MOUNT LEMMON CULVERT STUDY

Prepared for:

PIMA COUNTY DEPARTMENT OF TRANSPORTATION
AND FLOOD CONTROL DISTRICT
201 N. STONE AVE., 4TH FLOOR
TUCSON, AZ 85701

Prepared by:

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NOVEMBER 19, 1993

Registered Professional Engineer (Civil)
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CLINTON MURRAY
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Date Signed 11/19/93
ARIZONA, U.S.A.

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I. INTRODUCTION

Residential and commercial properties along Carter Canyon Road, Turkey Run Road, and Sabino Canyon Parkway, must be accessed by crossing the streams which flow down the bottom of these canyons. All of these streams are steep and have sufficient drainage area to generate large quantities of runoff during significant storm events. The existing stream channels usually range in depth from 4 to 6 feet, and the embankments are near vertical. Property owners who must cross these channels have experienced frequent problems with floods washing out the structures or clogging with sediment deposition which renders them non-functional for future flows. Most of the existing culverts do not have adequate capacity to convey runoff by even the frequent, low-magnitude storms, and the floodwaters which cannot pass through the structure are diverted onto the adjoining road. This diversion has subsequently caused periodic damages to the roadway embankments and pavement surfaces along both public and private roads. Pima County Operations Department expends a significant amount of time and cost with the frequent repairs needed to repair these flood damages.

The Pima County Department of Transportation and Flood Control District authorized CMG Drainage Engineering, Inc., to conduct a study to define the scope of the problem and to recommend alternative approaches which would reduce the flood damage potential associated with future private driveway access installations. The specific objectives of this study are to provide guidelines for private access improvements, assess what improvements need to be made to existing private access across the watercourses and to provide more hydrologic and hydraulic information.

The study area centers around the watercourses along Carter Canyon Road, Turkey Run Road, and Sabino Parkway within the Summerhaven area on Mount Lemmon. The specific limits of the study are outlined on Figure 1 of this report. The scope of work outline which defines the specific tasks conducted by CMG Drainage Engineering, Inc., as a part of this study is provided in Appendix A of this report.

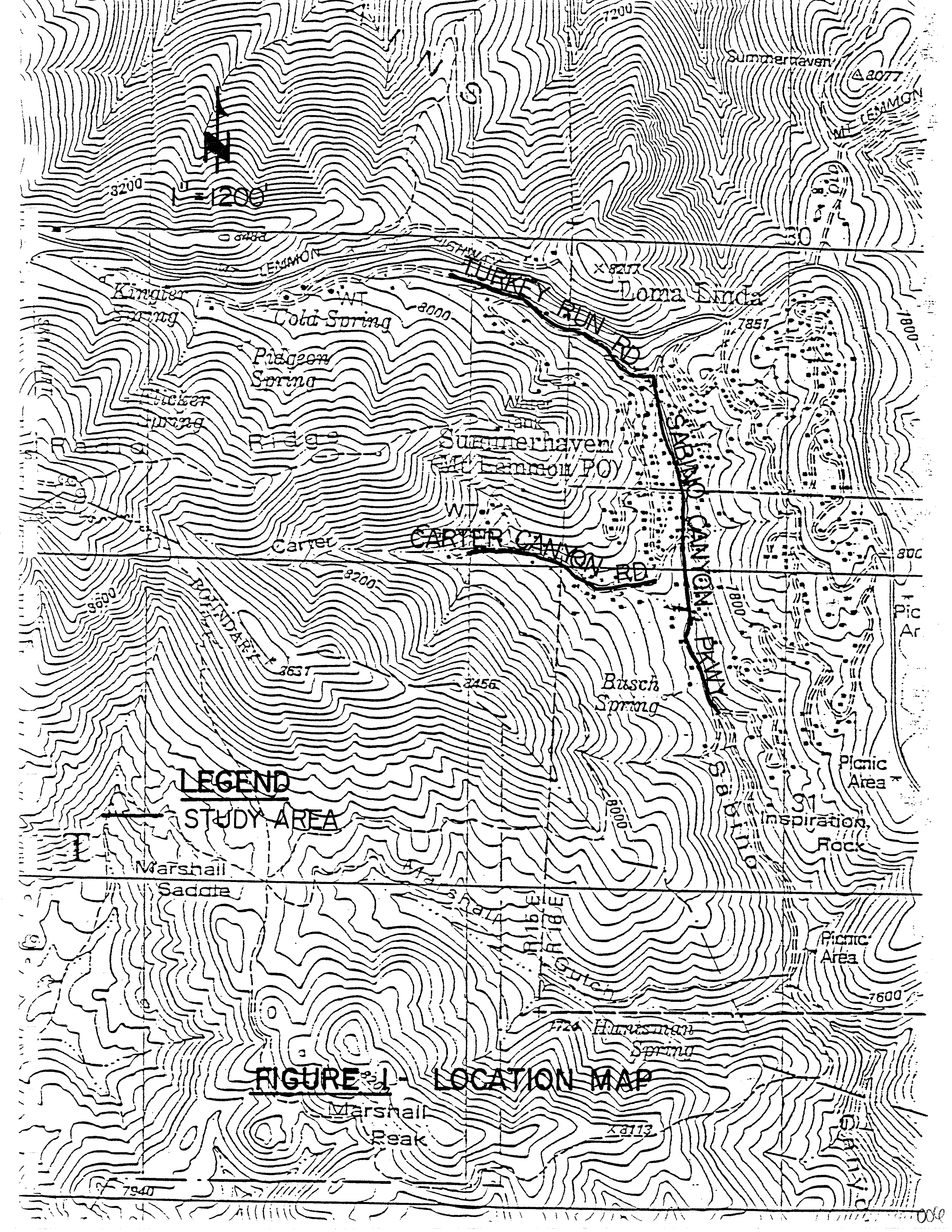

FIGURE I - LOCATION MAP

II. HYDROLOGIC ANALYSIS

The watershed areas draining to the study streams have contributing areas ranging from 0.37 to 1.37 square miles. The terrain within these watersheds is extremely steep, having cross slopes in the range of 30 to 50 percent. The channel slopes range from 4 to 10 percent. The forest cover consists of dense stands of Ponderosa pine and associated understory. Vegetative cover density was estimated to be at least 50 percent.

Figure 2 of this report is a copy of a U.S.G.S. quadrangle sheet showing the basin boundaries and concentration points where discharge determinations were made. Two approaches for determination of the discharge values were used for this study. The Pima County method was used because this is the standard methodology applied throughout Pima County; however, it was derived for application within the desert areas which predominate within the county. The U.S. Army Corps of Engineers computer program HEC-1 was the second methodology used to model the watershed runoff conditions and provide a comparison with the Pima County methodology results.

The six points of concentration where discharge determinations were made are shown on Figure 2. These concentration points were selected based upon a beginning and ending point of the stream study segments and where there is a significant change in contributing drainage area which would affect the computed discharge results. Tables 1 and 2 of this report list the results of the discharge determinations as made by the Pima County method and the U.S. Army Corps of Engineers computer program HEC-1. Examination of the results given in these

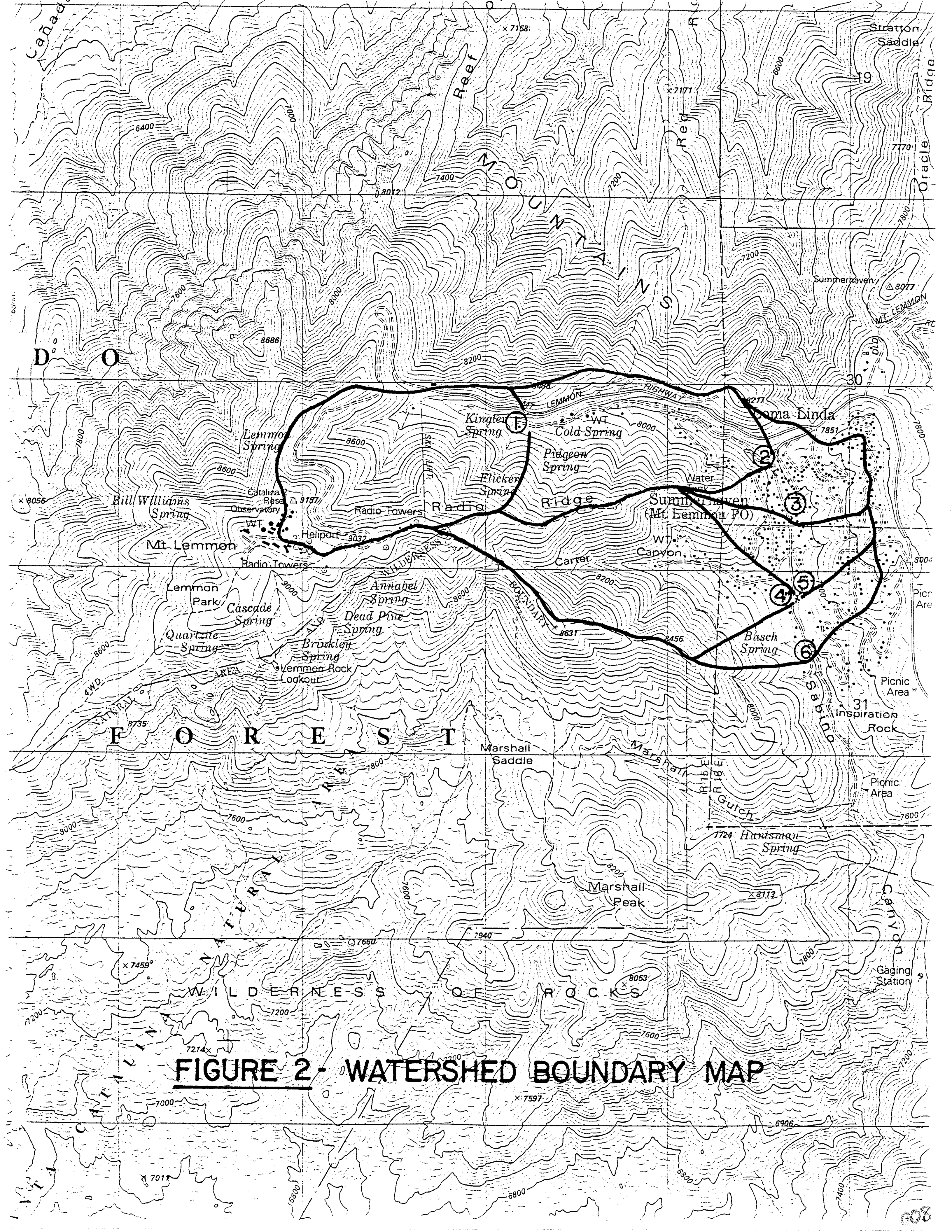

FIGURE 2 - WATERSHED BOUNDARY MAP

TABLE 1 - SUMMARY OF HYDROLOGIC COMPUTATIONS

BY PIMA COUNTY METHOD

Concentration Point	Drainage Area (acres)	Discharges (cfs) Q2	Q5	Q10	Q100
1	234.2	72	197	313	837
2	436.2	89	258	423	1207
3	504.2	90	277	449	1292
4	233.7	72	197	312	837
5	582.0	134	422	689	1441
6	879.3	123	403	662	1944

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TABLE 2 - SUMMARY OF HYDROLOGIC COMPUTATIONS

BY HEC-1 METHOD

Concentration Point	Drainage Area (acres)	Discharges (cfs) Q2	Q5	Q10	Q100
1	234.2	101	184	277	649
2	436.2	168	304	407	1114
3	504.2	190	342	532	1267
4	233.7	95	197	295	658
5	582.0	214	394	580	1413
6	879.3	326	598	879	2137

tables will show that the results given by both methodologies are relatively consistent. Generally, the Pima County method gives slightly higher results than the U.S. Army Corps of Engineers HEC-1 model; however, there are instances where this pattern is broken such as CP#5 and CP#6 where the HEC-1 model yielded higher results than the Pima County method.

After examination of these study results, a decision was made to utilize the HEC-1 model results. The primary basis for this decision was merely the uncertainty regarding the application of the Pima County method to a Ponderosa pine forest area. Utilization of the Pima County method results would not have substantively changed the results of this study because the discharge values yielded by both methodologies are relatively similar.

Hydrologic design data sheets for the Pima County methodology discharge determinations are provided in Appendix B of this report. Appendix C contains the HEC-1 model input and output.

III. FIELD SURVEYS AND INTERVIEWS

3.1 Field Investigations

Field investigations were conducted to identify existing channel conditions and to inventory the location and size of existing driveway culverts. A cross section of the stream channel was surveyed at locations which were observed to represent the average cross section geometrics for the local stream reach. These channel cross sections were used in subsequent tasks of this study to determine channel capacity and to use as a basis for design of future channel crossing structures. The locations where these channel cross sections were surveyed are shown on Figure 3 of this report.

All of the existing culvert structures were located and measured as a part of these field investigations. A photograph of each of these culverts is provided in Appendix D of this report. Appendix D also contains photographs of some of these culverts taken during the January 1993 floods. These photographs were provided by Mr. Robert Zimmerman of Mt. Lemmon Realty.

The Pima County Department of Transportation and Flood Control District's Operations Department was conducting repairs to the roadway embankments and some of the drainage culverts at the time these field surveys were being conducted. Several instances of roadway embankment damage and repairs were observed. These damages occurred as a result of the road proximity to the channel and as a result of flow diversions caused by the existing culvert structures. In many instances, the roadway embankment defines one of the channel banks, and this relationship precludes that periodic erosion will occur regardless of external influences such as changes in flow patterns caused by the culverts. There were also several locations where erosion had occurred just upstream or downstream of an existing culvert crossing, and these damages appear directly related to a diversion resulting from the obstruction or turbulence associated with flow spilling over a driveway. The location where the roadway damages and repairs were observed as a part of these field investigations is indicated on Figure 3 of this report.

