	W=	86.0	77.0	84.0	74.5	82.0	72.5	79.5	71.0	78.5	69.5
3,840	D=	9.2	8.2	8.95	7.9	8.7	7.7	8.55	7.6	8.4	7.4
	W=	92.0	82.0	89.5	79.0	87.0	77.0	85.5	76.0	84.0	74.0
4,480	D=	9.65	8.6	9.4	8.4	9.2	8.2	8.95	8.0	8.8	7.8
	W=	96.5	86.0	94.0	84.0	92.0	82.0	89.5	80.0	88.0	78.0
5,120	D=	10.1	8.95	9.75	8.65	9.5	8.5	9.3	8.3	9.2	8.1
	W=	101.0	89.5	97.5	86.5	95.0	85.0	93.0	83.0	92.0	81.0
5,760	D=	10.5	9.35	10.2	9.1	9.95	8.85	9.75	8.65	9.5	8.5
	W=	105.0	93.5	102.0	91.0	99.5	88.5	97.5	86.5	95.0	85.0
6,400	D=	10.85	9.6	10.55	9.4	10.25	9.2	10.1	8.95	9.85	8.8
	W=	108.5	96.0	105.5	94.0	102.5	92.0	101.0	89.5	98.5	88.0
9,600	D=	12.2	10.85	11.85	10.55	11.6	10.35	11.35	10.15	11.1	9.9
	W=	122.0	108.5	118.5	105.5	116.0	103.5	113.5	101.5	111.0	99.0
12,800	D=	13.2	11.8	12.85	11.5	12.45	11.2	12.25	10.95	12.05	10.75
	W=	132.0	118.0	128.5	115.0	124.5	112.0	122.5	109.5	120.5	107.5
16,000	D=	14.0	12.4	13.6	12.15	13.3	11.85	13.0	11.6	12.8	11.4
	W=	140.0	124.0	136.0	121.5	133.0	118.5	130.0	116.0	128.0	114.0
19,200	D=	14.5	13.0	14.2	12.65	13.8	12.4	13.6	12.15	13.3	11.85
	W=	145.0	130.0	142.0	126.5	138.0	124.0	136.0	121.5	133.0	118.5
22,400	D=	15.0	13.45	14.6	13.05	14.35	12.8	14.0	12.5	13.65	12.25
	W=	150.0	134.5	146.0	130.5	143.5	128.0	140.0	125.0	136.5	122.5
25,600	D=	15.45	13.75	15.0	13.40	14.6	13.05	14.35	12.8	14.15	12.45
	W=	154.5	137.5	150.0	134.0	146.0	130.5	143.5	128.0	141.5	124.5
28,800	D=	15.7	14.1	15.45	13.7	15.0	13.35	14.65	13.1	14.4	12.8
	W=	157.0	141.0	154.5	137.0	150.0	133.5	146.5	131.0	144.0	128.0
32,000	D=	16.1	14.35	15.7	14.05	15.3	13.6	15.0	13.35	14.6	13.05
	W=	161.0	143.5	157.0	140.5	153.0	136.0	150.0	133.5	146.0	130.5

EARTH (E): SS=2:1; N=0.030; B=6D

GUNITE (G): SS=2:1; N=0.022; B=6D

TABLE X

REQUIRED HEIGHT OF FINISHED FLOOR ABOVE TOP OF CURB

Slope	Required Height FF above TC
0.0025	24"
0.0050	20"
0.0075	18"
0.0100	16"
0.0125	14"
0.0150	14"
0.0175	14"

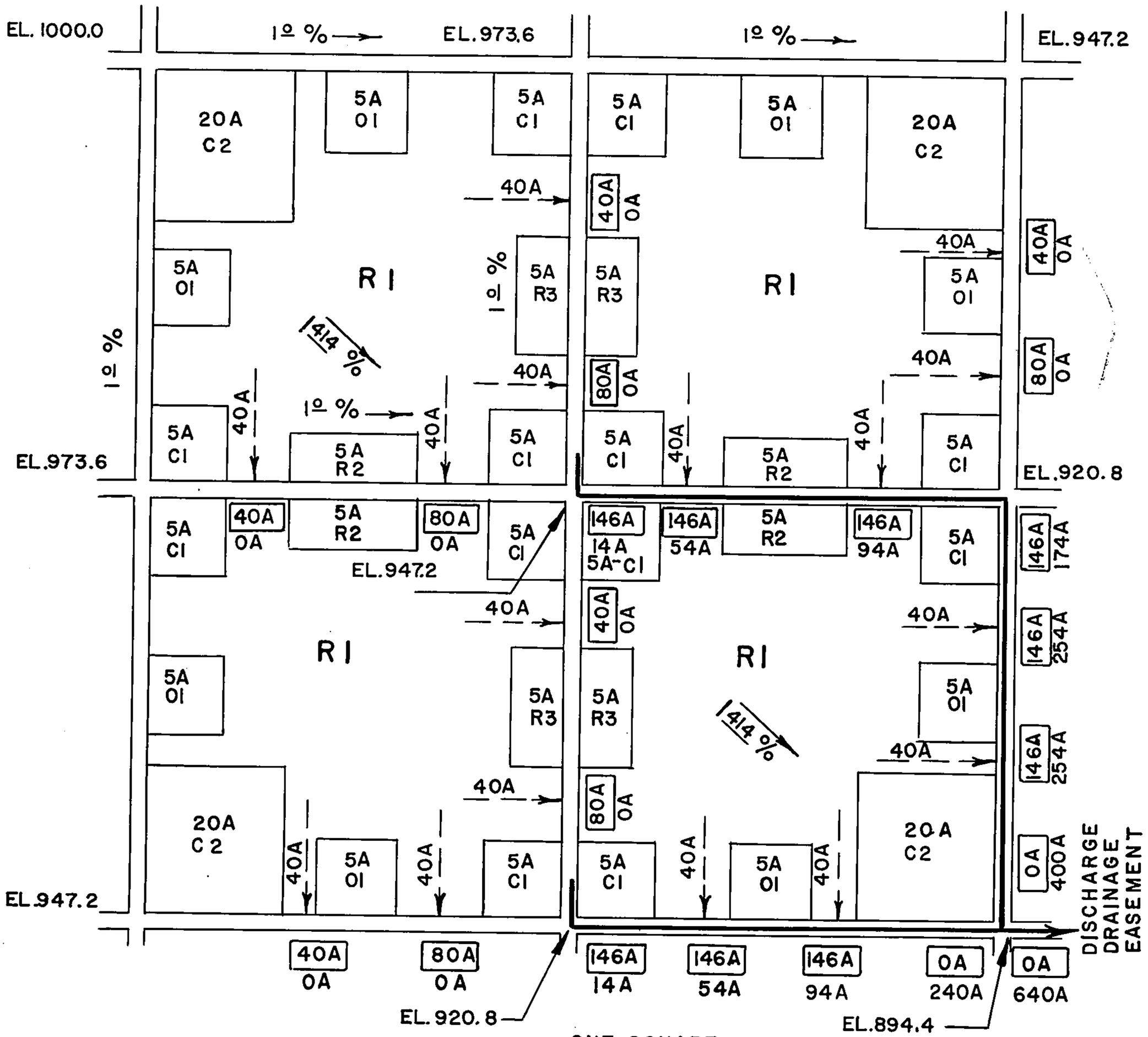
Notes:

