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RENO DRAINAGE STUDY

ANALYSIS OF THE PANTHER VALLEY DRAINAGE DEFICIENCY AREA

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PRELIMINARY

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CHAPTER I

INTRODUCTION

A. PROJECT BACKGROUND

The City of Reno, Nevada is located at the base of the eastern slope of the Sierra Nevada Mountain Range in the Truckee Meadows basin. The present population is approximately 101,000. Reno City limits encompasses approximately 28,200 acres and extends from approximately South McCarran Boulevard in the south to the Stead area in the north.

Perhaps the most significant hydrologic feature is the Truckee River that flows northeast out of Lake Tahoe, passing through the Reno-Sparks metropolitan areas before turning north to Pyramid Lake. The Truckee River has caused significant flooding in the past, though the flooding threat has been reduced by flood control dams in the upper reaches.

There have been numerous storm drainage reports (dating back to 1957) dealing not only with the Truckee River flood potential, but local drainage flood potential. Table 1 lists these various studies. In addition, there have been numerous smaller drainage studies completed for various subdivisions in the Reno area.

TABLE 1

STORM DRAINAGE REPORTS

A Master Plan Report on Storm Drainage and Sanitary Sewerage for the City of Reno, October 1957 - Clyde C. Kennedy.

An Addendum Report on Storm Drainage, August 1963 - Kennedy Engineers.

Flood Plain Information, Truckee River, Reno-Sparks-Truckee Meadows, Nevada, October 1970 - Department of the Army, Sacramento District, Corps of Engineers.

City of Reno In-house Storm Drain Deficiency Report, started 1976.

Truckee Meadows Investigation (Reno-Sparks Metropolitan Area) Nevada Plan for Channel Modifications - Truckee River - Twin Lakes Drive to U.S. Highway 395 (River Mile 55.12 to 50.49, March 1982 - Leeds, Hill and Jewett, Inc.

B. PRESENT PROJECT

Although a significant number of the proposed projects in the various drainage reports have been completed, there are still numerous isolated areas within the city where flooding continues to be a problem.

The City of Reno recently authorized a study that would review these various drainage deficiency areas in an attempt to define what the problem or problems are at the various locations. In addition, the City requested that the existing rainfall intensity duration-frequency curve for the Reno area developed in 1960 be updated. During the negotiations, it was decided that rainfall isopleth maps be developed in conjunction with the new rainfall intensity curves which would enhance the rainfall intensity accuracy for those areas not adjacent to the Reno-Cannon International Airport.

At the present time, twenty drainage deficiency areas have been identified Table 2 lists the various deficiency areas by priority.

This particular report analyzes Drainage Deficiency Area Priority 15 at Panther Valley.

TABLE 2

STORM DRAINAGE DEFICIENCY AREAS[1]

PRIORITY	LOCATION
1	Stead - including Stead Blvd. and Old State Complex (full drainage study)
2	Huffaker Hills Area
3	Harding and Gulling
4	Plumas Street near West Moana
5	Rewana Farms, north of Peckham
6	Market Street and Miami Way
7	Roberts Street near Yori Avenue (Libby C. Booth School)
8	Thomas Jefferson Drive and Aguila Avenue
9	Belford Road and Sharon Way
10	Second Street at the railroad crossing
11	Charles Drive - Clough Road area
12	Marsh Avenue and LaRue Avenue
13	Riverside Drive and Ralston Street
14	Lake Ridge Golf Course area
15	Panther Valley area
16	Longley Lane and McCarran Blvd.
17	University Drain at Longley Lane
18	Grant Drive and West Moana Lane
19	Parr Blvd. near Catron Drive
20	Dry Creek Drainage

[1]Refer to "Reno Drainage Study Preliminary Report: Analysis of Drainage Deficiency Areas Within the City Limits", December 1984, Figure 1.

CHAPTER II

DESIGN CONSIDERATIONS

A. INTRODUCTION

The purpose of the individual Storm Drainage Deficiency Reports is to analyze a particular problem area identified by the City staff as given in the Priority List, Table 2. The design considerations necessary for this analysis are set forth in this chapter.

B. STORM DRAINAGE SYSTEM

The city has storm drainage mapping that is relatively up to date. There are several areas where the existing facilities are inadequate, especially when considering future growth.

Part of the scope of this study is to field verify the existing storm drainage structures at the various drainage deficiency areas and incorporate this storm drainage network in the final map.

C. HYDROLOGY - HYDRAULICS CONSIDERATIONS

1. HYDRAULIC DESIGN

The city has a policy requiring design of the majority of storm water facilities to pass 5-year return frequency storm flows. Major drainage facilities, where the drainage basin is 100 acres or greater, are sized to pass 100-year return frequency storm flows. Although the ordinance does not state it specifically, it is recommended that storm drains sized for 5-year storm events be sized to pass these flows with no static head. This will allow additional flows to pass with some head for storm events exceeding the 5-year return frequency.

2. RATIONAL METHOD

The Rational Method is the most used method in this country for computing quantities of storm water runoff. It allows consideration of local conditions and relates runoff directly to rainfall by the following equation:

$$Q = cia$$

where:

- Q = peak runoff rate in cubic feet per second
- c = runoff coefficient which is actually the ratio of the peak runoff rate for particular surface types and permeabilities to the average rainfall rate for a period known as the time of concentration.
- i = average rainfall intensity in inches per hour for a period equal to the time of concentration.
- a = drainage area in acres

3. RUNOFF COEFFICIENT

The proper selection of runoff coefficient "c" is critical for storm water runoff computations. It is dependent on a number of factors including slope condition and imperviousness of the surface, as well as the degree of saturation.

The expected land use can greatly affect the amount of runoff which will significantly increase with increased development. After discussions with City staff, values of the runoff coefficient "c" were developed based on the present and future Reno Land Use Maps for the area as shown on Figures 1 and 2. They are listed in Table 3.