Many of the existing culvert structures were observed to be partially or completely filled in with sediment deposition. This deposition is due to either the culvert having been constructed below natural stream grade or as a result of its inadequate size. Culverts with diameters in the range of 24 to 30 inches very frequently showed signs of sediment deposition and debris clogging. Some of the culverts with diameters of 36 inches or larger showed a lesser degree of blockage due to sediment transport conditions. The culverts which appeared to show the least amount of problem with debris deposits and sedimentation were the few structures which were built with the corrugated metal arch pipes. Mr. Bob Zimmerman's observations also confirmed that the arch pipes tended to have a lesser problem with blockage and were able to convey a larger amount of flow.

3.2 Records of Historical Flood Damages

The area residents and property owners who have property adjoining the study streams were sent a letter to notify them that this study was being conducted and to request any information regarding past flood and flood-related problems.

A copy of the letter of notification is provided in Appendix E of this report. Appendix E also contains copies of written responses and telephone communications given by those area residents who responded to this letter.

Pima County Department of Transportation and Flood Control District files on past drainage complaints were reviewed as a part of this study. There were 3 complaints related to placement of fill which the complainants believed to have caused a diversion of flow and subsequent damages to their property. There were 4 complaints related to culvert installations which the complainants felt were responsible for adverse changes in flow patterns. There was 1 complaint related to maintenance activities which Pima County was conducting on public roads.

All known permits which have been issued by Pima County for construction of drainageway crossings in the Summerhaven area were reviewed. Only 7 permits have been previously issued by Pima County for construction of private driveway crossings of the streams along Carter Canyon Road, Turkey Run Road, and Sabino Canyon Parkway. There are 16 driveway crossings and 6 public road crossings (within the project limits) in existence as of September 1993.

IV. HYDRAULIC ANALYSES

Hydraulic computations were conducted to determine the capacity of the existing stream channel and of the existing culvert structures beneath the driveways. The purpose of these calculations was to compare capacity of the natural channel with the culvert sections which then define the loss of conveyance resulting from those culvert structures.

Normal depth computations were conducted to determine the capacity of the typical channel sections which represent distinct reaches of the study streams. A table summarizing the results of the hydraulic computations along with the normal depth calculation sheets are provided in Appendix F of this report. The first page following the summary of the capacity computations contains a line diagram showing the cross section location, channel depth, and computed capacity for each of the stream sections surveyed as a part of this study.

The capacity of the existing culvert structures was estimated using the Bureau of Public Roads inlet headwater nomograph charts. A summary table listing the results of these hydraulic calculations is provided in Appendix G of this report. Examination of the summary table provided in Appendix G will show that the capacity of most of the existing driveway culverts is less than 100 cfs. The only structures which exceed this capacity are the locations having two or more pipes with a diameter of 36 inches or greater. Comparison of the computed culvert capacities with the flood frequency/discharge data given in Table 2 shows that almost all of these culverts have a discharge capacity less than the 2-year return period flow.

V. EVALUATION OF ALTERNATIVES

5.1 Objectives Statement

The intent of this study is to define the structure types and sizes which will function to pass as much flow as possible with little or no sediment deposition, to decrease the probability of a breakout onto the adjoining roadway, and to define required stabilization measures needed to prevent future washouts of new culvert structures. These objectives must be met without significant modification to the natural channel cross section.

The criteria used to evaluate the feasibility of alternative driveway crossing structures were the system's hydraulic capacity, its effect on natural channel flow characteristics, and the structure's installation cost. The intent of the design is to provide as much capacity as possible for passage of floodwaters, sediment, and debris and to direct the overflow across the structure in a manner that will facilitate its passage into the downstream channel rather than onto the adjoining roadway or its embankment. The cost of installation for the recommended measures must be equitable so as not to create such an economic burden that it prohibits use of the property.

5.2 Alternatives Evaluation

Seven different types of culvert crossing structures were evaluated as a part of this study. The following paragraphs present a brief description of those alternatives, potential impact upon the natural stream channel's capacity to convey water and sediment, and an estimate of their relative cost.

5.2.1 Circular Corrugated Metal Pipes

Circular corrugated metal pipe is the primary material used for almost all of the existing culvert structures within the project area. The diameter of the existing culverts varies from 24 inches to 48 inches, and their ability to pass water and sediment has largely been a function of the diameter. Existing pipes with diameters ranging from 24 to 30 inches have largely demonstrated to be inadequate for conveyance of water or sediment. Most of these culverts are washed out or have frequently been washed out and replaced during past floods. Most of these culverts have significant problems with sediment deposition. Existing pipes with a diameter of 36 inches or larger have shown to be relatively successful where there are two or more pipes at a given crossing site. The smaller pipe diameters also frequently show noticeable upstream aggradation which results from their inability to convey the water and sediment supply. Erosion has frequently occurred on the roadway surface and the adjoining roadway embankments on the downstream side of these culverts because of floodwaters which are diverted or pass over the top of the these driveways.

The minimum recommended diameter for application of these materials would be 36 inches, and this diameter should only be used where at least two culvert structures can be placed. Diameters of 42 to 48 inches or greater are preferred where the natural channel geometrics facilitate this large of a pipe size. The evaluation rating for this alternative is given in Table 3 of this report.

TABLE 3 - RATING EVALUATION FOR CULVERT ALTERNATIVES

Alternative Type	Hydraulic Capacity	Cost Range	Comments
Corrugated Metal Pipe (circular)	low	\$5,000-\$7,000	easy to install
Corrugated Metal Pipe (arch)	medium	\$6,000-\$8,000	easy to install
Concrete Box Culvert	high	\$10,000-\$12,000	higher materials unit cost due to small project size and travel distances
ConSpan/ConArch	high	\$10,000-\$12,000	higher materials unit cost due to small project size and travel distances
Continental Bridge	high	\$12,000-\$20,000	built in California
Railroad Flat Cars	medium	\$15,000-\$20,000	shipped from California

Assumptions
- \$90/cubic yard concrete delivered to site
- no concrete surface on crossing
- no unusual sub-surface soils conditions that would add to excavation cost
- no culvert wingwalls

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5.2.2 Corrugated Metal Arch Pipes

There are a few locations where corrugated metal arch pipes have been used at existing driveway culvert crossings. Observations made by Mr. Bob Zimmerman found these culverts to be performing better than their circular counterpart. Hydraulic computations have determined that the capacity of the corrugated metal arch pipes is approximately 40 to 50 percent greater than a comparable circular pipe with the same rise dimension. The broader width of the culvert opening facilitates the passage of sediments more readily than does the circular pipes. Additional capacity provided by this design approach generally allows installation of culvert structures which at least equal the 2-year return period discharge. Associated flow impacts from a corrugated metal arch pipe installation will be a function of the headwall design and whether or not the driveway grade can be set at an elevation low enough to facilitate passage of floodwaters over the top of the driveway crossing and back into the downstream channel, or whether or not the fill for the driveway crossing diverts flow onto the adjoining roadway area. The cost element for purchase of this type of pipe structure is slightly higher than that for comparable circular pipe; however, both approaches are relatively similar.

5.2.3 Concrete Box Culverts

Concrete box culverts provide a hydraulic capacity approximately twice that of a comparable corrugated metal or corrugated metal arch pipe with a similar rise dimension. The concrete box culverts are also superior in terms of ability to pass sediment and debris. Impacts upon the natural stream flow characteristics are minimized because they provide a much higher conveyance capacity. Cost of

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installation is estimated to be approximately twice that of the corrugated metal pipe installation having a comparable rise dimension.

5.2.4 ConSpan/ConArch Culverts

ConSpan culverts are a pre-cast reinforced concrete arch. Their advantage is a significant increase in hydraulic capacity over that offered by the corrugated metal pipe option. The minimum available span length for these culverts is 16 feet, which could be used on Sabino Canyon Parkway and some areas along Turkey Run Road. They are manufactured in Phoenix and must be transported approximately 150 miles to the Summerhaven area. This transport distance and the heavy equipment required to install the concrete members makes them significantly more costly than the corrugated metal pipe alternatives. Discussions with ConSpan representatives found that at least 500 feet of culvert length would have to be supplied in order for the ConSpan culverts to be cost competitive.

ConArch is the cast-in-place version of the ConSpan culverts. Discussion with ConArch representatives also determined that this methodology is not suited to small installations such as is needed for the Summerhaven area access driveways. The only approach by which this option would prove to be economically viable were if several of the structures were installed at one time to distribute the mobilization and setup costs.

5.2.5 Continental Bridge Structures

Continental Bridge manufactures a prefabricated steel bridge structure. These bridges have commonly been used along the river parks in Pima County. They are relatively attractive and have a very low deck height which minimizes obstruction of flow. The disadvantage of these structures are that they are manufactured out of state and the purchase cost along with transportation make them generally cost prohibitive for the driveway installations. This alternative does meet the design objectives of this project and should be permitted if the owners wish to bear the higher cost.

5.2.6 Railroad Flat Cars

A company in Redwood Valley, California, sells used railroad flat cars as a structural module for small bridge crossings. These flat cars come in a length of 89 feet, so they would have to be shortened significantly in order to be used at the driveway crossings. Their disadvantage is that they have a deck height of approximately 3 feet, which would present a significant obstruction to the passage of floodwaters. This factor and the cost of the structures does not make them feasible for application to the Summerhaven Area driveway installations.

5.2.7 Non-Standardized Alternatives

Research was conducted to identify possible methodologies which are not commonly used in Pima County. No other alternatives were discovered which suit the particular needs of this project. Discussions were held regarding the possibility of developing a design concept for a steel or wood bridge structure which would be applicable to this project's needs. This idea was abandoned because this study could not provide for the required engineering certifications associated with development of an alternative design.

VI. RESULTS AND RECOMMENDATIONS

The alternatives which performed best hydraulically are the concrete culverts and steel span bridges discussed in sections 5.2.3 through 5.2.5 of this report. These alternatives have the capacity to convey a quantity of flow approximately equal to the 5- to 10-year return period flood discharges. The limitation of using these approaches is economic because the cost generally runs in the range of $10,000 to $15,000. One of the objectives of this study was to keep the installation cost as low as possible while meeting the hydrualic design needs.

The only alternatives which fall in the range of the cost objectives are the corrugated metal pipe culverts. The corrugated metal arch pipes have approximately 40 to 50 percent more capacity than their circular equivalent and have proven via past floods to function better because they have less of a problem with debris and sediment deposition. The design capacity of these structures which can be achieved for the different stream segments studied as a part of this project is approximately equivalent to the 2-year return period event.

The recommendations offered in this report suggest using a standardized design for each of the study streams. The standardization facilitates development of concept construction drawings which can be used as a basis for building the structures, and the approach insures continuity of discharge capacity for future installations. The type of structure that this report recommends is the corrugated metal pipe arches (ACMP). The ACMP pipes provide approximately 40 to 50 percent more capacity than an equivalent circular pipe and are less prone to being blocked with debris and sediment.

Even though their hydraulic capacity is superior to a circular equivalent, the quantity of flow which can be conveyed by the ACMP pipes is limited to approximately the 2-year return period discharge. For flows larger than the 2-year return period event, the design concept must provide for the excess floodwaters to pass over the driveway crossing structure with minimal displacement of flow onto the adjoining roadway. This objective is achieved by limiting the rise dimension for the ACMP pipes to a minimum of 2 feet below the top of bank elevation on the side adjoining the road and by limiting the height of fill over the top of the pipes to the minimum allowable depth of 1.0 feet. Use of a minimum cover specification requires the ACMP pipe gauge to be 10 or 12 in order to insure a high enough rigidity to support anticipated live loads.

Inlet and outlet headwalls to the pipe culverts will be required since the design concept anticipates flow over the driveway crossing structure. The design concept provided herein is a modification to City of Tucson/Pima County standard detail #313. Concrete surfacing of the driveway crossing is also recommended; however, this design element of the crossing should be left to the discretion of the owner.

Some of the existing culverts have caused significant changes to the channel bottom profile by blocking the passage of sediment. Over time, these culverts have caused a significant amount of aggradation along the section of channel

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upstream of these culverts. The channel depth is usually only about 2 to 3 feet upstream of these locations. There are also some sections of channel that do not appear to have been altered by man-made activities, yet the depth of channel is also only 2 to 3 feet. These reaches of the channel do not readily support the use of pipe culverts because the available channel depth is not adequate for culvert placement without creating the likelihood that the structure would displace flow onto the adjoining roadway even during low-magnitude discharge events. Channel modifications to accommodate the design needs for the culvert system are not recommended because of the possibility of changes in flow direction which could act to the detriment of adjoining properties. The recommended approach for the driveway crossings at these locations is to use a concrete ford dip crossing. Specific criteria for determining where dip crossings should be used rather than a system of pipe culverts is provided in the following section of this report. Additionally, the initial step of the permitting process should determine if there are any other viable locations for achieving the needed access which would eliminate the need for a new culvert installation.

No estimate has been made to determine the probable number of future installations which will fall under the criteria outlined in this report. The likely case is that the number of problematic existing driveways will exceed the number of future installations, and this implies that there will be little to no relief from periodic flood damages during future flows. To at least partially remedy this situation, it is suggested that existing culvert systems which wash out during future floods be required to reconstruct the crossings to the standards adopted as a part of this study. This is the only method by which the on-going flood and erosion problems will be remedied over time.

The existing 36-inch CMP beneath Sabino Canyon Parkway at Turkey Run Road is almost completely blocked with sediment. Pima County road maintenance crews have been unable to remove these sediments, so the culvert remains completely unable to pass flow beneath the road. The deposition within the culvert occurs because it is hydraulically inadequate to convey the volume of runoff (and sediments) that are periodically delivered to it by Turkey Run Road Wash. Field observation of the existing culvert crossing led to the conclusion that this problem may not be resolved by installation of more or larger culverts. The available slope for the culvert is not steep enough to transport the sediments, and the hydraulic capacity of even a larger culvert such as a 60-inch diameter would not convey more than about the 2-year discharge.

The key feature to possible solutions to this problem will be the ability to easily remove sediment deposits from the culvert structure. Conventional methods have proven ineffective because of the large culvert length. The suggested approach proposes construction of a box culvert with a removable grate top to enable the access for sediment removal. The minimum dimensions of this culvert should provide about 30 square feet of cross section area to convey the 2- to 5-year flood discharge.