1. 14" is suggested as a minimum requirement for all slopes.

2. The computation is based upon a 32' street face of curb to face of curb (n=0.015). The street has a 6" crown at the center line. There is a 4' sidewalk, including the top of curb, on each side (n=0.015). There is a 16' grassed area on each side (n=0.030). The high water line is 8" below the finished floor in a 100 year frequency storm accumulated from a 40 acre area.

3. The requirements are based upon linear flow. Requirements due to energy and momentum phenomena must be added.

DISTRIBUTION OF FLOWS BETWEEN STREETS AND STORM SEWERS



ONE SQUARE MILE
SCALE 1"= 1000'

LEGEND

- 40A → COLLECTOR OR RESIDENTIAL STREET WITH FLOW (40A)
- ARTERIAL STREET
- STORM SEWER
- 80A — STREET FLOW
- 40A — STORM SEWER FLOW
- 5A / C1 — OUTLINE, ACREAGE, & ZONING OF TRACT
- ARBITRARY ELEVATION

1. SEE CHART II A FOR COMPUTATION OR RUNOFF COEFFICIENT
2. SEE CHART II B FOR STORM SEWER LAYOUT AND ESTIMATE

MASTER PLAN OF DRAINAGE
CITY OF ALBUQUERQUE - NEW MEXICO
AND ENVIRONS

ONE SQUARE MILE AREA
DEVELOPED TO PRODUCE
MAXIMUM RUNOFF

GORDON HERKENHOFF & ASSOC
CONSULTING ENGINEERS
ALBUQUERQUE, NEW MEXICO

CHART XI

CHART XI-A

COMPUTATION OF RUNOFF COEFFICIENT

The following developments were chosen as typical:

1. C2 -- Princess Jeanne and Fair Plaza Shopping Center
2. C1 -- Northwest Corner Menaul and Eubank Boulevards NE
   Bel Air Shopping Center on Menaul Boulevard NE
   Shopping Center on San Pedro SE
3. R2 -- Three Developments Between 11th & 17th from Lomas to Coal
4. R3 -- Snow Apartments on Northeastern Blvd., NE
   McMullen Apartments on Cardenas Street SE
5. O1 -- San Pedro Blvd., NE immediately North of Copper
   Sierra at Silver SE
6. R1 -- Hoffmantown from Sandia Conservancy District Master Plan.

C2 & C1:
140A
21.9% Total

Type of Surface	Percent of Total Surface	Standard Run-off Coefficient	Weighted Run-off Coefficient
Roof	28%	0.825	0.231
Paving	64%	0.875	0.560
Unclassified	8%	0.250	0.020
Total	100%	--	0.811

R2,R3,O1:
80A
12.5% Total

Type of Surface	Percent of Total Surface	Standard Run-off Coefficient	Weighted Run-off Coefficient
Roof	21%	0.825	0.173
Paving	42%	0.875	0.368
Unwalled Lawns	11%	0.150	0.017
Unclassified	26%	0.250	0.065
Total	100%	--	0.623