Present Land Use Map

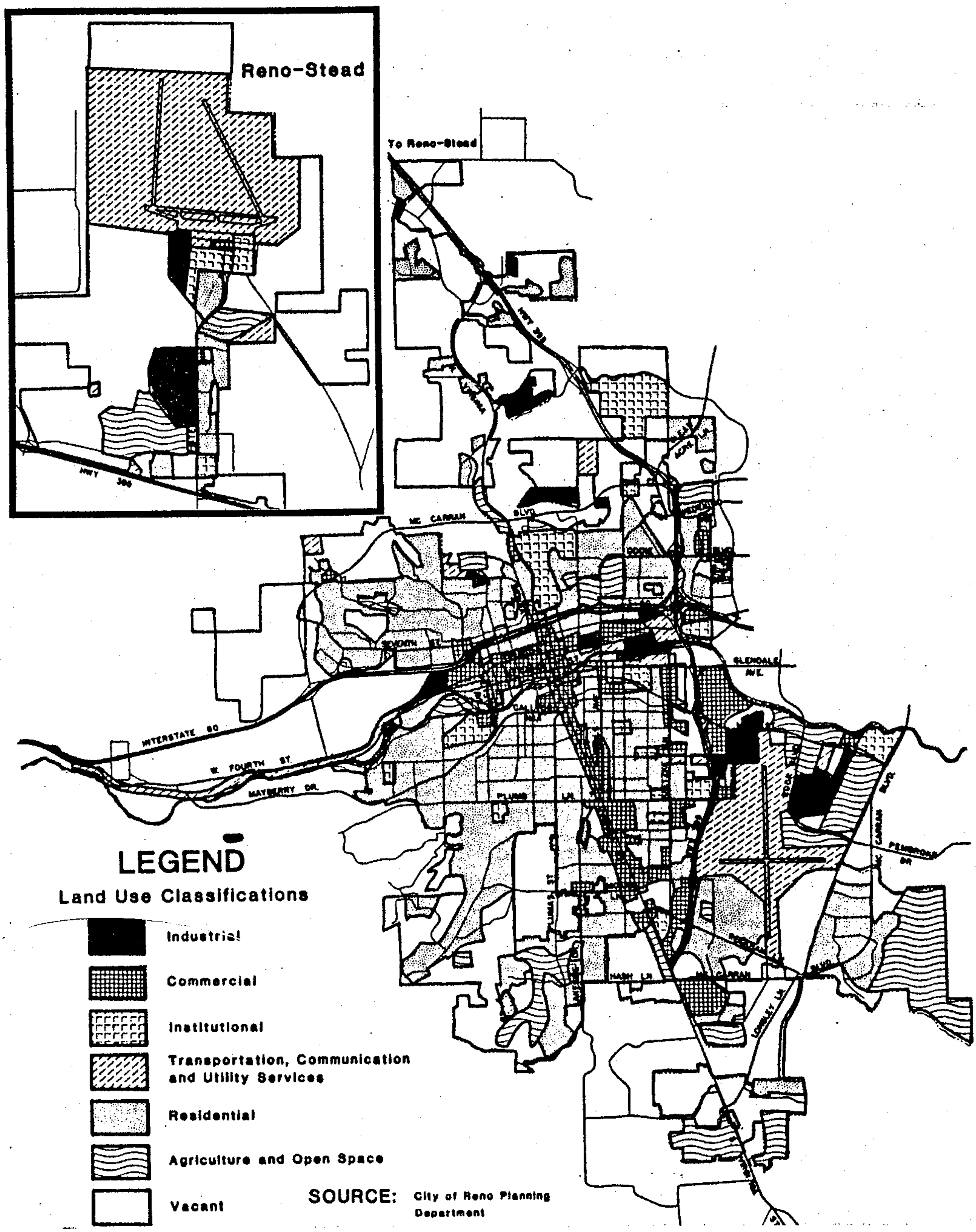


Figure 1

Future Land Use Map

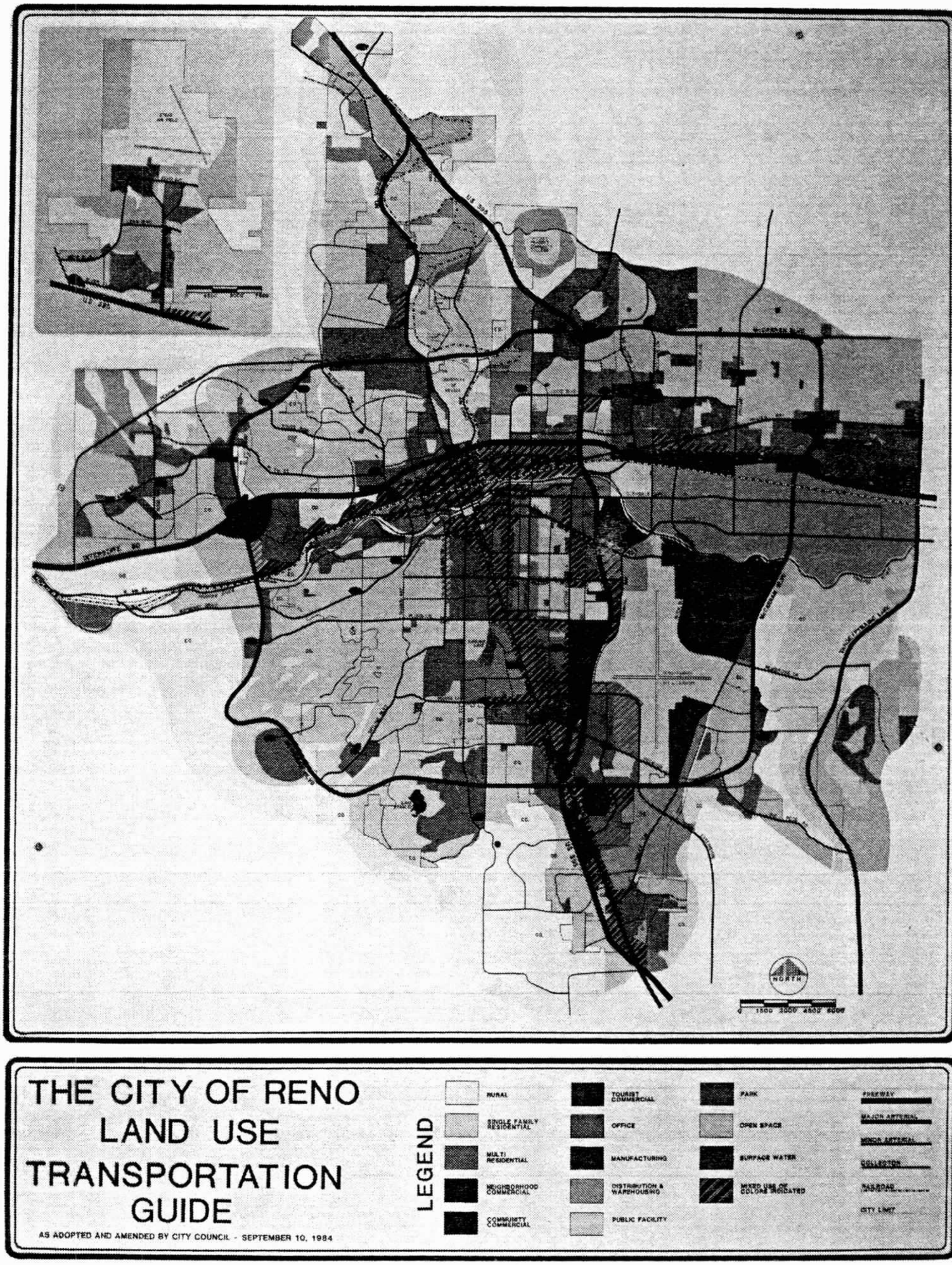


Figure 2

TABLE 3

RUNOFF COEFFICIENTS "C"

Land Use Type	Runoff Coefficient "C"
Rural	0.25-0.35
Single Family Residential	0.45-0.55
Multi-residential	0.60-0.70
Neighborhood Commercial	0.85
Community Commercial	0.85
Tourist Commercial	0.85
Office	0.85
Manufacturing	0.85-0.90
Distribution and Warehousing	0.85-0.90
Public Facility	0.50-0.85
Park	0.25
Open Space	0.20-0.30

These values are somewhat conservative when used for entire areas, as it assumes maximum build-out in all these areas. Substantial portions of rural and low density areas may not develop to full potential. However, it is difficult to determine where growth will or will not develop, and costs of storm water drainage systems are very expensive. Thus, it is generally preferable to size the system for maximum development rather than having to upsize the system later.

The City Ordinance generally does not allow increased runoff from that already existing for new developments. All additional runoff generated from increased development must be kept on site by the use of on-site storage. This is especially true if the increased runoff would exceed the existing downstream storm drainage facilities capacity.