VII. CONCEPT CONSTRUCTION DRAWINGS

The concept plan presented herein offers a standardized approach to the driveway culvert installations along each of the study streams. Field investigations have determined that the designs presented herein will apply to most crossing locations within the project area. There are some locations where the recommended approach will not apply, and this circumstance necessitates that Pima County staff be involved in a field inspection to make the final determination on site suitability. A checklist which will enable the Pima County staff to make the final determination on the most appropriate design for the new culvert crossing structure is provided in Appendix H of this report. The following three subsections of the report describe the design characteristics for the standardized driveway crossings along Carter Canyon Wash, Turkey Run Road, and Sabino Canyon Parkway.

7.1 Carter Canyon Wash Design Recommendations

The average channel depth along Carter Canyon Wash is about 4 to 4.5 feet, and the bottomwidth ranges from 6 to 8 feet. The selected culvert structure for this average channel geometry is a single 64" x 43" corrugated metal arch pipe. The corregation should be 2-2/3" x 1/2". The H2O load capacity for this pipe is 32,000 lbs. per axle. This pipe culvert has a capacity of about 90 cfs at a headwater depth of 4 feet. The surface profile for the driveway crossing has a minimum depression depth of 6 inches. Larger depressions may be provided (up to 1.0 foot) where the natural channel depth exceeds 4.5 feet.

A typical cross section of the culvert and headwall face is provided on Figure 4 of this report. All standard Pima County requirements shall apply.

7.2 Turkey Run Road Wash Design Recommendations

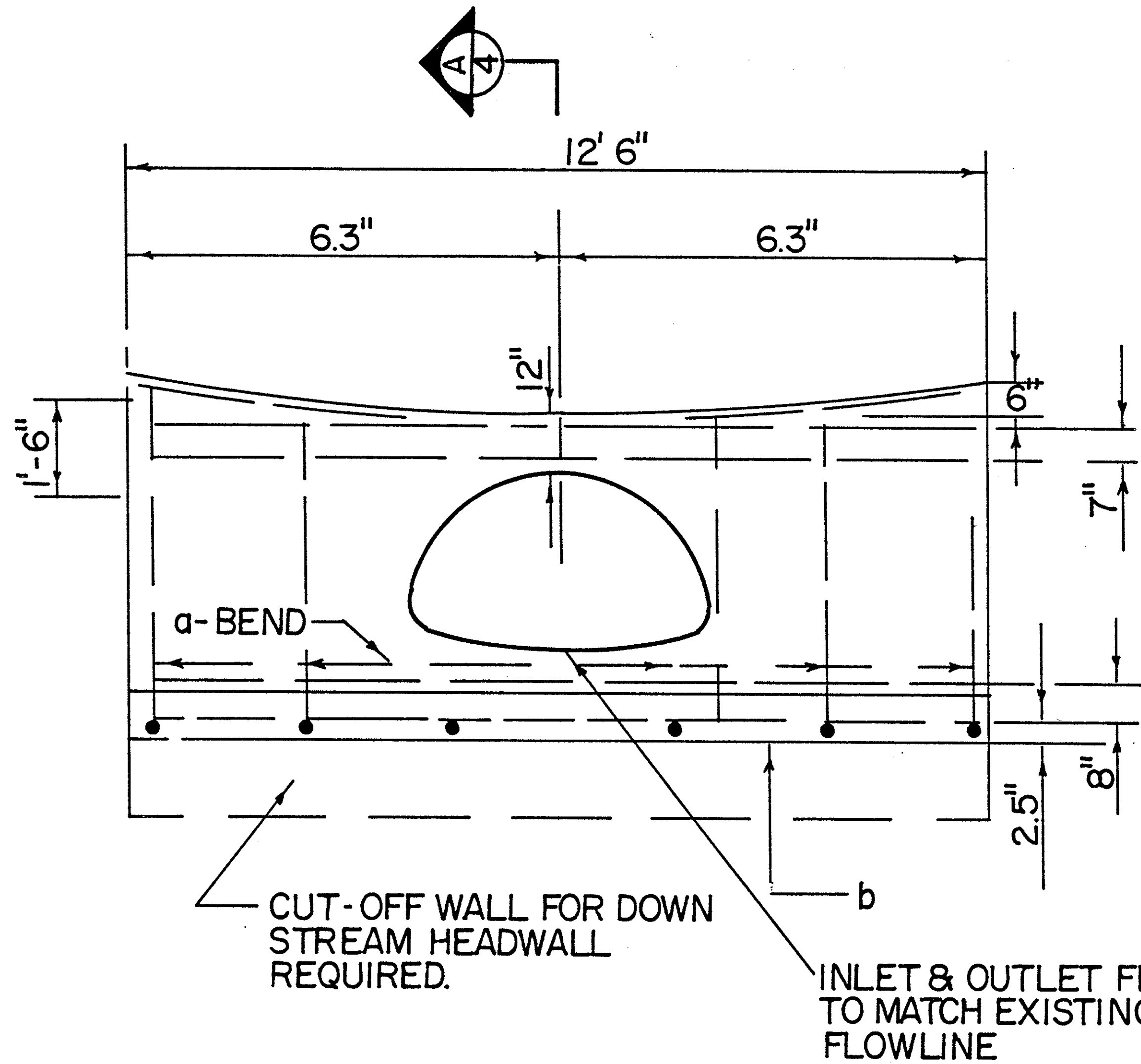
The average channel depth along Turkey Run Wash is about 5 to 6 feet, and the bottomwidth ranges from 6 to 10 feet. The selected culvert structure for this average channel geometry is one 71" x 47" corrugated metal arch pipe. The corregation should be 2-2/3" x 1/2". The H2O load capacity for this pipe is 32,000 lbs. per axle. This pipe culvert has a capacity of about 155 cfs at a headwater depth of 6 feet. The surface profile for the driveway crossing has a minimum depression depth of 1 foot. Larger depressions may be provided (up to 2.0 feet) where the natural channel depth exceeds 6.0 feet.

A typical cross section of the culvert and headwall face is provided on Figure 5 of this report. All standard Pima County specifications shall apply.

7.3 Sabino Creek Design Recommendations

The average channel depth along Sabino Creek varies from 3 to 7 feet, and the bottomwidth ranges from 8 to 10 feet. The selected culvert structure for this average channel geometry is one 71" x 47" corrugated metal arch pipe. The corregation should be 2-2/3" x 1/2". The H2O load capacity for this pipe is 32,000 lbs. per axle. This pipe culvert has a capacity of about 155 cfs at a headwater depth of 6 feet. The surface profile for the driveway crossing has a minimum depression depth of 1 foot. Larger depressions may be provided (up to 2.0 feet) where the natural channel depth exceeds 6 feet.

FIGURE 4 - CARTER CANYON WASH CULVERT AND HEADWALL CROSS-SECTION

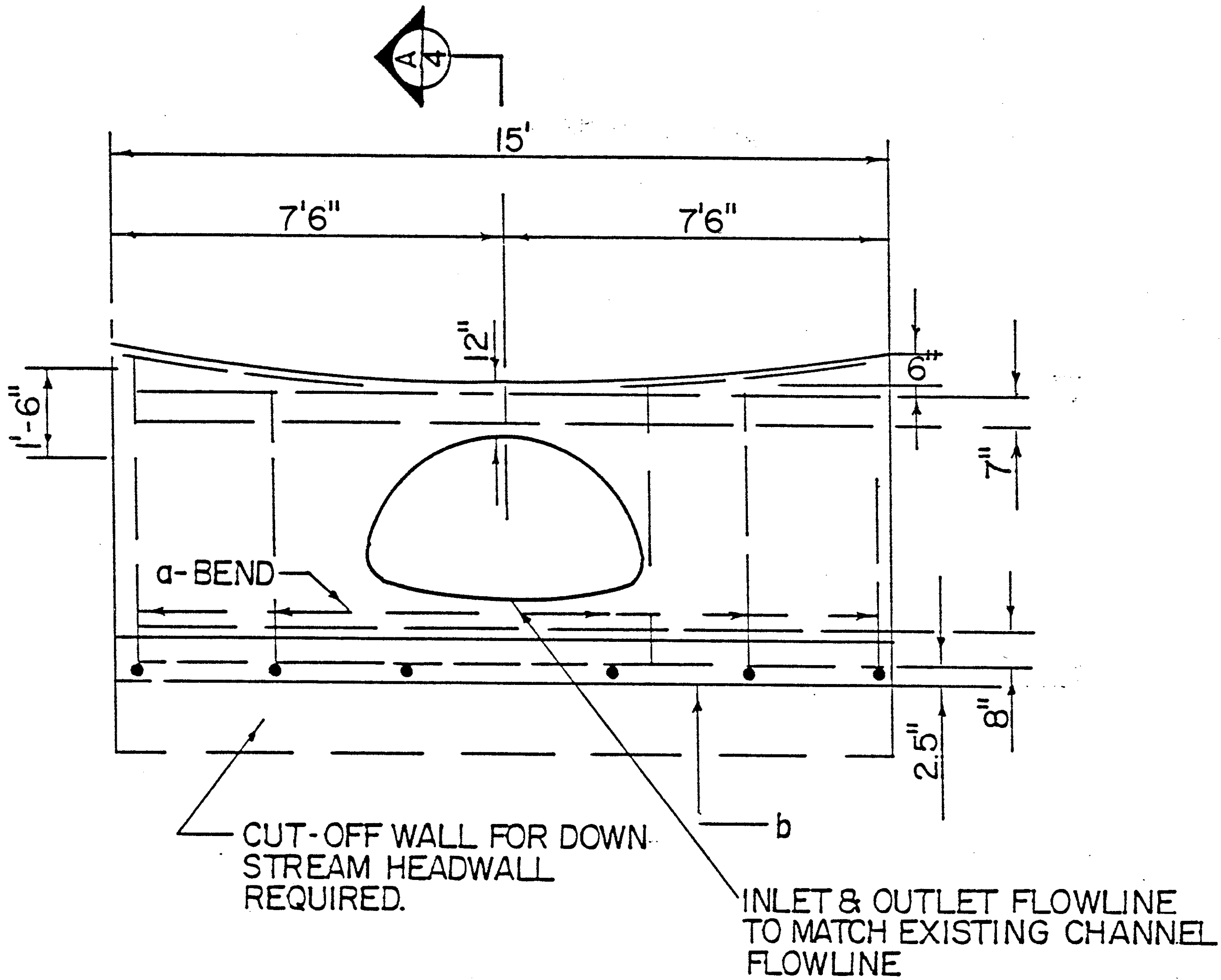


HEADWALL - CARTER CANYON WASH
1 - 64" x 43" ACMP

STRAIGHT HEADWALL SINGLE PIPE

- REFER TO SHEET 4 FOR SECTION THRU
- HEADWALL AND GENERAL NOTES
  REFER TO SHEET 4 FOR DIMENSION TABLE
- PIPE GAUGE SHALL BE 12 GAUGE WITH 2 2/3" x 1/2" CORREGATIONS, H20 LOAD = 32,000 LBS./AXLE
- CARTER CANYON WASH 1-64" x 43" ACMP

FIGURE 5 - TURKEY RUN RD. WASH CULVERT AND HEADWALL CROSS-SECTION



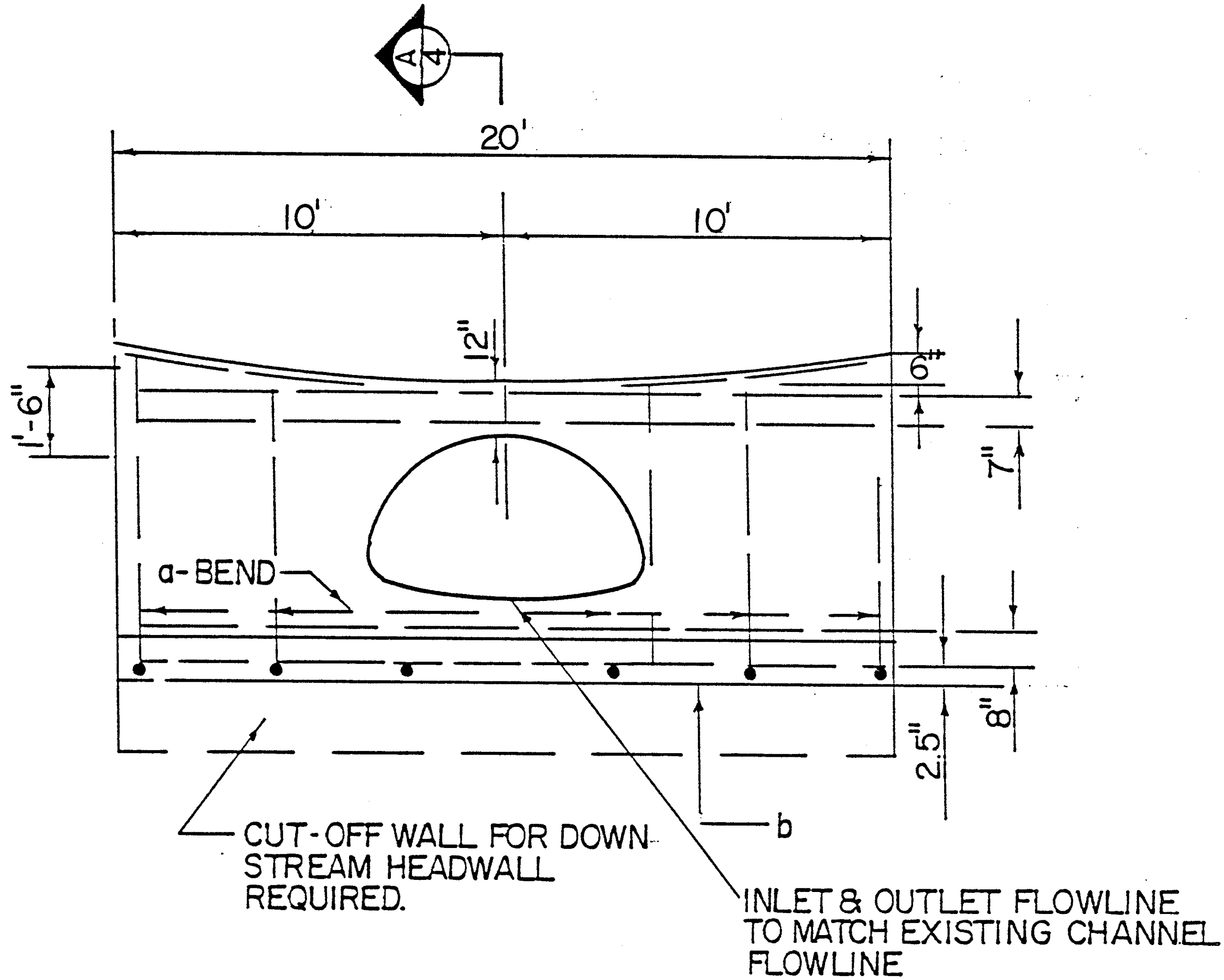
HEADWALL- TURKEY RUN ROAD WASH
1 - 71" x 47" ACMP

STRAIGHT HEADWALL SINGLE PIPE

- REFER TO SHEET 4 FOR SECTION THRU
- HEADWALL AND GENERAL NOTES REFER TO SHEET 4 FOR DIMENSION TABLE
- PIPE GAUGE SHALL BE 12 GAUGE WITH $2\frac{2}{3}$" x $\frac{1}{2}$" CORREGATIONS, H20 LOAD = 32,000 LBS./AXLE

A typical cross section of the culvert and headwall face is provided on Figure 6 of this report. All standard Pima County specifications shall apply.