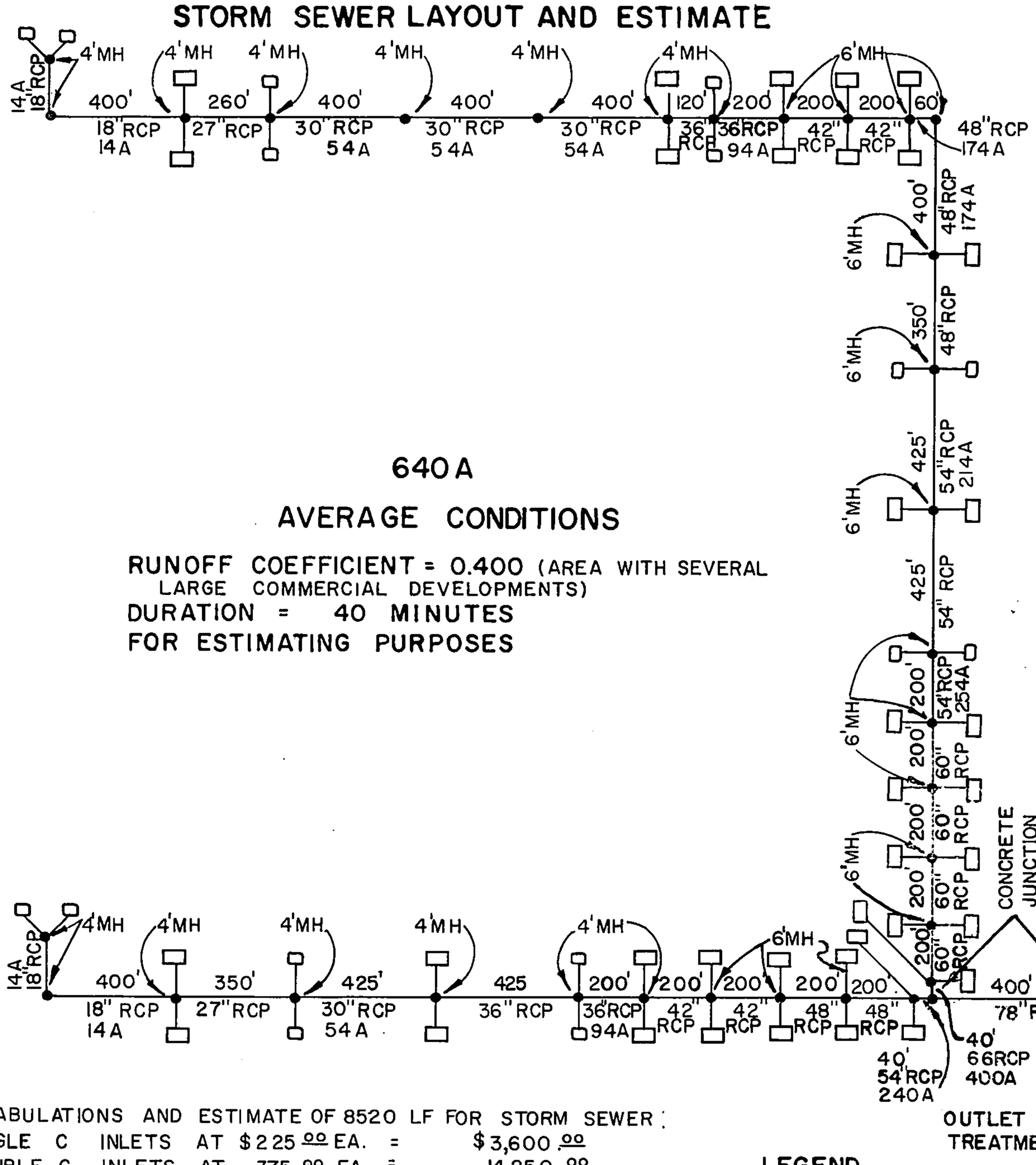
R1: Sandia Conservancy District Master Plan = 0.283
420A
65.6% Total

Computation of Weighted Runoff Coefficient for 640 Acres (One Square Mile) with development for maximum runoff conditions:

C1 & C2: 21.9% x 0.811 = 0.178
R2,R3, & O1: 12.5% x 0.623 = 0.078
R1: 65.6% x 0.283 = 0.186
Total Weighted Coefficient = 0.442

Weighted runoff coefficient = 0.45 for maximum runoff conditions.

STORM SEWER LAYOUT AND ESTIMATE



640 A

AVERAGE CONDITIONS

RUNOFF COEFFICIENT = 0.400 (AREA WITH SEVERAL LARGE COMMERCIAL DEVELOPMENTS)
DURATION = 40 MINUTES
FOR ESTIMATING PURPOSES

TABULATIONS AND ESTIMATE OF 8520 LF FOR STORM SEWER:

16 SINGLE C INLETS	AT $225.00	EA.	=	$3,600.00
38 DOUBLE C INLETS	AT 375.00	EA	=	14,250.00
14-4' DIA. MANHOLES	AT 250.00	EA	=	3,500.00
18-6' DIA. MANHOLES	AT 300.00	EA	=	5,400.00
1,400' 15" RCP AT INLETS	AT 5.00	/LF	=	7,000.00
1,800' 18" RCP AT INLETS	AT 6.00	/LF	=	10,500.00
2 CONC. JUNCTION BOX COMPLETE	AT 2,000.00	EA		4,000.00
1,000' 18" RCP INCL TRENCH & BKFL	AT 6.00	/LF	=	6,000.00
610' 27" RCP	AT 11.00	/LF	=	6,710.00
1,625' 30" RCP	AT 12.00	/LF	=	19,500.00
945' 36" RCP	AT 14.00	/LF	=	13,230.00
800' 42" RCP	AT 15.00	/LF	=	12,000.00
1210' 48" RCP	AT 17.00	/LF	=	20,570.00
1090' 54" RCP	AT 20.00	/LF	=	21,800.00
800' 60" RCP	AT 22.00	/LF	=	17,600.00
40' 66" RCP	AT 28.00	/LF	=	1,120.00
400' 78" RCP	AT 33.00	/LF	=	13,200.00
1 OUTLET TREATMENT	AT 7,500	EA	=	7,500.00
		TOTAL	=	$187,780.00
			=	$22.04/LF