However, exceptions have been allowed in the past. Thus, it is recommended that a more detailed hydraulic study be required for the individual drainage systems at the design or pre-design stage. At this time the actual zoning or land use for the area in question should be re-evaluated to arrive at an acceptable runoff coefficient "c". This report will consider two cases. Case I assumes that additional runoff will be kept on site. This case will use Figure 1, the present land use map, to develop runoff coefficients. Case II assumes that additional runoff will be allowed and maximum development will take place. This case will use Figure 2, the future land use map, to develop runoff coefficients.

4. RAINFALL INTENSITY AND DURATION

An accurate measurement of rainfall intensity and duration "i" is necessary to determine storm water flows for a particular area.

The existing rainfall intensity-duration-frequency (IDF) curves for the Reno area were developed in 1960 and are based on rainfall records through 1939.

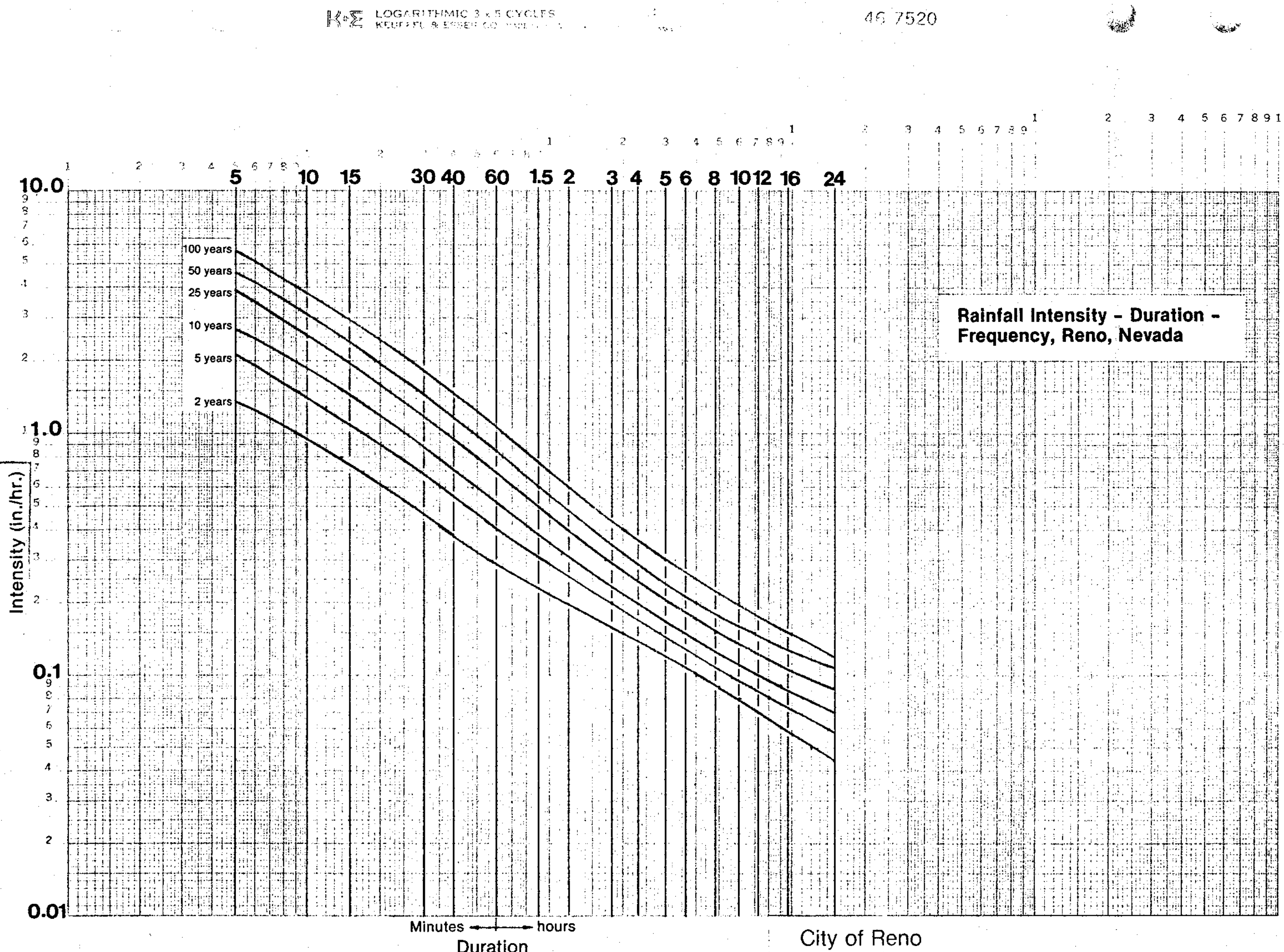
One of the major tasks of this study is to develop new rainfall IDF curves based on more updated data that is available.