FIGURE 6 - SABINO CREEK CULVERT AND HEADWALL CROSS-SECTION



HEADWALL- SABINO CREEK
1 - 71" x 47" ACMP

STRAIGHT HEADWALL SINGLE PIPE

- REFER TO SHEET 4 FOR SECTION THRU
- HEADWALL AND GENERAL NOTES
  REFER TO SHEET 4 FOR DIMENSION TABLE
- PIPE GAUGE SHALL BE 12 GAUGE WITH 2 2/3 " x 1/2" CORREGATIONS, H20 LOAD = 32,000 LBS/AXLE

APPENDIX A

SCOPE OF WORK OUTLINE

SCOPE OF WORK OUTLINE

Task 1 - Hydrologic Analysis

Discharge computations will be conducted using the Pima County method to determine the 10-year and 100-year return period discharges for those watercourses impacting Carter Canyon Road, Turkey Run Road, and Sabino Canyon Park. Points of concentration will be defined as necessary to identify reaches with similar hydrologic characteristics.

Task 2 - Reach Definition

Hydrologic analyses and field investigations will aid the definition of uniform reaches. The uniform reaches will be subdivided based upon the hydrologic characteristics, channel geometrics, and flow hydraulics. Mannings ratings will be prepared for each of the uniform reaches to define flow conditions such as velocity and depth for the 10- and 100-year return period events.

Task 3 - Interviews

Interviews will be conducted with some of the area residents and the Operations Department personnel to determine the scope of drainage problems which have occurred in the past. The area residents which will be targeted for interviews will be those which currently have inadequate drainage structures, those who have experienced damages due to culvert structures on adjoining properties, and those who have limited use of the property due to inadequate access. Written notice will be transmitted to up to 30 of the area residents requesting information and scheduling either a site visit or telephone interview.

Task 4 - Development of General Design Criteria

The data base developed in Tasks 1 through 3 will be used to develop general design criteria for the private drainageway access across the watercourses. One element of this effort will be to work with Pima County in determining what the needed design criteria should be. Determination of the most appropriate design criteria for access driveways has not been previously established by Pima County. Therefore, consideration must be given to determining the acceptable minimum design criteria which will be needed in order to prevent future damages to adjoining property and roads. The approaches which will be evaluated will include dip sections or culvert crossings and associated structural stabilization improvements such as cut-off walls and cross slopes needed to convey anticipated sediment loads. Where culvert structures are recommended, the design will include recommendations for size, type, and stabilization measures such as headwalls and cut-off walls. These typical criteria will be provided for each of the uniform reaches identified in the hydrologic analysis.

The final element of this task will include preparation of concept-level construction drawings and specifications. The criteria provided will include compaction, types of backfill material, concrete specifications, reinforcement, and minimum culvert separation distances.

Task 5 - Inventory of Existing Private Access Improvements

An inventory of existing private access improvements across the drainageways will be provided. This inventory will include the property address, type of structure, identified flood-related damages due to previous flows, interview information from the affected property owners, a determination of whether this structure has been permitted by Pima County, and a photograph of the access provided in the report document.

Task 6 - Report and Deliverables

A report will be prepared summarizing the results of the hydrologic and hydraulic analyses, field investigations, interviews, and existing structure inventory. In addition, this report will contain the concept construction drawing plan for the culvert crossing structures for each uniform reach. Pima County will be provided with 3 copies of the draft report and 10 copies of the final report as amended by PCDOT & FCD staff. A floppy disk will be provided including all of the pertinent input/output file information used in developing the report.

APPENDIX B

HYDROLOGIC DESIGN DATA SHEETS

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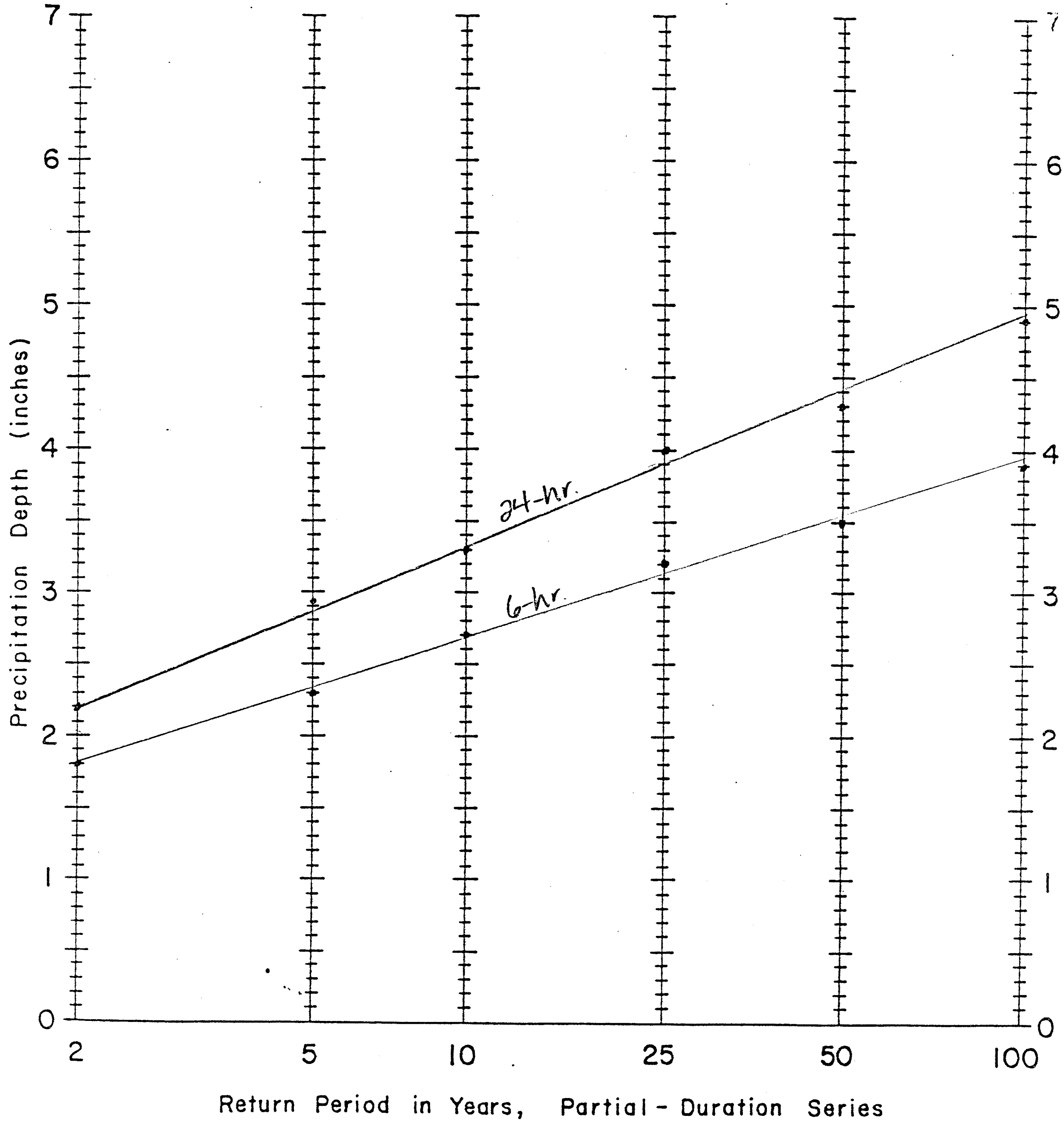
Mt. Lemmon - Summerhaven

Latitude ____________ Longitude ____________

110°45' 32° 27'



Precipitation Depth Versus Return Period For Partial - Duration Series

Mt Lemmon - Summerhaven

Return Period (Years)	Precipitation Values (inches)			
	6 Hour Duration		24 Hour Duration	
	MAP VALUE	CORRECTED VALUE	MAP VALUE	CORRECTED VALUE
2	1.8	1.8	2.2	2.2
5	2.3	2.33	2.92	2.87
10	2.7	2.69	3.3	3.32
25	3.2	3.13	4.0	3.90
50	3.5	3.55	4.3	4.40
100	3.9	3.95	4.9	4.95

X_1 = 2-yr. 6-hr.
X_2 = 2-yr. 24-hr.

2-yr. 1-hr.

$$Y_2 = -0.011 + 0.942(X_1^2/X_2)$$
$$Y_2 = -0.11 + 0.942\left[(1.8)^2/2.2\right]$$
$$Y_2 = 1.28 \text{ in/hr.}$$

X_3 = 100-yr. 6-hr.
X_4 = 100-yr 24-hr.

100-yr. 1-hr.

$$Y_{100} = 0.494 + 0.755(X_3^2/X_4)$$
$$Y_{100} = 0.494 + 0.755\left[(3.95)^2/4.95\right]$$
$$Y_{100} = 2.87 \text{ in/hr.}$$

PROJECT NAME AND LOCATION: MT. LEMMON - SUMMERHAVEN

DRAINAGE CONCENTRATION POINT: 1

WATERSHED AREA (A): 234.20 acres

LENGTH OF WATERCOURSE (Lc): 5600. ft

LENGTH TO CENTER OF GRAVITY (Lca): 2800. ft

INCREMENTAL CHANGE IN LENGTH (Li) - ft	INCREMENTAL CHANGE IN ELEV (Hi) - ft
700.	120.0
4900.	840.0

MEAN SLOPE (Sc): .1714 ft BASIN FACTOR (Nb): .0600

WATERSHED TYPE(S): MOUNTAIN

RAINFALL VALUES

	EVENT					
	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
P 1	1.38	1.74	1.99	2.30	2.59	2.87
P 2	1.52	1.94	2.22	2.59	2.92	3.24
P 3	1.62	2.08	2.38	2.78	3.13	3.49
P 6	1.80	2.33	2.68	3.13	3.54	3.95
P24	2.20	2.87	3.33	3.90	4.43	4.95

SOIL GROUPS

100. % D, CN= 78, COVER TYPE= PONDEROSA PINE, COVER DENSITY= 50 %

IMPERVIOUS COVER= 5. %

RAINFALL/RUNOFF AND PEAK DISCHARGE DATA

	EVENT					
	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
RUNOFF SUPPLY RATE (q/i):	.151	.261	.327	.400	.457	.505
Tc (FUNCTION OF i) :	44.70	35.96	32.86	30.29	28.72	27.60
SOLUTION OF Tc (MINUTES):	34	23	19	16	14	13
RAINFL INT. @ Tc (IN/HR):	2.009	3.208	4.058	5.116	6.141	7.012
RUNOFF RATE @ Tc (IN/HR):	.304	.836	1.325	2.048	2.809	3.544
PEAK DISCHARGE (CFS) :	72.	197.	313.	483.	663.	837.

PROJECT NAME AND LOCATION: MT LEMMON - SUMMERHAVEN

DRAINAGE CONCENTRATION POINT: 2

WATERSHED AREA (A): 436.20 acres

LENGTH OF WATERCOURSE (Lc): 10000. ft

LENGTH TO CENTER OF GRAVITY (Lca): 4200. ft

INCREMENTAL CHANGE IN LENGTH (Li) - ft	INCREMENTAL CHANGE IN ELEV (Hi) - ft
3100.	720.0
6900.	640.0

MEAN SLOPE (Sc): .1182 ft BASIN FACTOR (Nb): .0600

WATERSHED TYPE(S): MOUNTAIN

RAINFALL VALUES

	EVENT					
	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
P 1	1.38	1.74	1.99	2.30	2.59	2.87
P 2	1.52	1.94	2.22	2.59	2.92	3.24
P 3	1.62	2.08	2.38	2.78	3.13	3.49
P 6	1.80	2.33	2.68	3.13	3.54	3.95
P24	2.20	2.87	3.33	3.90	4.43	4.95

SOIL GROUPS

100. % D, CN= 78, COVER TYPE= PONDEROSA PINE, COVER DENSITY= 50 %

IMPERVIOUS COVER= 5. %

RAINFALL/RUNOFF AND PEAK DISCHARGE DATA

	EVENT					
	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
RUNOFF SUPPLY RATE (q/i):	.151	.261	.327	.400	.457	.505
Tc (FUNCTION OF i) :	69.71	56.08	51.25	47.24	44.79	43.04
SOLUTION OF Tc (MINUTES):	62	41	33	28	24	22
RAINFL INT. @ Tc (IN/HR):	1.337	2.249	2.944	3.779	4.664	5.431
RUNOFF RATE @ Tc (IN/HR):	.202	.586	.962	1.513	2.133	2.745
PEAK DISCHARGE (CFS) :	89.	258.	423.	665.	938.	1207.