105.690

LEGEND

- DOUBLE "C" INLET
- SINGLE "C" INLET

MASTER PLAN OF DRAINAGE
CITY OF ALBUQUERQUE – NEW MEXICO
AND ENVIRONS

ESTIMATE FOR DRAINAGE FACILITIES REQUIRED IN THE DEVELOPMENT SHOWN ON CHART XI

GORDON HERKENHOFF & ASSOC
CONSULTING ENGINEERS
ALBUQUERQUE, NEW MEXICO

CHART XI B

TABLE XII

SUGGESTED CRITERIA FOR ANALYZING STORM RUNOFF

AND

CARRYING CAPACITIES OF FACILITIES

Hydrology:

Runoff computation = Rational Formula : $Q = AIR$

Q= Runoff Quantity in Acre Inches per Hour or Cubic Feet per Second
A= Contributing Area in Acres
I= Intensity in Inches per Hour for a duration equal to the time of accumulation (duration) in minutes from Chart 1, Appendix A.
R= Runoff Coefficient *(No Unit)

*Suggested Runoff Coefficient for Estimates =

0.30 for undeveloped and purely residential areas
0.35 for average metropolitan areas
0.40 for areas with several large commercial developments
0.45 for maximum runoff conditions in suburban areas
0.80 for shopping center or complete commercial development

*Standard Coefficients for Computing Weighted Runoff Coefficients =

Kuichling's Standard Coefficients

Carrying Capacities of Facilities:

Carrying Capacities: $Q = AV$
: Manning's Formula: $V = \frac{1.486}{n} R^{2/3} S^{\frac{1}{2}}$

A= Cross-section area of conductor in square feet
V= Velocity in feet per second
n= Roughness coefficient* of conductor surface
R= Hydraulic Radius of Conductor = Cross-section area/wetted perimeter
S= Lowest gradient for any considerable reach of the carrier.
Q= Quantity of flow in cubic feet per second.

*Suggested Roughness Coefficients:

Concrete Pipe = 0.013
Corrugated Metal Pipe or Arches = 0.021
Concrete: Gunite = 0.022
Asphaltic (Street finish course) = 0.015
Smooth Float (Sidewalk & Curb & Gutter) = 0.015
Asphalt Macadam = 0.022
Earth = 0.030

NE-1

NE-2

EC-1

SE-1

NE-3

MILITARY RESERVATION

NE-4

EC-2

NE-5

NE-6

SE-2

EC-3

NE-7

SE-3

SW-1

SW-2

SW-4

SW-3

NW-1

NW-2

NW-3

MASTER PLAN OF DRAINAGE
CITY OF ALBUQUERQUE – NEW MEXICO AND ENVIRONS

MAP DELINEATING AREAS COVERED BY ESTIMATES
ALBUQUERQUE AREA

GORDON HERKENHOFF & ASSOC.
CONSULTING ENGINEERS
ALBUQUERQUE, NEW MEXICO

CHART XIII

APPENDIX A

METEOROLOGICAL PHENOMENA

RELATING TO THE CREATION OF RAINFALL

Rainfall in the Albuquerque area can result from meteorological phenomena as follows: (1) convective lifting, (2) orographic lifting, (3) convective-orographic lifting, or (4) convergence.

Convective lifting is the rising of heated surface air, and it is the most common of occurrences for this region. Water vapor within the rising air or in the path of its ascent condenses at higher altitudes, and the basis for a thundershower is formed. The resulting storm is generally one of high intensity and relatively short duration. Although the intensity of convective storms is always high, duration will vary with the amount of moisture present. Blair* has stated that completely saturated surface air where convective lifting occurs will not likely produce more than one inch of rainfall, unless some replacement of moist air takes place. Moisture content in surface air is generally small during hot summer months when convective lifting is prevalent, and this leads to the conclusion that this phenomenon alone cannot produce extremely severe storms.

Orographic lifting is the act of air masses being lifted by topographic barriers such as mountains. The proximity of terrain that lends itself to this occurrence establishes that storms resulting from this activity are very probable. Precipitation resulting from orographic phenomenon alone is not usually intense, but these storms may have an extended duration. This classification of storm generally does not result in severe flooding, unless the

*BLAIR, THOMAS A. - "Weather Elements": A text book in elementary meteorology; New York, Prentice-Hall, Inc; 1942 Revised Edition.

combined intensity and duration are such that large ponds form in lowland areas.

Convective-orographic lifting undoubtedly produces the most severe storms common to this region. The orographic phase provides the required replacement of moist air which prolongs precipitation resulting from convective activity. The probability of high intensity rainfall resulting from convection along with possibilities of increased duration from orographic action gives this category of storm the priority for consideration.

Convergence occurs when air masses of different temperatures meet, and the warmer air ascends. The amount of precipitation will depend directly upon the quantity of moisture in the rising air mass. This type of storm can occur here, but it is believed that due to physical characteristics common the region, other meteorological occurrences are more prevalent. This phenomenon is capable of producing severe storms in some areas of the country, but the low moisture content common to warm air masses in this area reduces its potential.

Weather records establish that a direct relationship between high temperatures and precipitation exists in this area. A 30 year average maximum daily mean temperature of over 80° F. has been recorded for July through September, and the average precipitation for each month is over one inch. These records serve to verify the importance of convective activity in producing precipitation.

APPENDIX A

ENERGY AND MOMENTUM OF GRAVITY FLOWS

Consideration of the energy and momentum of flows is required at confluence and abrupt changes in alignment or gradient of all carrier facilities in order to prevent malfunctions at these critical points.