In addition, the scope of work includes the analysis of spatial variation of rainfall in the study area. This requires developing rainfall isopleth maps for both the summer and winter seasons, based upon available rain gauging stations in the area.

Three sources of rainfall information were analyzed in developing the rainfall IDF curves. These are:

1) National Weather Service, "Technical Paper No. 40," 1964
2) NOAA "Rainfall Atlas 2 - Volume Vii," 1972
3) Analysis of raw precipitation data from the National Weather Service Climatic Center in Asheville, North Carolina for the Reno-Cannon Airport from 1952 to 1983.

Rainfall IDF curves were developed from each individual source of information. After careful analysis it was decided that the curve based strictly on rainfall records at the Cannon Airport (Figure 3) combined with the use of the rainfall isopleth maps would present the most accurate rainfall intensity records for the various drainages in the study area. It should be noted that the data presented is recommended for use only within the study area. Use of the rainfall IDF curves for areas outside the study area should be done so with caution and careful engineering judgment.



City of Reno

Rainfall Intensity - Duration - Frequency Curves for General Reno Area

Based on Rainfall Data from Cannon Airport Gauging Station

Figure 3

The rainfall isopleth maps are based on nine unofficial gauging stations in the area that have daily rainfall information available for use. These stations are located at Dickerson Road, Royal Drive, Upper Skyline, Ganser, La Veaga Court, Verdi, Sparks Fire Station, Sierra Sage Road, and Christmas Tree.

Each rainfall event at every location was compared and a ratio computed to the corresponding values recorded by the local weather service station at the Reno Cannon Airport. The summer season was assumed to extend from May through October and the winter season was assumed to extend from November through April. The two rainfall isopleth maps are shown as Figures 4 and 5.

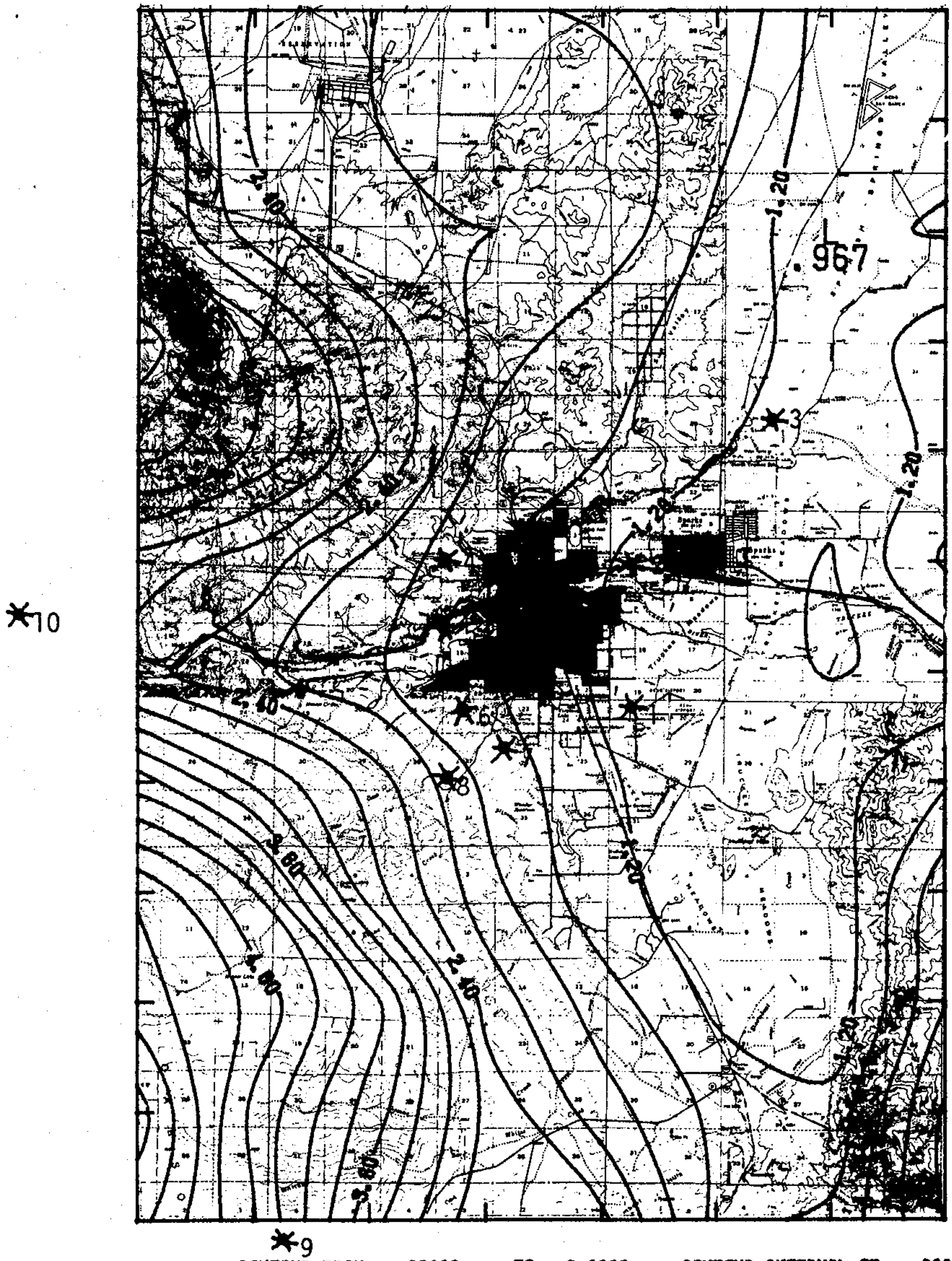
2,000-scale overlays of these isopleth maps have been completed to be used in conjunction with the standard 7.5 minute topographic quadrangle maps of the Reno area.