PROJECT NAME AND LOCATION: MT.LEMMON - SUMMERHAVEN

DRAINAGE CONCENTRATION POINT: 3

WATERSHED AREA (A): 504.20 acres

LENGTH OF WATERCOURSE (Lc): 10960. ft

LENGTH TO CENTER OF GRAVITY (Lca): 4700. ft

INCREMENTAL CHANGE IN LENGTH (Li) - ft	INCREMENTAL CHANGE IN ELEV (Hi) - ft
3100.	720.0
7860.	680.0

MEAN SLOPE (Sc): .1093 ft BASIN FACTOR (Nb): .0600

WATERSHED TYPE(S): MOUNTAIN

RAINFALL VALUES

	EVENT 2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
P 1	1.38	1.74	1.99	2.30	2.59	2.87
P 2	1.52	1.94	2.22	2.59	2.92	3.24
P 3	1.62	2.08	2.38	2.78	3.13	3.49
P 6	1.80	2.33	2.68	3.13	3.54	3.95
P24	2.20	2.87	3.33	3.90	4.43	4.95

SOIL GROUPS

100. % D, CN= 78, COVER TYPE= PONDEROSA PINE, COVER DENSITY= 50 %

IMPERVIOUS COVER= 5. %

RAINFALL/RUNOFF AND PEAK DISCHARGE DATA

	EVENT 2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
RUNOFF SUPPLY RATE (q/i):	.151	.261	.327	.400	.457	.505
Tc (FUNCTION OF i) :	76.48	61.52	56.22	51.82	49.14	47.21
SOLUTION OF Tc (MINUTES):	72	46	38	31	28	25
RAINFL INT. @ Tc (IN/HR):	1.171	2.092	2.705	3.549	4.250	5.029
RUNOFF RATE @ Tc (IN/HR):	.177	.545	.884	1.421	1.943	2.542
PEAK DISCHARGE (CFS) :	90.	277.	449.	722.	988.	1292.

PROJECT NAME AND LOCATION: MT. LEMMON - SUMMERHAVEN

DRAINAGE CONCENTRATION POINT: 4

WATERSHED AREA (A): 233.70 acres

LENGTH OF WATERCOURSE (Lc): 6740. ft

LENGTH TO CENTER OF GRAVITY (Lca): 2920. ft

INCREMENTAL CHANGE IN LENGTH (Li) - ft	INCREMENTAL CHANGE IN ELEV (Hi) - ft
6740.	1360.0

MEAN SLOPE (Sc): .2018 ft   BASIN FACTOR (Nb): .0600

WATERSHED TYPE(S): MOUNTAIN

RAINFALL VALUES

	EVENT					
	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
P 1	1.38	1.74	1.99	2.30	2.59	2.87
P 2	1.52	1.94	2.22	2.59	2.92	3.24
P 3	1.62	2.08	2.38	2.78	3.13	3.49
P 6	1.80	2.33	2.68	3.13	3.54	3.95
P24	2.20	2.87	3.33	3.90	4.43	4.95

SOIL GROUPS

100. % D, CN= 78, COVER TYPE= PONDEROSA PINE, COVER DENSITY= 50 %

IMPERVIOUS COVER= 5. %

RAINFALL/RUNOFF AND PEAK DISCHARGE DATA

	EVENT					
	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
RUNOFF SUPPLY RATE (q/i):	.151	.261	.327	.400	.457	.505
Tc (FUNCTION OF i) :	44.84	36.07	32.96	30.38	28.81	27.68
SOLUTION OF Tc (MINUTES):	34	23	19	16	14	13
RAINFL INT. @ Tc (IN/HR):	2.009	3.208	4.058	5.116	6.141	7.012
RUNOFF RATE @ Tc (IN/HR):	.304	.836	1.325	2.048	2.809	3.544
PEAK DISCHARGE (CFS) :	72.	197.	312.	482.	662.	835.

PROJECT NAME AND LOCATION: MT.LEMMON - SUMMERHAVEN

DRAINAGE CONCENTRATION POINT: 5

WATERSHED AREA (A): 815.00 acres

LENGTH OF WATERCOURSE (Lc): 12200. ft

LENGTH TO CENTER OF GRAVITY (Lca): 4800. ft

INCREMENTAL CHANGE IN LENGTH (Li) - ft	INCREMENTAL CHANGE IN ELEV (Hi) - ft
3100.	720.0
9100.	760.0

MEAN SLOPE (Sc): .1035 ft BASIN FACTOR (Nb): .0600

WATERSHED TYPE(S): MOUNTAIN

RAINFALL VALUES

	EVENT					
	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
P 1	1.38	1.74	1.99	2.30	2.59	2.87
P 2	1.52	1.94	2.22	2.59	2.92	3.24
P 3	1.62	2.08	2.38	2.78	3.13	3.49
P 6	1.80	2.33	2.68	3.13	3.54	3.95
P24	2.20	2.87	3.33	3.90	4.43	4.95

SOIL GROUPS

100. % D, CN= 78, COVER TYPE= PONDEROSA PINE, COVER DENSITY= 50 %

IMPERVIOUS COVER= 5. %

RAINFALL/RUNOFF AND PEAK DISCHARGE DATA

	EVENT					
	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
RUNOFF SUPPLY RATE (q/i):	.151	.261	.327	.400	.457	.505
Tc (FUNCTION OF i) :	81.22	65.34	59.70	55.04	52.18	50.14
SOLUTION OF Tc (MINUTES):	79	50	41	34	30	27
RAINFL INT. @ Tc (IN/HR):	1.080	1.970	2.566	3.364	4.094	4.828
RUNOFF RATE @ Tc (IN/HR):	.163	.514	.838	1.347	1.872	2.440
PEAK DISCHARGE (CFS) :	134.	422.	689.	1106.	1538.	2005.

PROJECT NAME AND LOCATION: MT. LEMMON - SUMMERHAVEN

DRAINAGE CONCENTRATION POINT: 6

WATERSHED AREA (A): 879.30 acres

LENGTH OF WATERCOURSE (Lc): 13360. ft

LENGTH TO CENTER OF GRAVITY (Lca): 6140. ft

INCREMENTAL CHANGE IN LENGTH (Li) - ft	INCREMENTAL CHANGE IN ELEV (Hi) - ft
3100.	720.0
10260.	800.0

MEAN SLOPE (Sc): .0957 ft BASIN FACTOR (Nb): .0600

WATERSHED TYPE(S): MOUNTAIN

RAINFALL VALUES

	EVENT					
	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
P 1	1.38	1.74	1.99	2.30	2.59	2.87
P 2	1.52	1.94	2.22	2.59	2.92	3.24
P 3	1.62	2.08	2.38	2.78	3.13	3.49
P 6	1.80	2.33	2.68	3.13	3.54	3.95
P24	2.20	2.87	3.33	3.90	4.43	4.95

SOIL GROUPS

100. % D, CN= 78, COVER TYPE= PONDEROSA PINE, COVER DENSITY= 50 %

IMPERVIOUS COVER= 5. %

RAINFALL/RUNOFF AND PEAK DISCHARGE DATA

	EVENT					
	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
RUNOFF SUPPLY RATE (q/i):	.151	.261	.327	.400	.457	.505
Tc (FUNCTION OF i) :	92.70	74.58	68.15	62.82	59.56	57.23
SOLUTION OF Tc (MINUTES):	96	60	49	41	35	32
RAINFL INT. @ Tc (IN/HR):	.914	1.744	2.288	2.973	3.705	4.339
RUNOFF RATE @ Tc (IN/HR):	.138	.455	.747	1.190	1.695	2.193
PEAK DISCHARGE (CFS) :	123.	403.	662.	1055.	1502.	1944.

APPENDIX C

HEC-1 MODEL INPUT/OUTPUT

```
1                                         ****
                  FLOOD HYDROGRAPH PACKAGE HEC-1 (IBM XT 512K VERSION) -FEB 1,1985
      U.S. ARMY CORPS OF ENGINEERS, THE HYDROLOGIC ENGINEERING CENTER, 609 SECOND STREET, DAVIS, CA. 95616
                                          ****


     THIS HEC-1 VERSION CONTAINS ALL OPTIONS EXCEPT ECONOMICS, AND THE NUMBER OF PLANS ARE REDUCED TO 3


1                                     HEC-1 INPUT                                            PAGE  1

  LINE        ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

    1         ID   MT. LEMMON - SUMMERHAVEN
    2         ID   24-HR. STORM   50% COVER DENSITY
    3         ID   FILE : LEMMON2.H1I
              *DIAGRAM
    4         IT      15       0       0     145
    5         IO       4
    6         IN      15       0       0
    7         JR    PREC     1.0   1.305   1.509   1.773    2.00    2.25

    8         KK       1
    9         KM        BASIN #1
   10         BA   0.366
   11         PB     2.2
              *
              *        24 HOUR   SCS TYPE II RAINFALL DISTRIBUTION
              *
   12         PC    .000    .002    .005    .008    .011    .014    .017    .020    .023    .026
   13         PC    .029    .032    .035    .038    .041    .044    .048    .052    .056    .060
   14         PC    .064    .068    .072    .076    .080    .085    .090    .095    .100    .105
   15         PC    .110    .115    .120    .126    .133    .140    .147    .155    .163    .172
   16         PC    .181    .191    .203    .218    .236    .257    .283    .387    .663    .707
   17         PC    .735    .758    .776    .791    .804    .815    .825    .834    .842    .849
   18         PC    .856    .863    .869    .875    .881    .887    .893    .898    .903    .908
   19         PC    .913    .918    .922    .926    .930    .934    .938    .942    .946    .950
   20         PC    .953    .956    .959    .962    .965    .968    .971    .974    .977    .980
   21         PC    .983    .986    .989    .992    .995    .998    1.00
   22         LS       0      78
   23         UK    2000    0.38   0.070     100
   24         RK    5600    0.17   0.060     100    TRAP       4     1.3

   25         KK       2
   26         KM        BASIN #2
   27         BA   0.316
   28         LS       0      78
   29         UK    1550     .36   0.070     100
   30         RK    3900     .10   0.060     100    TRAP     4.5     1.5     YES

   31         KK       3
   32         KM        BASIN #3
   33         BA   0.106
   34         LS       0      78
   35         UK    1600     .38   0.070     100
   36         RK     900   0.060   0.060     100    TRAP       8       1     YES

   37         KK       5
```

046

```
         39            BA   0.121
         40            LS      0      78
         41            UK   1800     .33   0.070     100
         42            RK   1300    .060   0.060     100    TRAP      7      1     YES
```



```
       LINE            ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10


         43            KK      4
         44            KM        BASIN #4
         45            BA   0.365
         46            LS      0      78
         47            UK   1600     .40   0.070     100
         48            RK   6000   0.130   0.060     100    TRAP    7.5      1

         49            KK
         50            KM        COMBINING BASINS 4 & 5
         51            HC      2

         52            KK      6
         53            KM        BASIN #6
         54            BA   0.101
         55            LS      0      78
         56            UK   2100     .32   0.070     100
         57            RK   1200    0.06   0.060     100    TRAP      8      2     YES
         58            ZZ
1
              SCHEMATIC DIAGRAM OF STREAM NETWORK
 INPUT
  LINE     (V) ROUTING         (--->) DIVERSION OR PUMP FLOW

   NO.     (.) CONNECTOR       (<---) RETURN OF DIVERTED OR PUMPED FLOW

    8          1
               V
               V
   25          2 ***
               V
               V
   31          3 ***
               V
               V
   37          5 ***
               .
               .
   43          .      4
               .      .
               .      .
   49          ........
               V
               V
   52          6 ***

 (***) RUNOFF ALSO COMPUTED AT THIS LOCATION
1                                         ****
1

             PEAK FLOW AND STAGE (END-OF-PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
                        FLOWS IN CUBIC FEET PER SECOND,  AREA IN SQUARE MILES
                                   TIME TO PEAK IN HOURS


                                                  RATIOS APPLIED TO PRECIPITATION
 OPERATION       STATION     AREA    PLAN       RATIO 1  RATIO 2  RATIO 3  RATIO 4  RATIO 5  RATIO 6
```

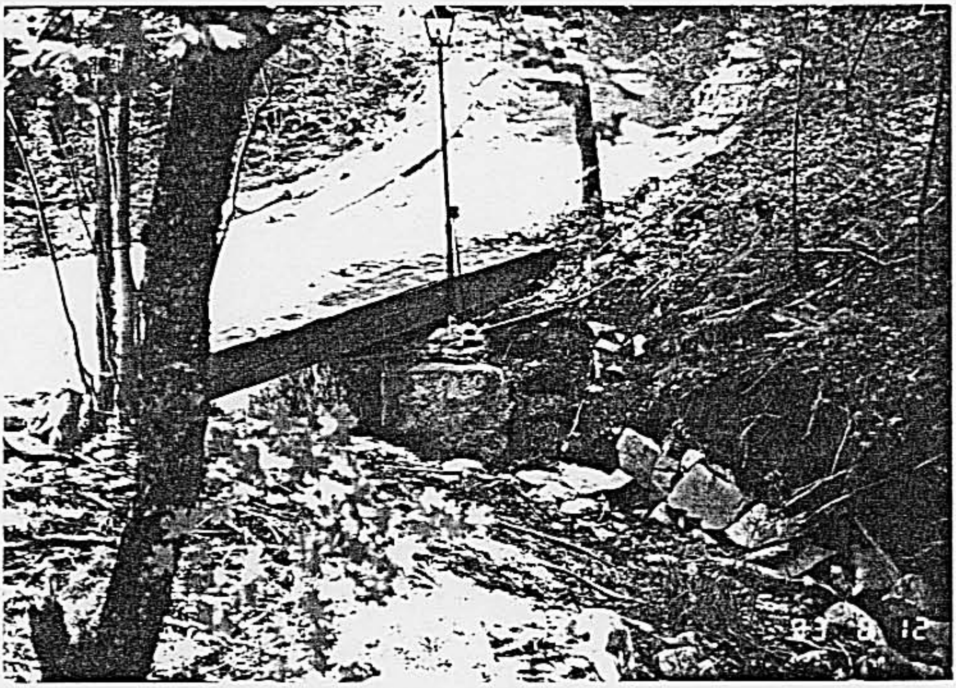
PEAK FLOW AND STAGE (END-OF-PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
FLOWS IN CUBIC FEET PER SECOND, AREA IN SQUARE MILES
TIME TO PEAK IN HOURS