Energy resulting from gravity flows is a function of the slope of the carrier and the depth and velocity of the flow, and the momentum applicable to flows is a function of the quantity of flow, its velocity, and friction losses attributable to the surface characteristics of the carrier. Each of these factors can be controlled to some extent, but velocity and depths are most easily regulated. Velocity in open channels, as expressed in Manning's Formula ($V = \frac{1.486}{n} R^{2/3} S^{\frac{1}{2}}$), is derived from considerations of surface characteristics, shape, and slope of the carrier. In carriers such as storm sewers and streets, slope will provide the best method for control, since surface characteristics and shape are fixed within narrow limitations. In drainage channels, the shape will be most easily altered, and some controls with regard to surface characteristics are possible.

Energy and momentum with regard to gravity flows are most commonly described by referring to the state of flow expressed as subcritical, critical, or supercritical. Critical flow is that state in which specific energy and specific force are a minimum for a given discharge. At critical flow in a prismatic channel, the velocity head is equal to one-half the hydraulic depth (the cross-section area divided by the width of the channel at the surface of the water). Critical flow is likely to become unstable, since minor changes

in channel characteristics may create major changes in depth. Subcritical flow lends to treatment by classical hydraulic methods, since disturbances are always propogated upstream. Supercritical flow is completely unpredictable, because distrubances occur downstream. The critical state of flow should be avoided in design when there is any possibility of a change in flow characteristics. Flows in the subcritical state require the least attention in design work, and supercritical flow can be handled as long as no major changes or obstructions occur within the channel.

Although energy and momentum can be computed for supercritical flows, modeling has proved to be the most effective means of determining methods of design for facilities involving this state of flow. Phenomena resulting from convergence of large but unequal flows as well as in dissipating energy in large flows in the supercritical state are somewhat incomprehensible from computations, thus modeling will provide the best results.

APPENDIX A

TECHNICAL DEVELOPMENT OF INTENSITY-DURATION-FREQUENCY RELATIONSHIPS

1. 24 Hour Rainfall Records from the U.S. Weather Bureau Station at the Albuquerque Airport (1891-1962=71 yrs) and Station No. 2 (1935-1953= 19 Years) were tabulated in .2 inch intervals from 0.5 inches to 2.3 inches. This is a total of 90 years of record, and the rate of occurrence for each increment of rainfall was computed.

2. A plot of 24 hour rainfall in inches against occurrence in years approximates a straight line on logarithmic 2 (Log-Log) paper.

3. From empirical data, it was found that as a general rule 90% of the rain will fall in a two hour period; thus the intensity in inches per hour for any given frequency is calculated as follows:

$$I = Ro \times 0.9/2 = 0.45\ Ro$$ where

I is the intensity in inches per hour for a frequency of occurrence of 0 years.

Ro is the 24 hour rainfall in inches for a frequency of occurrence of 0 years.

4. Yarnell's Standard Equation $I = \frac{A}{t+B}$ was used. The Intensity (I) in inches per hour was derived from the equation derived in paragraph 3 above for the 10, 25, 50, and 100 year frequency of occurrence.

t was taken as 120 minutes

B was taken as constant from Yarnell's Standard Equations for the Southwest Area, and then was corrected to permit A to be established as the nearest whole number.

A was computed, and corrected to the nearest whole number.

Data is as follows:

90 years of record for plotting curve Paragraph 2:

24 Hour Rainfall Inches	Total No. Storms	Rate of Occurrence of storm in years
0.5-0.7	176	0.51
0.7-0.9	82	1.1
0.9-1.1	28	3.2
1.1-1.3	15	6.0
1.3-1.5	5	18.0
1.5-1.7	7	12.8
1.7-1.9	4	22.5
1.9-2.1	3	30.0
2.1-2.3	1	90.0

Data (Continued)

Intensities in Inches Per Hour From Equation Paragraph 3:

I(10 Yrs) = (0.45)(1.42) = 0.64 in./Hr.
I(25 Yrs) = (0.45)(1.87) = 0.84 in./Hr.
I(50 Yrs) = (0.45)(2.30) = 1.04 in./Hr.
I(100 Yrs)= (0.45)(2.88) = 1.30 in./Hr.

Yarnell Standard Equations:

I(10 Yrs) = 122/t+23
I(25 Yrs) = 155/t+26
I(50 Yrs) = 160/t+21
I(100 Yrs)= 210/t+26

Computation of Equations for this Area:

10 Year: 0.64 = A/120+23; A=91.49; Corrected A=91
0.64 = 91/120+B; B=24
I(10 Yrs) = 91/t+24 (Check = 0.632 for 120 min)

25 Year: 0.84 = A/120+26; A=122.9; Corrected A=123
0.84 = 123/120+B; B=27
I(25 Yr) = 123/t+27 (Check = 0.837 for 120 min)

50 Year: 1.04 = A/120+21; A = 146.4; Corrected A = 146
1.04 = 146/120+B; B=20
I(50 Yr) = 146/t+20 (Check = 1.041 for 120 min)

100 Year: 1.30 = A/120+26; A = 188.8; Corrected A=189
1.30 = 189/120+B; B=25
I(100 Yr) = 189/t+25 (Check = 1.304 for 120 min.)