Figure 6 describes the use of the rainfall isopleth maps for a typical drainage area. Basically the drainage area is divided into subareas, each corresponding to the area under a particular isopleth range. A weighted average is obtained and this average is multiplied by the rainfall intensity taken from the rainfall IDF curve for the Reno-Cannon Airport to derive a modified rainfall intensity for the drainage basin in question.

In using these rainfall isopleth maps, it is recommended that a rainfall intensity correction factor be calculated for both the summer and the winter season. The highest correction factor should be used in calculating the rainfall intensity to be used in the Rational Formula.

City of Reno

Rainfall Isopleth Map for Wet Season November to April



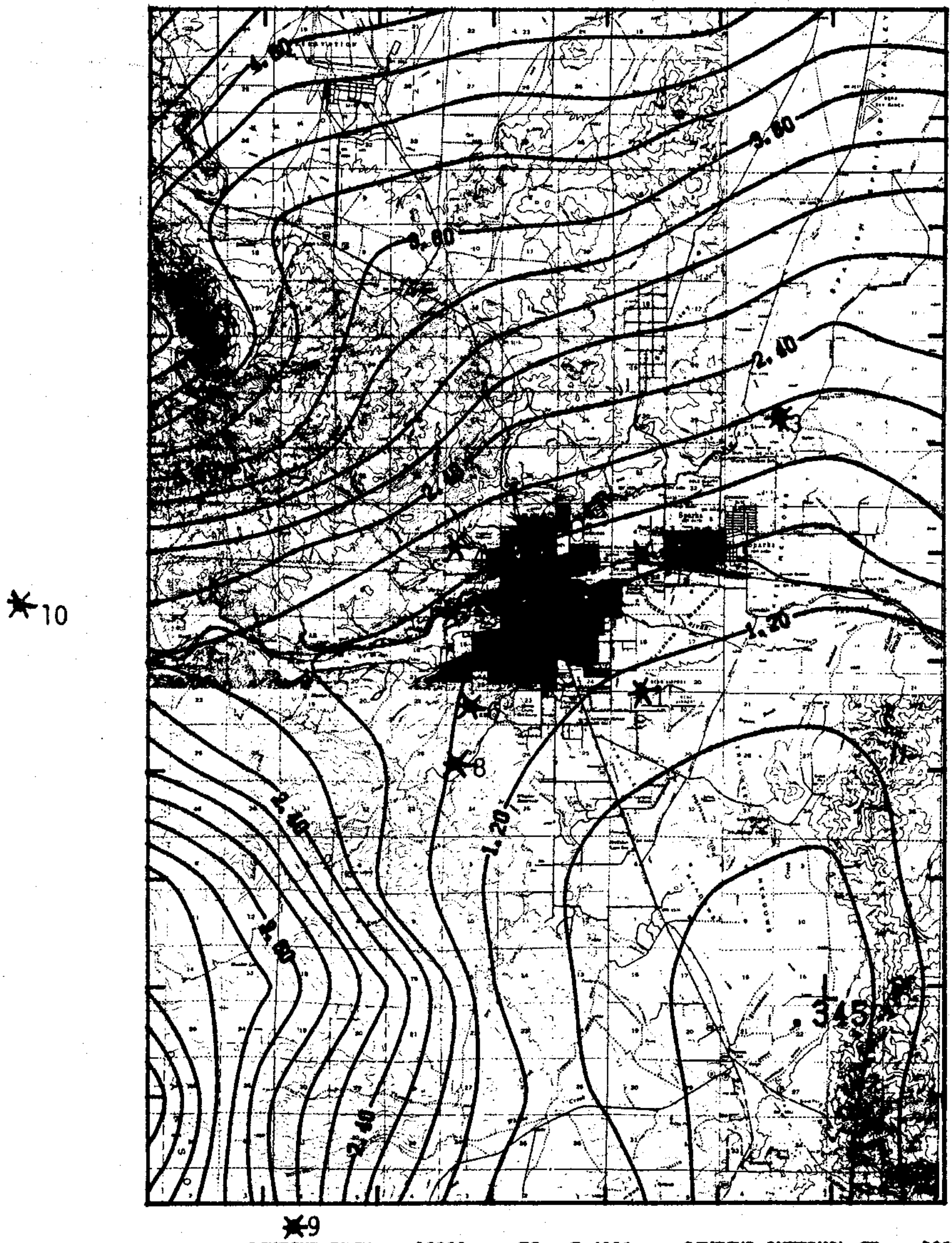
CONTOUR FROM .80000 TO 8.0000 CONTOUR INTERVAL OF .80000 PT(3,3)= 4.0878

1. Cannon Airport
2. Sparks Fire Station
3. La Veaga Ct.
4. Royal Dr.
5. Dickerson Rd.
6. Ganser
7. Sierra Sage Ln.
8. Upper Skyline
9. Christmas Tree
10. Verdi