OPERATION	STATION	AREA	PLAN		RATIO 1 1.00	RATIO 2 1.30	RATIO 3 1.51	RATIO 4 1.77	RATIO 5 2.00	RATIO 6 2.25
					RATIOS APPLIED TO PRECIPITATION					
HYDROGRAPH AT	1	.37	1	FLOW	101.	184.	277.	411.	522.	649.
				TIME	12.50	12.25	12.25	12.25	12.25	12.25
HYDROGRAPH AT	2	.68	1	FLOW	168.	304.	470.	694.	889.	1114.
				TIME	12.50	12.25	12.25	12.25	12.25	12.25
HYDROGRAPH AT	3	.79	1	FLOW	190.	342.	532.	786.	1010.	1267.
				TIME	12.50	12.25	12.25	12.25	12.25	12.25
HYDROGRAPH AT	5	.91	1	FLOW	214.	394.	580.	869.	1122.	1413.
				TIME	12.50	12.50	12.25	12.25	12.25	12.25
HYDROGRAPH AT	4	.37	1	FLOW	95.	197.	295.	419.	531.	658.
				TIME	12.50	12.25	12.25	12.25	12.25	12.25
2 COMBINED AT	#5	1.27	1	FLOW	309.	566.	876.	1288.	1653.	2071.
				TIME	12.50	12.25	12.25	12.25	12.25	12.25
HYDROGRAPH AT	6	1.38	1	FLOW	326.	598.	879.	1314.	1697.	2137.
				TIME	12.50	12.50	12.25	12.25	12.25	12.25

*** NORMAL END OF HEC-1 ***

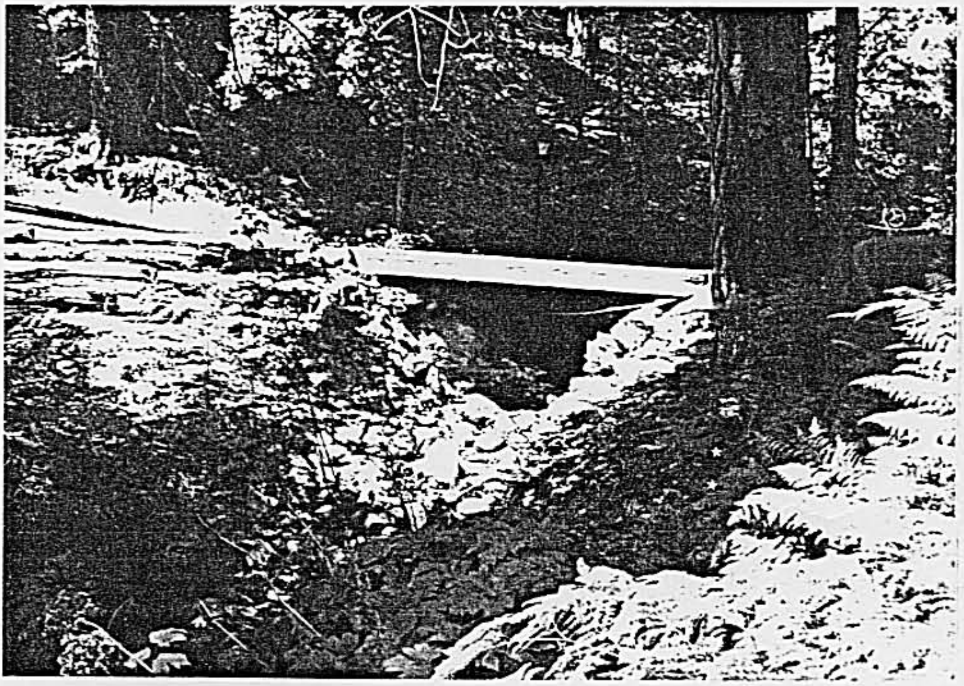
APPENDIX D

CULVERT PHOTOGRAPHS

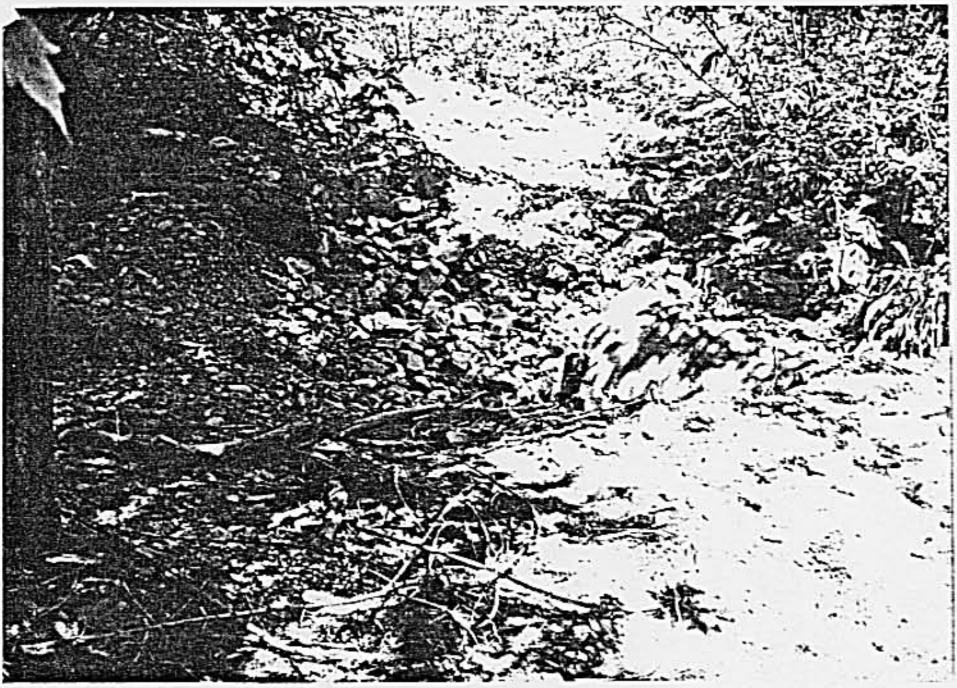


Upstream side of crossing at 11000 Carter Canyon Road.

NO PERMIT FOUND



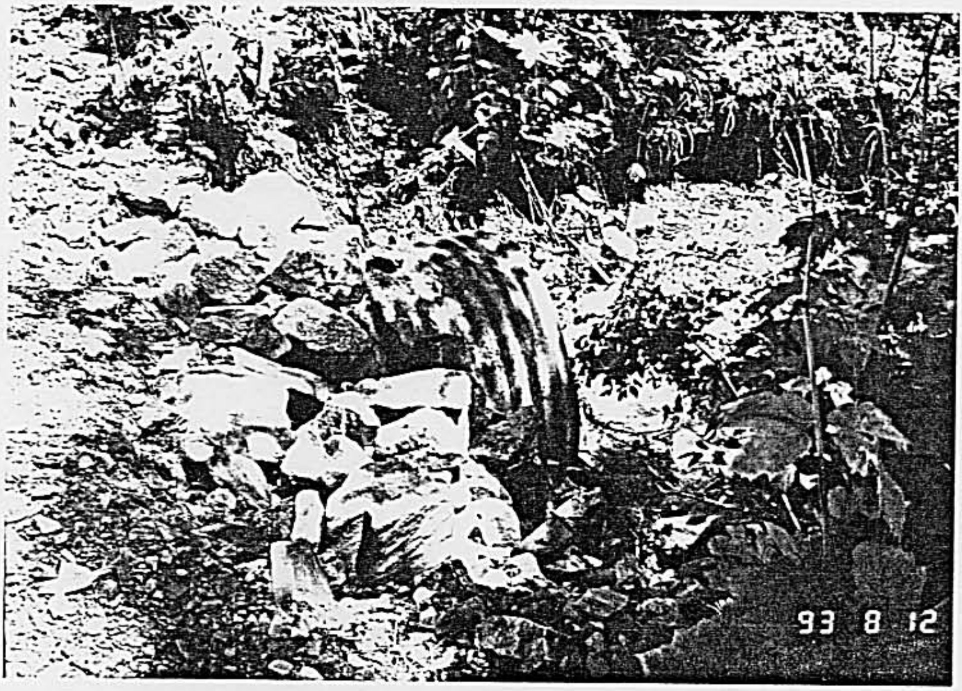
Downstream side of crossing at 11000 Carter Canyon Road.



Upstream side of crossing at 11113 Carter Canyon Road.

NO PERMIT FOUND

#2: 1-36" CMP Q_{CAP} = 35 cfs



Downstream side of crossing at 11113 Carter Canyon Road.

Downstream of crossing at 11113 Carter Canyon Road.

NO PERMIT FOUND

Upstream of washed out driveway at 11119 Carter Canyon Rd.

PERMIT ON FILE WITH PIMA COUNTY

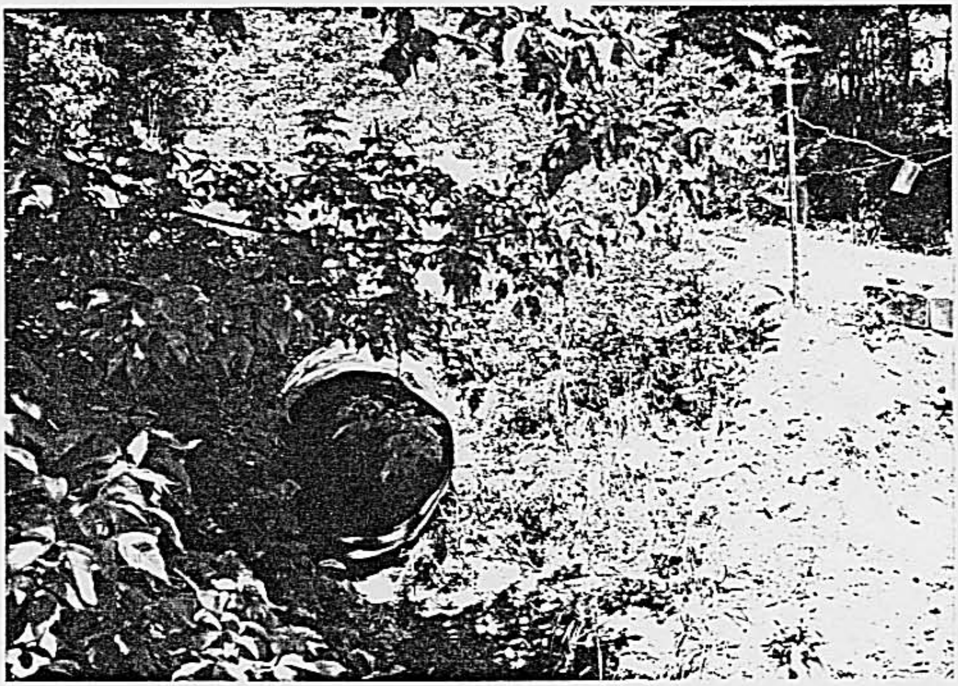
#3: 1-36" CMP

Lots 25/26

Upstream side of crossing at lots 25 and 26, Block 30, along Carter Canyon Road

NO PERMIT FOUND

#4: 1-36" CMP Q_{cap} = 35 cfs



Downstream side of crossing at lots 25 and 26, Block 30, along Carter Canyon Road

Upstream side of crossing at 11149 Carter Canyon Road.

NO PERMIT FOUND

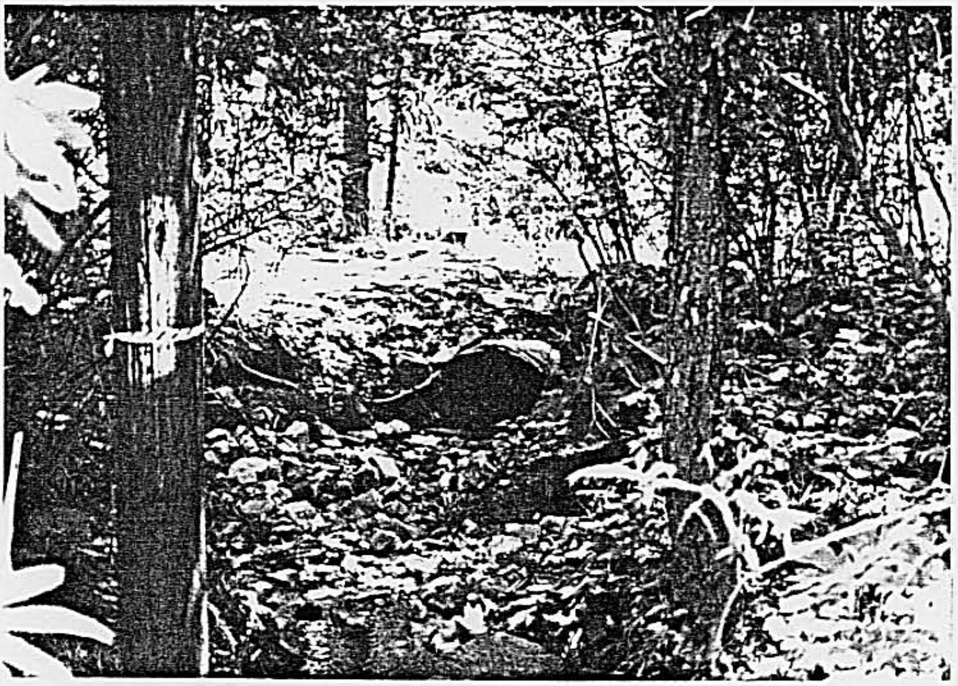
#5: 2-30" CMPs Q_{CAP} = 90 cfs

Downstream side of crossing at 11149 Carter Canyon Road.