PLATE 1

PLATE 2

PLATE 3

PLATE 4

PLATE 5

PLATE 6

PLATE 10

PLATE 9

PLATE 8

PLATE 7

PLATE 11

PLATE 12

PLATE 13

PLATE 14

MASTER PLAN OF DRAINAGE CITY OF ALBUQUERQUE – NEW MEXICO AND ENVIRONS

PLATE LOCATION MAP

PLATE INDEX

GORDON HERKENHOFF & ASSOC.
CONSULTING ENGINEERS
ALBUQUERQUE, NEW MEXICO

MATCH LINE PLATE 2

MATCH LINE PLATE 3

TOWN OF ALAMEDA GRANT

LEGEND

- LIMIT OF DRAINAGE AREA
- LIMIT OF SUB-DRAINAGE AREA
- DRAINAGE EASEMENT
- NATURAL DRAINAGE AREA — 2537A
- DRAINAGE AREA AFTER DEVELOPMENT — 2537A
- WEATHER STATION (CLIMATIC CONDITIONS) — △
- RUNOFF MEASURING STATIONS — ○

SCALE 1"-2000'

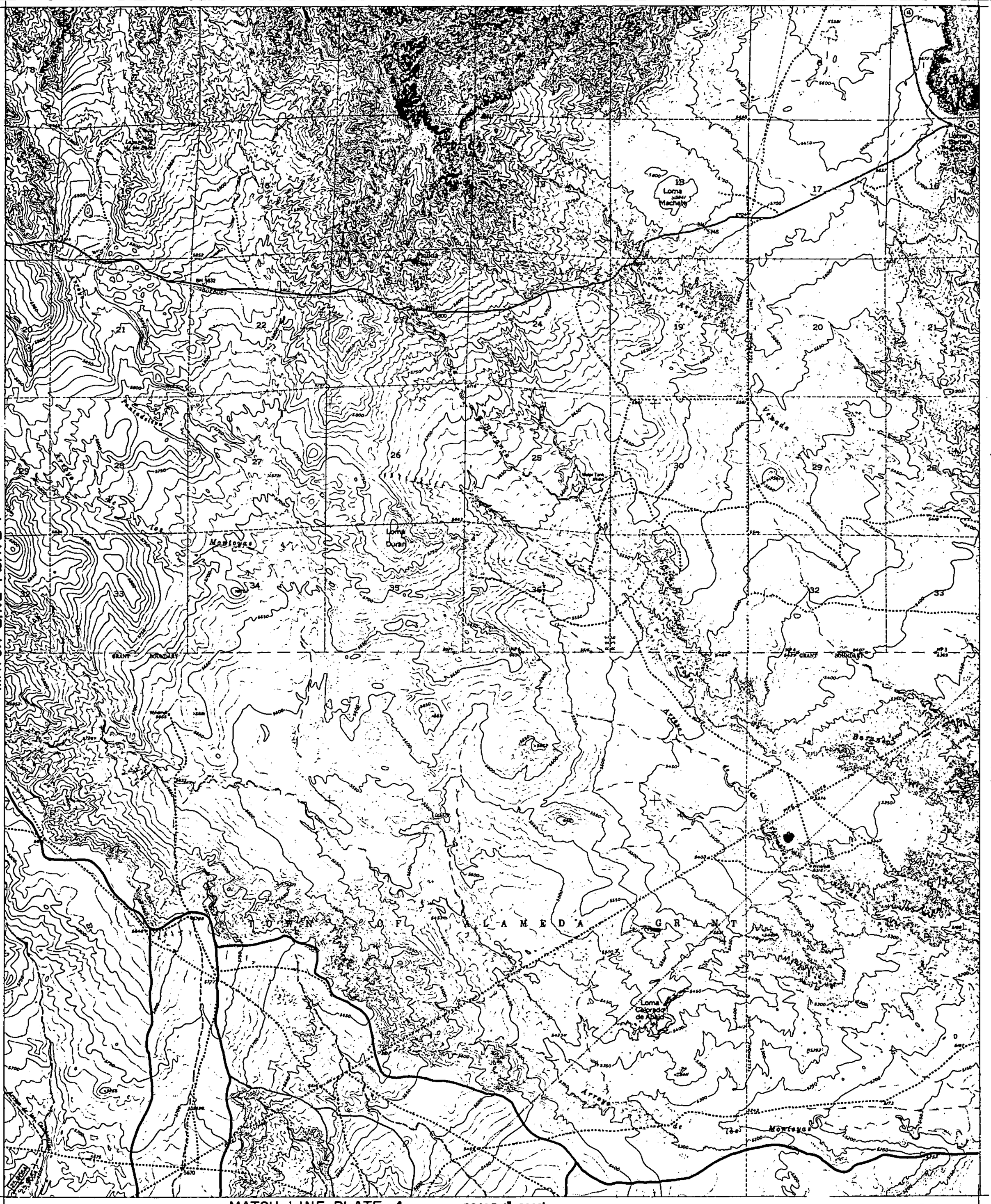
0 2000 4000 6000

CONTOUR INTERVAL 10 FEET
DATUM IS MEAN SEA LEVEL

MASTER PLAN OF DRAINAGE
CITY OF ALBUQUERQUE, NEW MEXICO
AND ENVIRONS

PLATE 1

MATCH LINE PLATE I



MATCH LINE PLATE 4

LEGEND

- LIMIT OF DRAINAGE AREA
- LIMIT OF SUB-DRAINAGE AREA
- DRAINAGE EASEMENT
- NATURAL DRAINAGE AREA
- DRAINAGE AREA AFTER DEVELOPMENT — 2537A / 2537A
- WEATHER STATION (CLIMATIC CONDITIONS) — △
- RUNOFF MEASURING STATIONS — ○

SCALE 1"=2000'