Figure 4

City of Reno

Rainfall Isopleth Map for Dry Season May to October



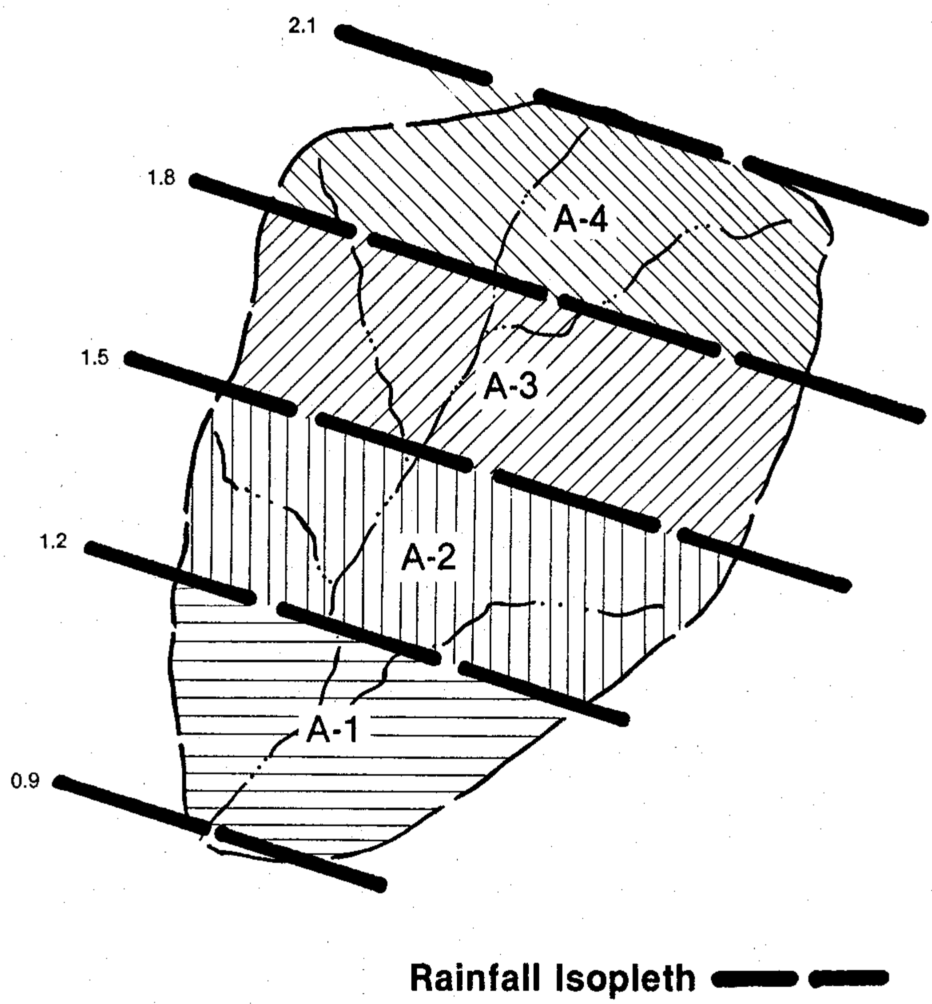
CONTOUR FROM .30000 TO 5.4000 CONTOUR INTERVAL OF .30000 PT(3,3)= 2.8281

1. Cannon Airport
2. Sparks Fire Station
3. La Veaga Ct.
4. Royal Dr.
5. Dickerson Rd.
6. Ganser
7. Sierra Sage Ln.
8. Upper Skyline
9. Christmas Tree
10. Verdi

Figure 5

City of Reno

Rainfall Isopleth Map Usage (Typical Example)



Rainfall Intensity Correction Factor =

$$\frac{\textbf{A-1}\left(\frac{0.9+1.2}{2}\right) + \textbf{A-2}\left(\frac{1.2+1.5}{2}\right) + \textbf{A-3}\left(\frac{1.5+1.8}{2}\right) + \textbf{A-4}\left(\frac{1.8+2.1}{2}\right)}{\mathbf{A_{TOTAL}}}$$

NOTE: This modified rainfall intensity factor is multiplied by the rainfall intensity value from the Cannon Airport Curves

Figure 6

5. TIME OF CONCENTRATION

The time of concentration, "t_c", is defined as the flow time from the most remote point in the drainage area to the point in question. It is composed of two parts, inlet time and conduit travel time. Inlet time consists of the time required for water to flow overland from the most remote point in the watershed to a defined channel such as a street gutter plus the gutter flow time to the first inlet. The time of concentration is affected by several factors such as steepness of terrain, vegetation or land cover, and existing soil moisture conditions.

Inlet time in this study for unimproved areas is determined using average overland velocities shown on Figure 7. (From SCS "Urban Hydrology for Small Watersheds", T.R. 55). Inlet time for improved areas can vary widely and accurate values are difficult to obtain. Values between 5 and 30 minutes are normally used. Design inlet times from 5 to 15 minutes are used for developed areas with steep slopes or closely spaced inlets. 10 to 15 minute periods are common for similar areas with flatter slopes and for areas with widely spaced inlets and/or very gentle slopes, inlet times of 20 to 30 minutes are normally used.

It is recommended that a minimum inlet time of 10 minutes be adopted by the City in this and future runoff analyses. A 5 minute time of concentration is unreasonable except for very small drainages and will give exceedingly high runoff valves that field analysis does not support.

D. ANALYSIS OF DRAINAGE DEFICIENCY AREAS

The third phase of this project addressed in the report in Chapter III is PROBLEM IDENTIFICATION. As is stated in Chapter I, twenty potential drainage deficiency areas have been identified by City staff for review.