Downstream side of crossing at 11179 Carter Canyon Road.
(driveway partially washed out)

NO PERMIT FOUND

#6: 1-36" CMP Q_{CAP} = 35 cfs



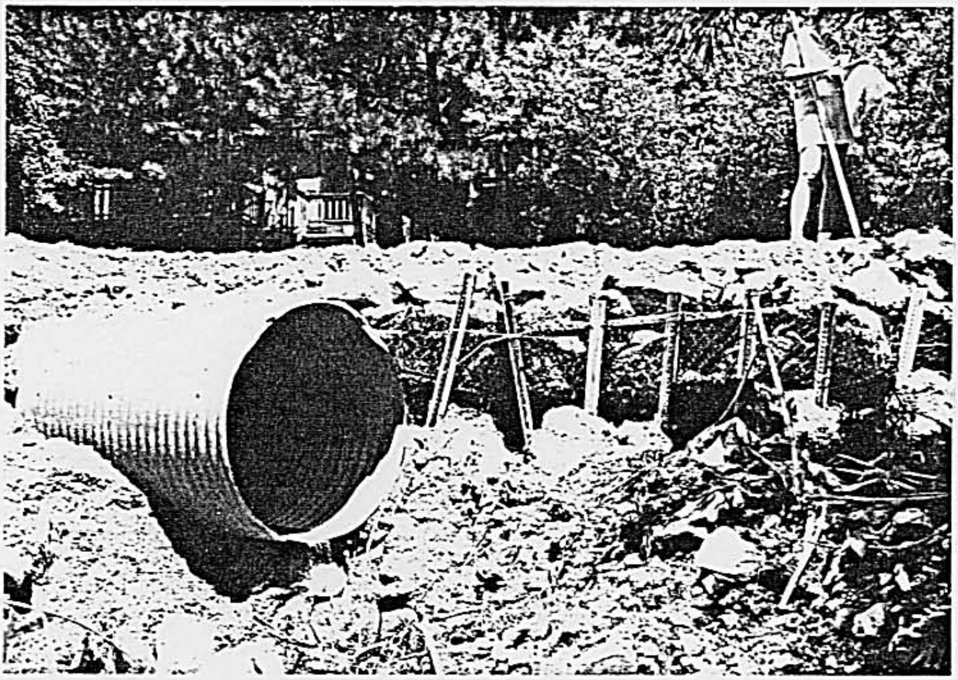
Upstream side of crossing just east of 11179 Carter Canyon Road.

#7: 1-36" CMP Q_{CAP} = 35 cfs

Upstream of crossing from the north to the south side of Carter Canyon Road (located at 11229 Carter Canyon Road).

#8: 1-42" CMP Q_{CAP} = 86 cfs

NO PERMIT FOUND



Downstream of crossing from the north to the south side of Carter Canyon Road (located at 11229 Carter Canyon Road).

Upstream side of crossing at 10950 Turkey Run Road.

NO PERMIT FOUND

#9: 1-36" CMP, 1-24" CMP Q_{CAP} = 119 cfs

Downstream side of crossing at 10950 Turkey Run Road.

NO PERMIT FOUND

Upstream side of crossing at Guthrie Road and Turkey Run Road.

NO PERMIT FOUND

#10: 1-36" CMP $Q_{CAP} = 35$ cfs

Upstream side of crossing at 11090 Turkey Run Road.

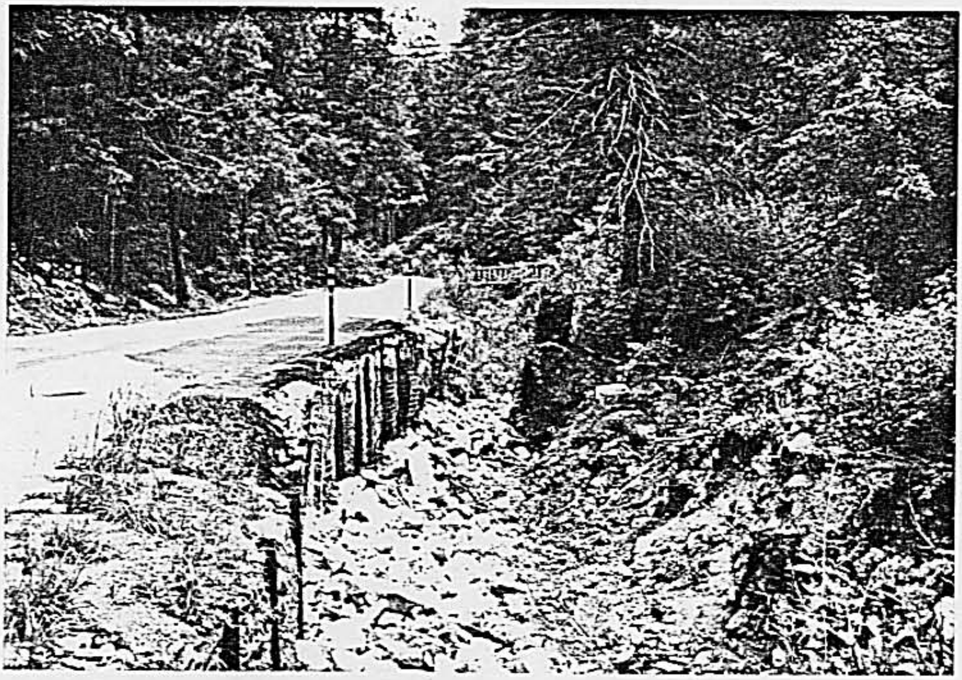
PERMIT ON FILE WITH PIMA COUNTY

#11: 2-42" CMPs $Q_{CAP} = 212$ cfs

Downstream of crossing at 11122 Turkey Run Road.

NO PERMIT FOUND

#12: 2-42"H x 60"W CMPs Q_{CAP} = 224 cfs



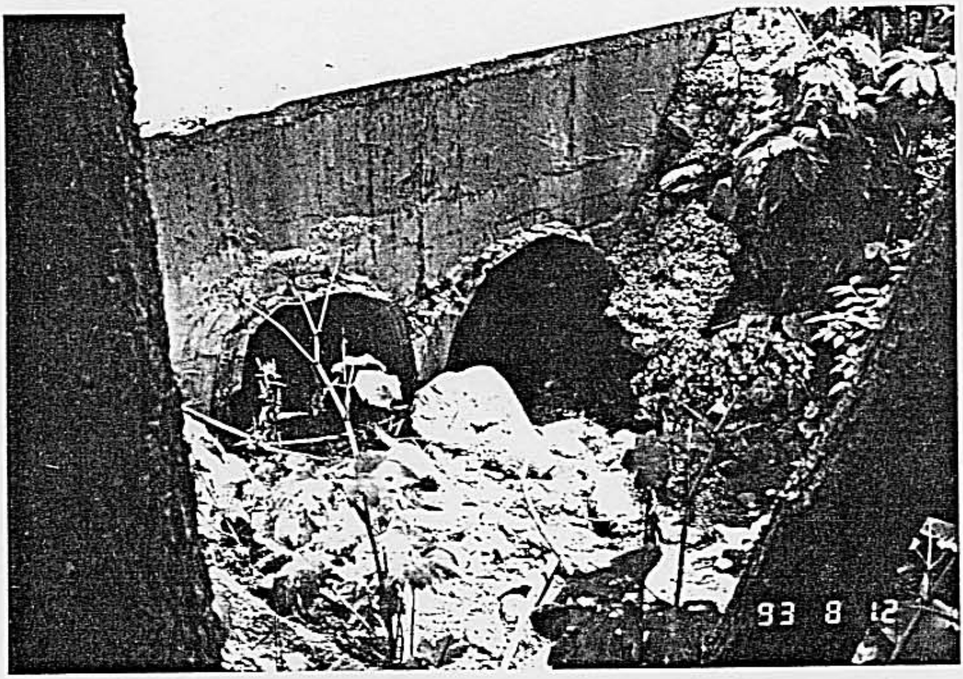
Looking downstream from crossing at 11122 Turkey Run Road.

Pipes coming from Turkey Run Road and ? into Sabino Canyon Park

NO PERMIT FOUND

From Turkey Run Road:

1-36" CMP Q_{cap} = 35 cfs



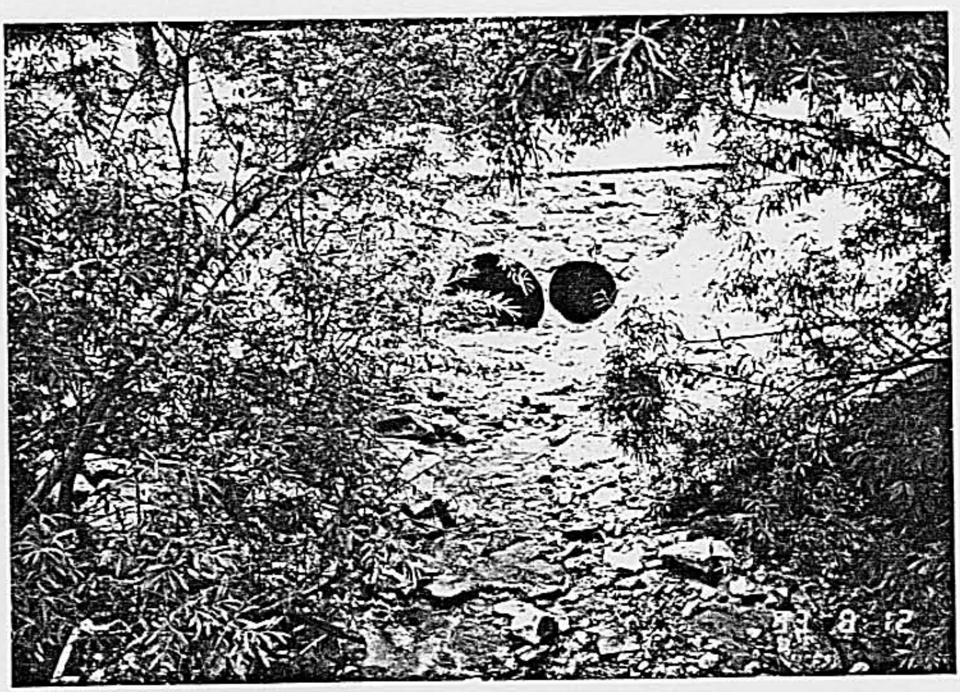
Upstream of 1st crossing at Kimball Springs along Sabino Canyon Park

NO PERMIT FOUND

2-48" CMPs Q_{cap} = 276 cfs

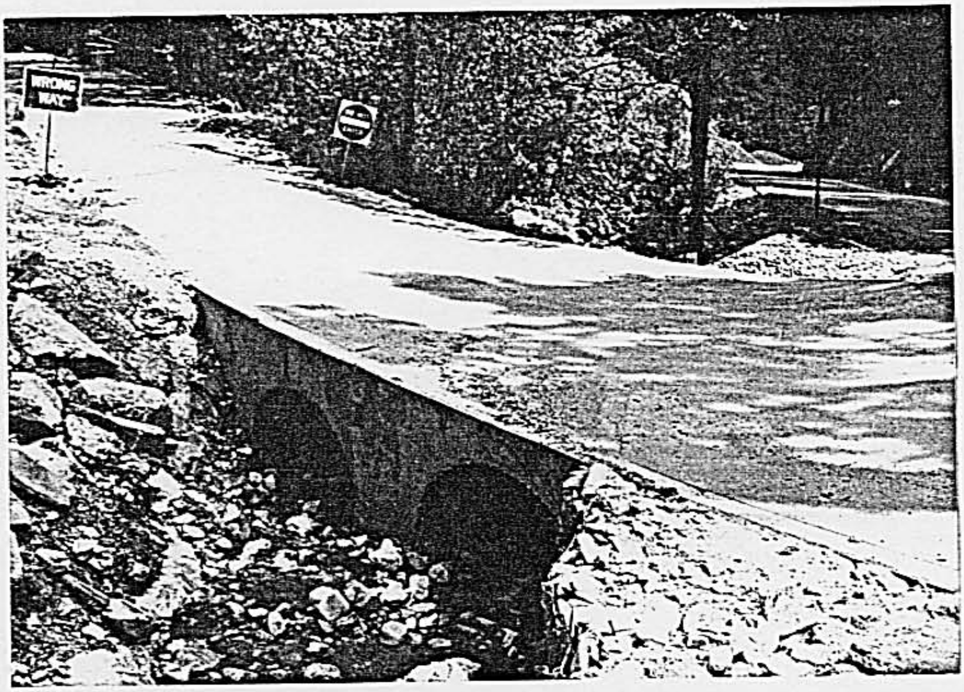
Downstream of 1st crossing at Kimball Springs along Sabino Canyon Park
PERMIT ON FILE WITH PIMA COUNTY

#14: 2-48" CMPs Q_{cap} = 276 cfs



Downstream side of 2nd crossing at Kimball Springs along Sabino Canyon Park
NO PERMIT FOUND

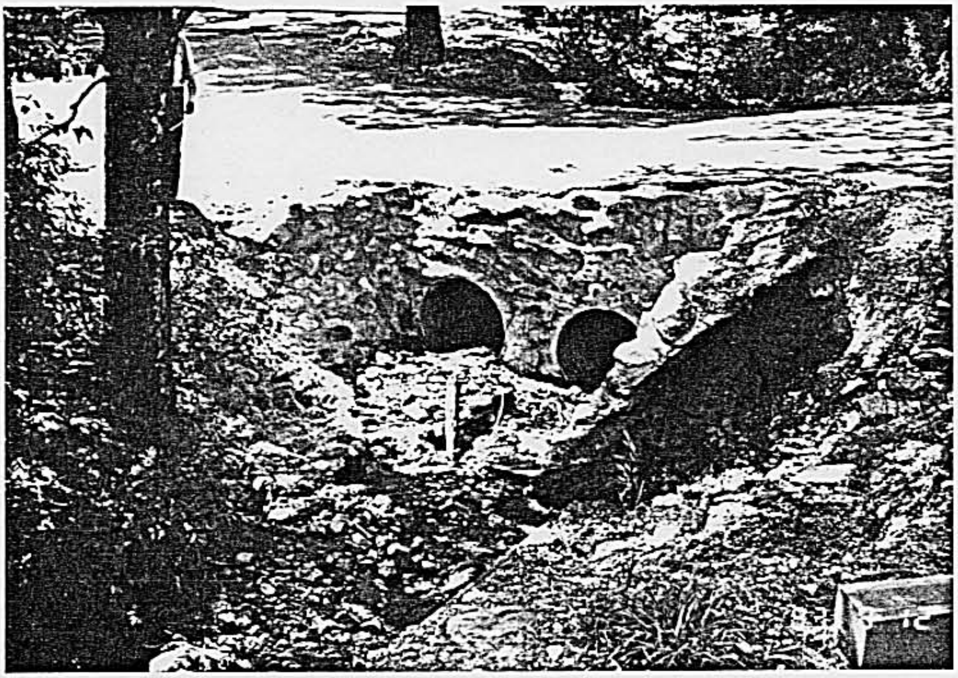
#15: 1-32" CMP, 2-24" CMPs Q_{cap} = 110 cfs