0 2000 4000 6000

CONTOUR INTERVAL 10 FEET
DATUM IS MEAN SEA LEVEL

MASTER PLAN OF DRAINAGE CITY OF ALBUQUERQUE, NEW MEXICO AND ENVIRONS

PLATE 2

MATCH LINE PLATE 1

MATCH LINE PLATE 4

MATCH LINE PLATE 10

LEGEND

LIMIT OF DRAINAGE AREA
LIMIT OF SUB-DRAINAGE AREA
DRAINAGE EASEMENT
NATURAL DRAINAGE AREA
DRAINAGE AREA AFTER DEVELOPMENT
WEATHER STATION (CLIMATIC CONDITIONS)
RUNOFF MEASURING STATIONS

2537A
2537A

SCALE 1"-2000'

0 2000 4000 6000

CONTOUR INTERVAL 10 FEET
DATUM IS MEAN SEA LEVEL

MASTER PLAN OF DRAINAGE
CITY OF ALBUQUERQUE, NEW MEXICO
AND ENVIRONS

PLATE 3

MATCH LINE PLATE 2

MATCH LINE PLATE 3

MATCH LINE PLATE 5

MATCH LINE PLATE 9

LEGEND

- LIMIT OF DRAINAGE AREA
- LIMIT OF SUB-DRAINAGE AREA
- DRAINAGE EASEMENT
- NATURAL DRAINAGE AREA — 2537A
- DRAINAGE AREA AFTER DEVELOPMENT — 2537A
- WEATHER STATION (CLIMATIC CONDITIONS) — △
- RUNOFF MEASURING STATIONS — ○

SCALE 1"-2000'

0 2000 4000 6000

CONTOUR INTERVAL 10 FEET

DATUM IS MEAN SEA LEVEL

MASTER PLAN OF DRAINAGE
CITY OF ALBUQUERQUE, NEW MEXICO
AND ENVIRONS

PLATE 4

MATCH LINE PLATE 4

MATCH LINE PLATE 6

MATCH LINE PLATE 8

LEGEND

- LIMIT OF DRAINAGE AREA
- LIMIT OF SUB-DRAINAGE AREA
- DRAINAGE EASEMENT
- NATURAL DRAINAGE AREA — 2537A
- DRAINAGE AREA AFTER DEVELOPMENT — 2537A
- WEATHER STATION (CLIMATIC CONDITIONS) — △
- RUNOFF MEASURING STATION — ○

SCALE 1"-2000'

0 2000 4000 6000

CONTOUR INTERVAL 10 FEET

DATUM IS MEAN SEA LEVEL

MASTER PLAN OF DRAINAGE CITY OF ALBUQUERQUE, NEW MEXICO AND ENVIRONS

PLATE 5

MATCH LINE PLATE 5

MATCH LINE PLATE 7

SANDOVAL CO
BERNALILLO CO

SANDIA PUEBLO

GRANT BOUNDARY (APPROXIMATE)

SANDIA MOUNTAINS

ALBUQUERQUE

ELEVATION 5000

Sandia Peak

Sandia Crest

Juan Tabo Canyon

Juan Tabo Cabin

Juan Tabo Picnic Area

Juan Tabo Ranger Sta

Las Huertas Picnic Area

Las Huertas Canyon

Las Huertas Creek

Capulin Canyon

Capulin Peak

Capulin Spring

Capulin Picnic Area

Osha Canyon

Osha Creek

Palomas Peak

Arroyo

Madera Canyon

Madera Picnic Area

Dry Camp Picnic Area

Tecolote Peak

Tecolote Canyon

Tijeras Canyon

Cienega Canyon

Cienega

Cienega Picnic Area

Sulphur Canyon

Sulphur Spring

Sulphur Picnic Area

Cañoncito

TIJERAS DRAINAGE AREA

Canyon

Pino

Canyon

Cañoncito

Canyon

Canyon

Canyon

MATCH LINE PLATE 6

MATCH LINE PLATE 8

MATCH LINE PLATE 14

TIJERAS DRAINAGE AREA

TIJERAS DRAINAGE TO THIS POINT 36,650 CFS

COYOTE - TIJERAS DRAINAGE AREA

CABEZON

CAÑON DE CARNUE

ALBUQUERQUE

SANDIA MILITARY RESERVATION

CIBOLA NATIONAL FOREST

LEGEND

- LIMIT OF DRAINAGE AREA
- LIMIT OF SUB-DRAINAGE AREA
- DRAINAGE EASEMENT
- NATURAL DRAINAGE AREA — 2537A
- DRAINAGE AREA AFTER DEVELOPMENT — 2537A
- WEATHER STATION (CLIMATIC CONDITIONS) — △
- RUNOFF MEASURING STATIONS — ○

SCALE 1"-2000'

0 2000 4000 6000

CONTOUR INTERVAL 10 FEET

DATUM IS MEAN SEA LEVEL

MASTER PLAN OF DRAINAGE
CITY OF ALBUQUERQUE, NEW MEXICO AND ENVIRONS

PLATE 7

MATCH LINE PLATE 9

MATCH LINE PLATE 7

MATCH LINE PLATE 13

LEGEND

- LIMIT OF DRAINAGE AREA
- LIMIT OF SUB-DRAINAGE AREA
- DRAINAGE EASEMENT
- NATURAL DRAINAGE AREA
- DRAINAGE AREA AFTER DEVELOPMENT — 2537A / 2537A
- WEATHER STATION (CLIMATIC CONDITIONS) — △
- RUNOFF MEASURING STATION — ○

SCALE 1"-2000'