City of Reno

Average Velocities for Estimating Travel for Overland Flow

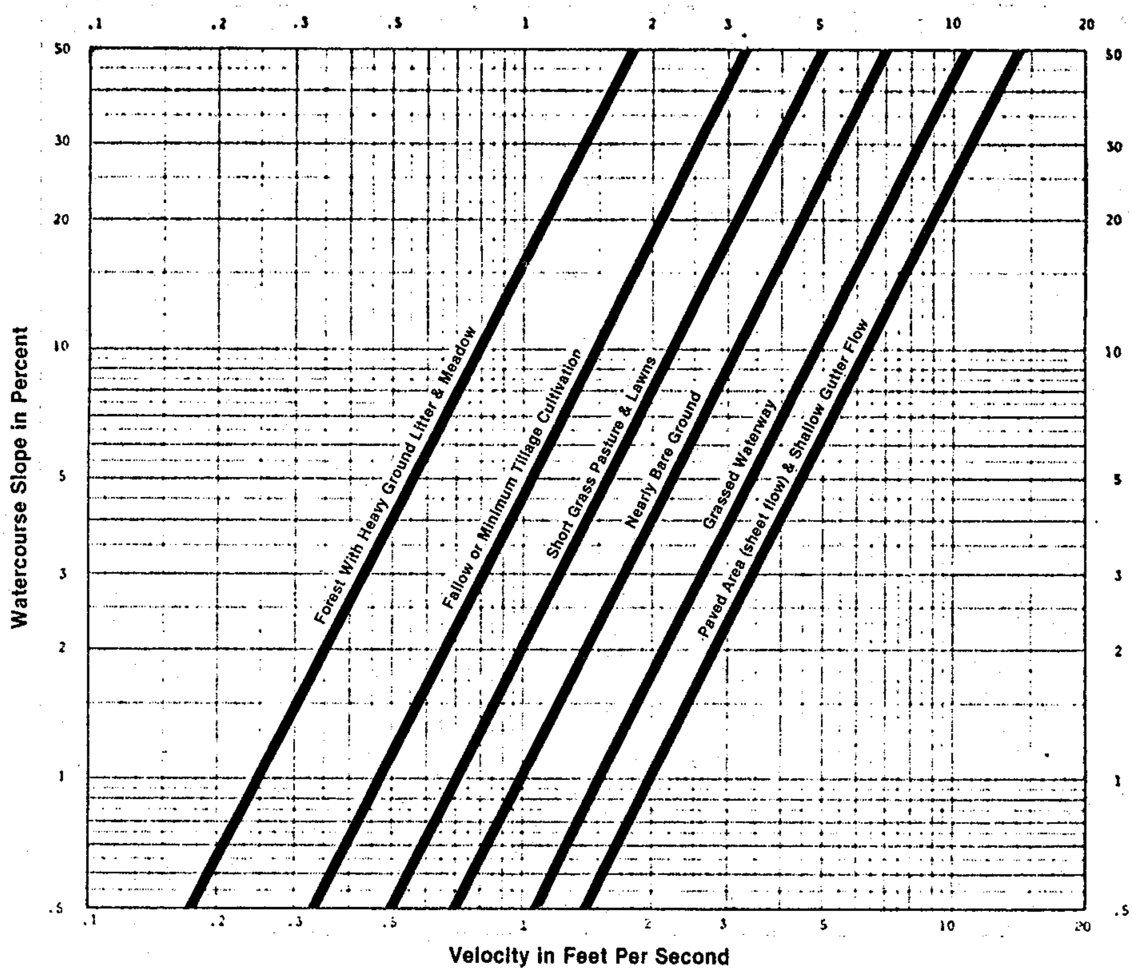


Figure 7

We propose to analyze these deficiency areas in the following manner: The existing storm drainage facilities will be plotted on 500 scale mapping available from Washoe County Department of Comprehensive Planning (formerly Regional Administrative Planning Agency) and will be field verified. Generally the flooding will occur at a particular node such as a culvert crossing. The drainage basin that contributes to a particular node will be identified. This drainage basin will be broken into sub-areas if required, each corresponding to the proposed land use (refer to Figures 1 and 2). Each land use has a runoff coefficient "C" assigned to it. A weighted average "C" will be calculated for the particular drainage basin.

A time of concentration "t_c" will be calculated as described in Section II-C-5 above. From this time of concentration, a rainfall intensity can be obtained from the rainfall IDF curve for the Reno-Cannon Airport. A modified rainfall intensity will be derived using the rainfall isopleth maps as described in Section II-C-4.

With this information, storm runoff flows for a five year return frequency storm (Q_5) and a one-hundred year return frequency storm (Q_{100}) can be calculated. These flows will be compared with the existing storm drainage node capacity to determine if the existing system is undersized. If the system is adequately sized, but flooding still occurs, attempts will be made to pinpoint where the problem may be, such as excessive siltation or poor inlet configuration.

CHAPTER III

FIELD ANALYSIS AND CONCLUSIONS

A. INTRODUCTION

The Panther Valley area is a large drainage system consisting of approximately 835 acres. (Refer to Figure 8 attached to the back of this report.)

The flows begin in the undeveloped rural hills in the north of the drainage basin and generally flow southwest through Panther Valley. The majority of the flows cross under a U.S. 395 overpass over the railroad and continue southwest in a drainage ditch along the Western Pacific Railroad tracks. The southern boundary of the drainage basin is basically North Virginia/Business 395.

The area presently consists of approximately 85% vacant or unimproved area, 13% residential and 2% industrial. This is slated for significant change on the future land use map which calls for approximately 40% rural/open space, 25% residential and 35% distribution and warehousing.

This change can significantly affect runoff and care should be taken at this time in planning future development with an emphasis on reducing any increase in storm runoff.

B. FIELD ANALYSIS

The Panther Valley drainage area consists largely of relatively steep, unimproved grassy hillsides with sparse sagebrush and other vegetation in the northern upper reaches. This gives way to flatter terrain and minor residential development consisting mainly of mobile homes in the Newport Lane and Collins Circle area. Farther downstream manufacturing and industrial sites become prevalent interspersed with residential development.

There are very few drainage facilities in the Antelope Road, Chisholm Trail area. There does appear to be a drainage easement connecting Chisholm, Collins and Antelope just north of Newport Lane. Existing 12-inch RCP's cross Antelope, Collins and Chisholm but are badly silted in.

Once flows cross Chisholm Trail they proceed west to a ditch along the southwest side of the Western Pacific Railroad.

Additional flows from a drainage basin to the north of Antelope Road also reach this ditch along the railroad. A dike was constructed approximately 7 feet high across the low swale just to the east of the railroad. It appears to be serving as a dam to create a storage basin and reduce flows reaching the railroad ditch.

The flows cross to the north side of the railroad via an 18-inch CMP just east of Link Lane and proceed west, crossing Link Lane via a 60-inch RCP.

There is a major drainage ditch approximately six feet deep with a seven foot bottom width and 1½:1 side slopes beginning downstream of this 60-inch culvert. A short distance downstream a second 60-inch RCP crosses a spur track. There is also a 42-inch RCP entering at this point that drains the Standard Motor Products, Inc. area. Downstream of these two pipe outlets the ditch deepens to close to twelve feet with a five foot bottom width and 1:1 side slopes.

The flows in the ditch enter a 42-inch RCP pipe just upstream (northeast) of the U.S. 395 overpass for a distance of approximately 500 feet discharging into a ditch downstream of Ranger Road. This pipe is partially blocked by significant amounts of tumbleweeds for perhaps 100 foot upstream of the inlet and the outlet shows significant siltation. Flows from the east generally cross Newport Lane as overland flow although there are a few small diameter pipe crossing that are in relatively poor shape. The flows collect along the east side of the railroad track in various low lying areas. Presently there is no place for this water to exit.

Flows from the northwest along both sides of U.S. 395 cross Panther Drive via a 48-inch CMP arch and enter a 24-inch RCP drop inlet to the 42-inch RCP approximately halfway along its length.