0 2000 4000 8000

CONTOUR INTERVAL 10 FEET

DATUM IS MEAN SEA LEVEL

MASTER PLAN OF DRAINAGE CITY OF ALBUQUERQUE, NEW MEXICO AND ENVIRONS

PLATE 8

MATCH LINE PLATE 4

MATCH LINE PLATE 10

MATCH LINE PLATE 8

MATCH LINE PLATE 12

LEGEND

- LIMIT OF DRAINAGE AREA
- LIMIT OF SUB-DRAINAGE AREA
- DRAINAGE EASEMENT
- NATURAL DRAINAGE AREA — 2537A
- DRAINAGE AREA AFTER DEVELOPMENT — 2537A
- WEATHER STATION (CLIMATIC CONDITIONS) — △
- RUNOFF MEASURING STATIONS — ⊙

SCALE 1"-2000'

0 2000 4000 8000

CONTOUR INTERVAL 10 FEET

DATUM IS MEAN SEA LEVEL

MASTER PLAN OF DRAINAGE
CITY OF ALBUQUERQUE, NEW MEXICO
AND ENVIRONS

PLATE 9

MATCH LINE PLATE 3

El Rincon

T O W N O F A T R I S C O G R A

MATCH LINE PLATE 9

MATCH LINE PLATE 11

P A J A R I T O

G A N T

LEGEND

- LIMIT OF DRAINAGE AREA
- LIMIT OF SUB-DRAINAGE AREA
- DRAINAGE EASEMENT
- NATURAL DRAINAGE AREA — 2537A
- DRAINAGE AREA AFTER DEVELOPMENT — 2537A
- WEATHER STATION (CLIMATIC CONDITION) — △
- RUNOFF MEASURING STATIONS — ⊙

SCALE 1"-2000'

0 2000 4000 8000

CONTOUR INTERVAL 10 FEET

DATUM IS MEAN SEA LEVEL

MASTER PLAN OF DRAINAGE
CITY OF ALBUQUERQUE, NEW MEXICO AND ENVIRONS

PLATE 10

MATCH LINE PLATE 10

MATCH LINE PLATE 12

LEGEND

LIMIT OF DRAINAGE AREA
LIMIT OF SUB DRAINAGE AREA
DRAINAGE EASEMENT
NATURAL DRAINAGE AREA — 2537A
DRAINAGE AREA AFTER DEVELOPMENT — 2537A
WEATHER STATION (CLIMATIC CONDITIONS)
RUNOFF MEASURING STATIONS

SCALE 1"-2000'

0 2000 4000 8000

CONTOUR INTERVAL 10 FEET
DATUM IS MEAN SEA LEVEL

MASTER PLAN OF DRAINAGE
CITY OF ALBUQUERQUE, NEW MEXICO
AND ENVIRONS

PLATE II

MATCH LINE PLATE 9

MATCH LINE PLATE 11

MATCH LINE PLATE 13

LEGEND

LIMIT OF DRAINAGE AREA	———
LIMIT OF SUB-DRAINAGE AREA	———
DRAINAGE EASEMENT	- - - -
NATURAL DRAINAGE AREA	2537A
DRAINAGE AREA AFTER DEVELOPMENT	2537A
WEATHER STATION (CLIMATIC CONDITIONS)	△
RUNOFF MEASURING STATIONS	⊙

SCALE 1"-2000'

0 2000 4000 8000

CONTOUR INTERVAL 10 FEET
DATUM IS MEAN SEA LEVEL

MASTER PLAN OF DRAINAGE
CITY OF ALBUQUERQUE, NEW MEXICO AND ENVIRONS

PLATE 12

MATCH LINE PLATE 8

MATCH LINE PLATE 12

MATCH LINE PLATE 14

NATURAL PONDING AREA APPROXIMATELY 2200 ACRE FEET

FORCE LINE NO LIFT

TOTAL NATURAL DRAINAGE

FORCED LINE 60' LIFT

NATURAL PONDING AREA APPROXIMATELY 2200 ACRE FEET

FORCED LINE 20' LIFT

NATURAL PONDING AREA APPROX. 37 ACRE FEET

DIKE

NATURAL OVERFLOW PONDING AREA

I N L E T A

F U E B L O

C A N A

LEGEND

LIMIT OF DRAINAGE AREA.
LIMIT OF SUB-DRAINAGE AREA
DRAINAGE EASEMENT
NATURAL DRAINAGE AREA — 2537A
DRAINAGE AREA AFTER DEVELOPMENT — 2537A
WEATHER STATION (CLIMATIC CONDITIONS)
RUNOFF MEASURING STATIONS

SCALE 1"-2000'

0 2000 4000 8000

CONTOUR INTERVAL 10 FEET
DATUM IS MEAN SEA LEVEL

MASTER PLAN OF DRAINAGE CITY OF ALBUQUERQUE, NEW MEXICO AND ENVIRONS

PLATE 13

MATCH LINE PLATE 7

MATCH LINE PLATE 13

COYOTE - TIJERAS DRAINAGE AREA

SANDIA MILITARY RESERVATION

ISLETA PUEBLO

GRANT

BERNALILLO CO
VALENCIA CO

LEGEND

- LIMIT OF DRAINAGE AREA
- LIMIT OF SUB DRAINAGE AREA
- DRAINAGE EASEMENT
- NATURAL DRAINAGE AREA
- DRAINAGE AREA AFTER DEVELOPMENT
- WEATHER STATION (CLIMATIC CONDITIONS)
- RUNOFF MEASURING STATIONS

SCALE 1"-2000'

0 2000 4000 6000

CONTOUR INTERVAL 10 FEET
DATUM IS MEAN SEA LEVEL

MASTER PLAN OF DRAINAGE
CITY OF ALBUQUERQUE, NEW MEXICO
AND ENVIRONS

PLATE 14