The ditch downstream of Ranger Road is approximately eight feet deep with a nine foot bottom width and 1:1 side slopes. Flows proceed south in this ditch along the west side of the Western Pacific Railroad. The ditch slowly deteriorates downstream. It becomes smaller and has considerable weed growth at the downstream end of the drainage basin. There is a private dirt road crossing the track and ditch at the southern end of the basin. Dual 24-inch CMP's cross this road. Just upstream a single 24-inch CMP crosses the railroad carrying flows from the east to the west side of the track. The ditch on the east side is not well defined. However, all flows generated on the east side of the tracks must exit via this culvert although much of the flows just pond up in low lying areas.

Downstream of the dual 24-inch CMP's a smaller drainage basin enters from the west crossing the road that parallels the tracks via a 36-inch CMP.

There is little in the way of drainage facilities to the east of the Western Pacific Railroad track. The area all tends to flow southwest towards the tracks. The area is relatively flat with resultant ponding in several areas. There is a poorly defined ditch along the east side of the railroad that carries the flows south to the 24-inch CMP crossing the tracks that is mentioned above.

C. ESTIMATED STORM RUNOFF

Estimated storm runoff is calculated for both the 5-year and the 100-year storm at selected nodes. These nodes are shown on Figure 8, the project boundary map appended at the back of the report. Table 4 summarizes these nodes, giving location, description of node, capacity of node and estimated storm runoff at the node. The existing capacity assumes inlet control. Generally a range is given. The lower value assumes no head at the inlet while the higher value is at maximum head on the culvert.

D. CONCLUSIONS

It is obvious from Table 4 that the existing drainage structures in the upper northwest reaches of the drainage (i.e. Antelope Road, Chisholm Trail area) are severely undersized.

Development in this area and south along Newport Lane is relatively sporadic and random. Care should be taken at this time to plan for storm drainage before additional development occurs. Planning should include potential storm drainage routing and obtaining drainage easements where required.

TABLE 4 - PANTHER VALLEY - EXISTING DRAINAGE FACILITIES SUMMARY

Node and Location	Existing Storm Drainage System	Existing Capacity (cfs)	Estimated Flows Present Land Use Q5 (cfs)	Estimated Flows Present Land Use Q100 (cfs)	Estimated Flows Future Land Use Q5 (cfs)	Estimated Flows Future Land Use Q100 (cfs)
a - Pipe crossing Antelope Rd. at Carolyn Way	12" RCP	3-6 30	40	105	40	105
b - Pipe crossing Antelope Rd. north of Carolyn Way	12" RCP	3-6 30	35	90	35	90
c - Pipe crossing Chisholm Trail	12" RCP	3-7 60	90	245	90	245
d - Beginning of ditch on northeast side of Western Pacific Railroad	unimproved ditch paralleling railroad	100 50	80	220	95	260
e - Link Rd. at Western Pacific Railroad crossing	18" CMP across RR 60" RCP across Link Rd.	7-22 100 120-250	120	300	170	430
f - End RR ditch at pipe inlet at U.S. 395 overpass	42" RCP	50-130 100	150	375	235	590
g - Pipe crossing of U.S. 395 northbound on-ramp	30" RCP	22-80 30	55	155	65	180
h - Pipe paralleling Panther Drive at U.S. 395 overpass	24" RCP	13-20 50	55	155	75	200
i - Pipe crossing U.S. 395 southbound on-ramp and off-ramp	30" RCP	22-65 100	45	120	150	405

TABLE 4 - PANTHER VALLEY - EXISTING DRAINAGE FACILITIES SUMMARY (continued)

Node and Location	Existing Storm Drainage System	Existing Capacity (cfs)	Estimated Flows Present Land Use		Estimated Flows Future Land Use	
			Q5 (cfs)	Q100 (cfs)	Q5 (cfs)	Q100 (cfs)
j - Pipe crossing Panther Drive near U.S. 395 overpass	50"x31" CMP arch to 24" RCP D.I.	40-65 13-25	190 100	460	320	775
k - RR ditch at private road crossing	Dual 24" CMP's	26-45	185 100	425	325	745
l - Pipe crossing RR just upstream of private road crossing	24" CMP	13-25	60 30	165	105	290

The drainage ditch along the east side of the Western Pacific Railroad upstream of the 18-inch CMP crossing is somewhat undersized for 100-year storm flows though it is adequate for 5-year storm flows. There is significant open area between the railroad and Newport Lane that will probably not develop much and can be used for overflow rather than being burdened with the costs of widening the existing ditch.

Downstream of Link Road to the 42-inch RCP the drainage ditch is adequate (to node e). However, the 42-inch RCP is slightly undersized for present land use 5-year storm flows and is significantly undersized for future land use storm flows if the additional runoff is not contained on site.

There is presently a problem with storm flows generated to the east of Newport Lane. There are few pipes that carry flows across the road and those that exist are silted in. Once flows cross Newport they pond between the road and the railroad in low lying areas. There is no improved ditch along the east side of the railroad and no culvert crossings of the railroad to allow flows to reach the major drainage ditch along the western side of the tracks.

The drainage systems crossing the freeway on- and off-ramps of U.S. 395 are adequate. A note should be made about node h. Estimated future flows ($Q5$ = 150 cfs and $Q100$ = 405 cfs) show a marked increase over present land use flows ($Q5$ = 45 cfs and $Q100$ = 120 cfs). This area is shown as distribution and warehousing on the future land use map and as basically vacant or unimproved on the present land use map. However, it is doubtful that much development will ever occur right next to the off-ramp and based on this assumption the existing pipe network is adequate.

Downstream of Ranger Road the ditch along the west side of the tracks becomes smaller and less defined with significant siltation and weed growth. It is not adequately sized for 100-year storm flows although it can handle the present 5-year storm flows. Again, there is a fair amount of open area that can presently be used as an overflow area although there is potential in this area for future development. Near the lower and southern limits of the drainage basin this ditch crosses a private dirt road via two 24-inch CMP's. These are much too small for the predicted flows. A 30-inch CMP crosses the road just downstream carrying flows from a minor drainage basin to the northwest.

Downstream and south of Ranger Road there is no improved or well defined ditch on the east side of the tracks. All flow that is generated tends to flow downhill to the tracks and pond in low areas. These flows eventually make their way south to a 24-inch CMP crossing the tracks (node k). This pipe is silted in and the outlet is severely damaged restricting flows further in a pipe that is much too small for the expected storm flows.

STORM DRAIN DEFICIENCY MAP

FIGURE 8

1"=500'