Upstream side of crossing at N. Middle Sabino and Sabino Canyon Park

NO PERMIT FOUND

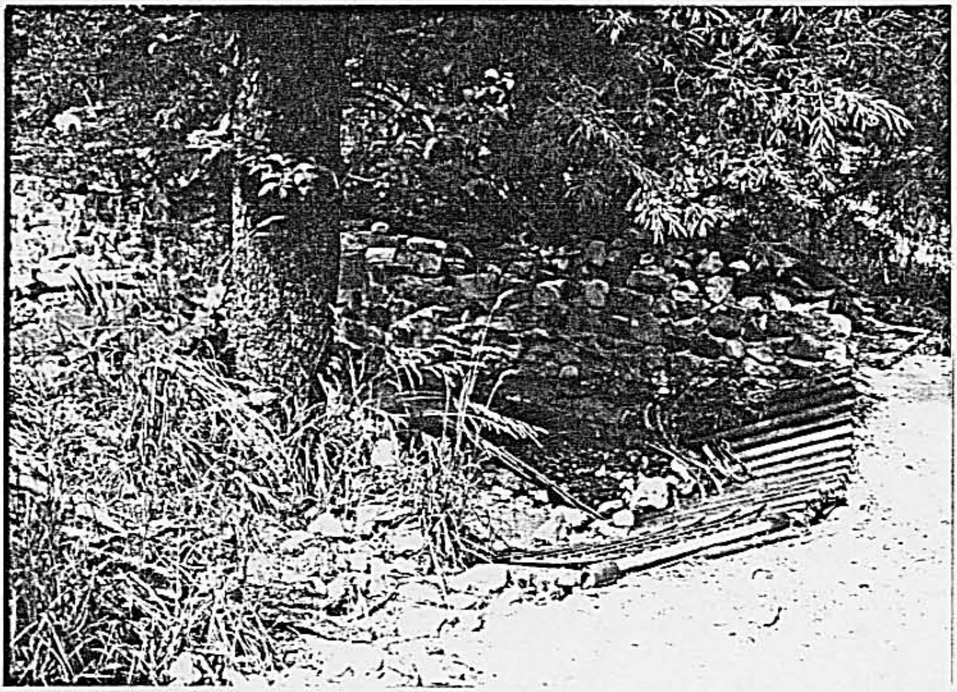
2-54"W x 36"H CMPs Q_{cap} = 100 cfs



Downstream of 3rd crossing at Kimball Springs along Sabino Canyon Park

PERMIT ON FILE WITH PIMA COUNTY

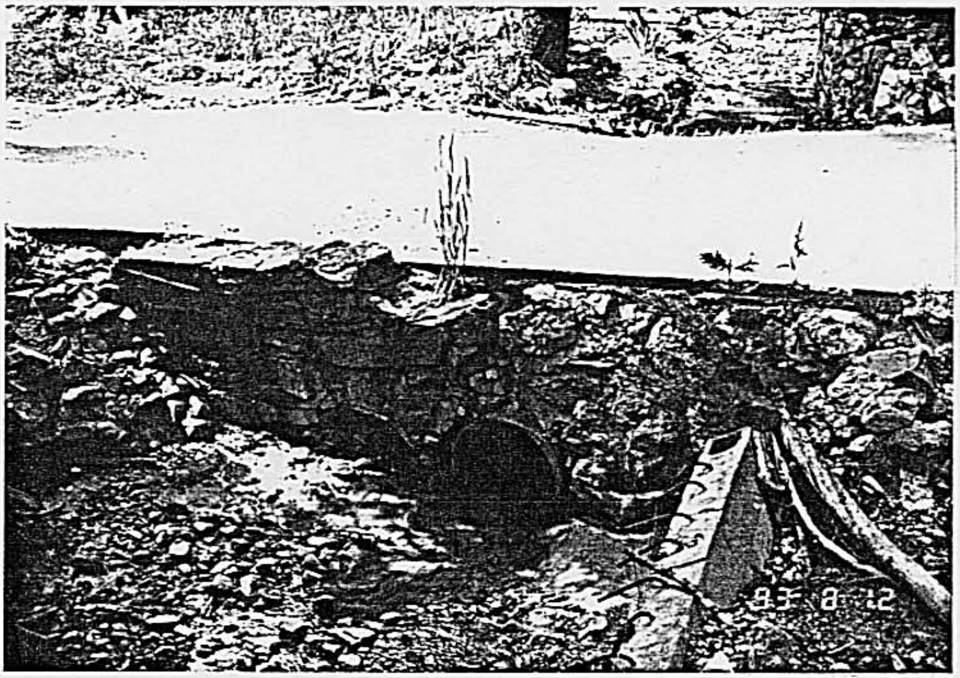
#16: 2-36" CMPs Q_{cap} = 140 cfs



Upstream side of crossing at 12824 Sabino Canyon Park

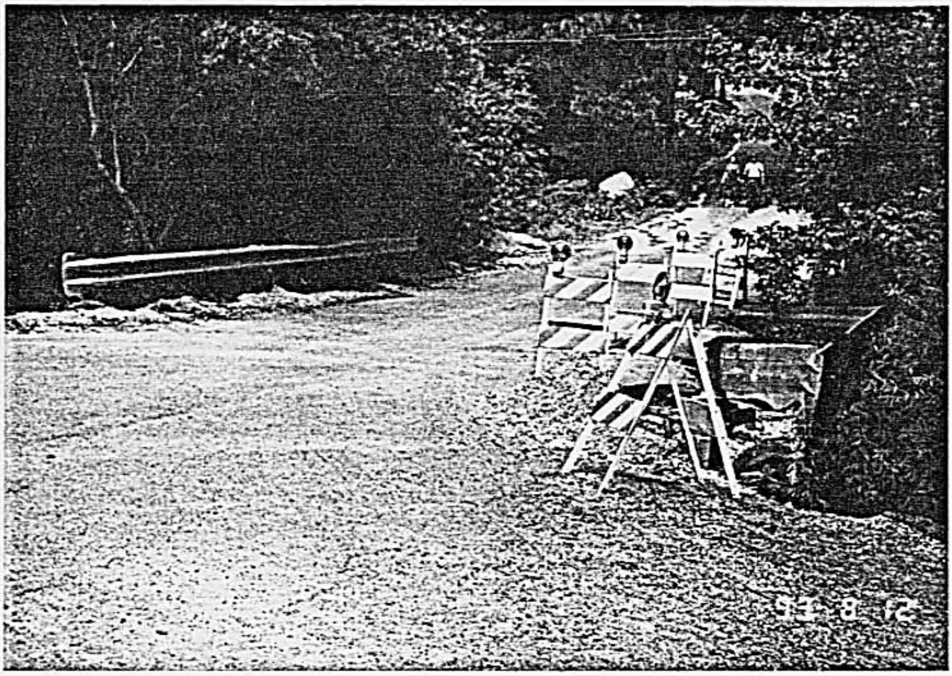
NO PERMIT FOUND

2-18" pipes $Q_{cap} = ?$



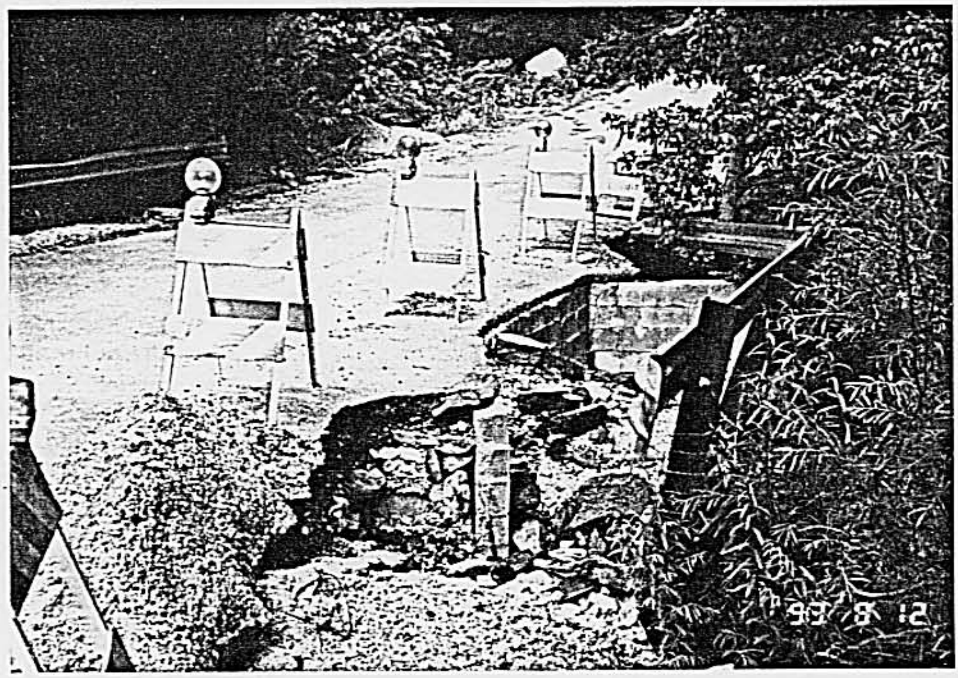
Downstream side of crossing at 12824 Sabino Canyon Park

#17: 2-18" pipes $Q_{cap} = ?$



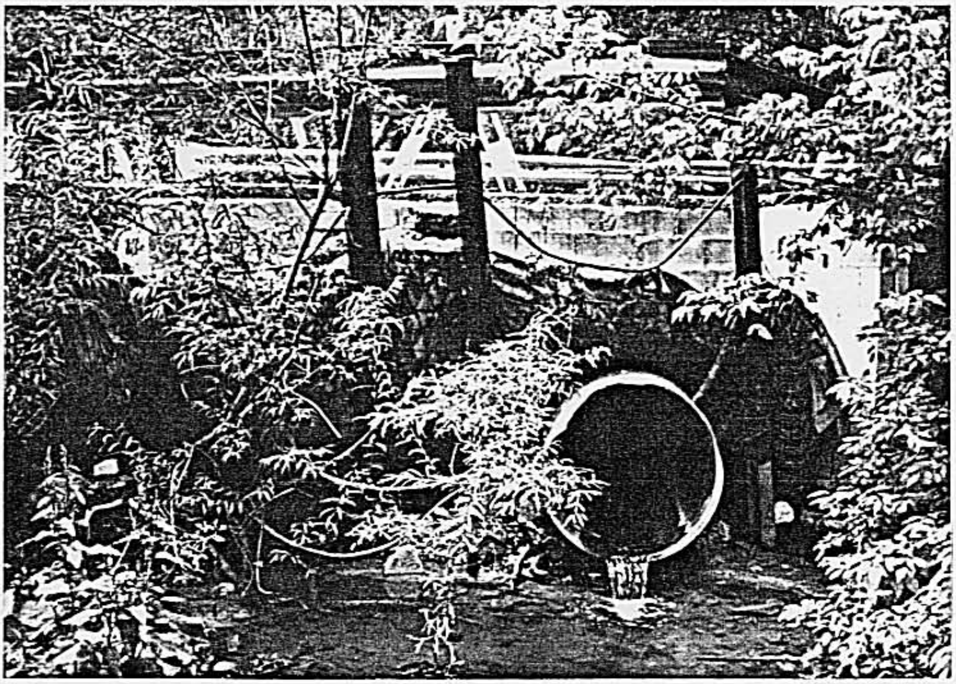
Dip crossing at Goat Hill Road along Sabino Canyon Park

NO PERMIT FOUND



Downstream side of dip crossing at Goat Hill Road along Sabino Canyon Park

NO PERMIT FOUND



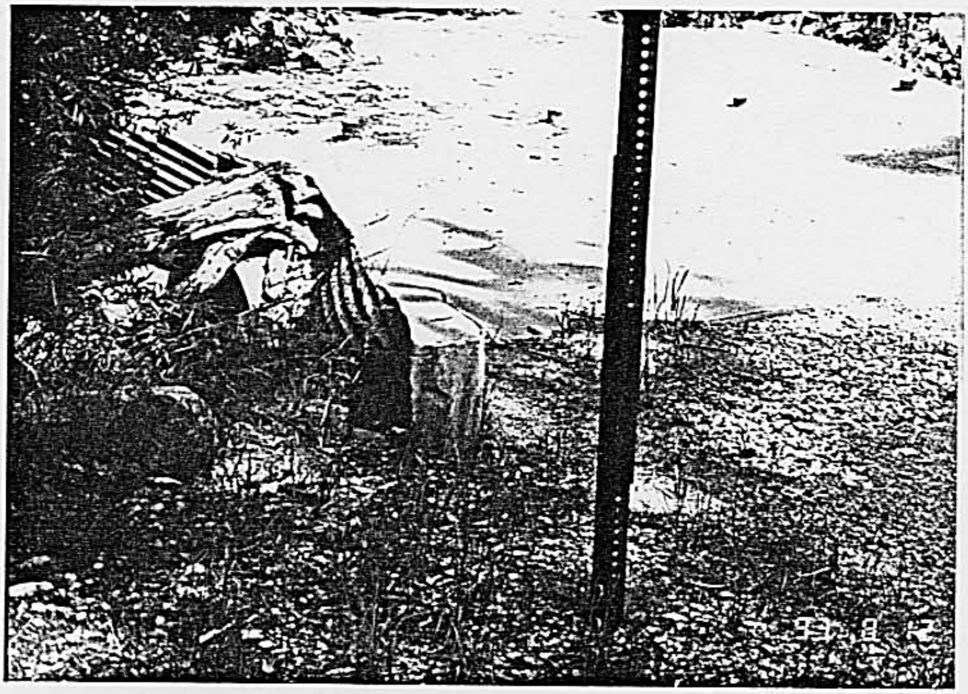
Downstream side of crossing at Goat Hill Road along Sabino Canyon Park

NO PERMIT FOUND

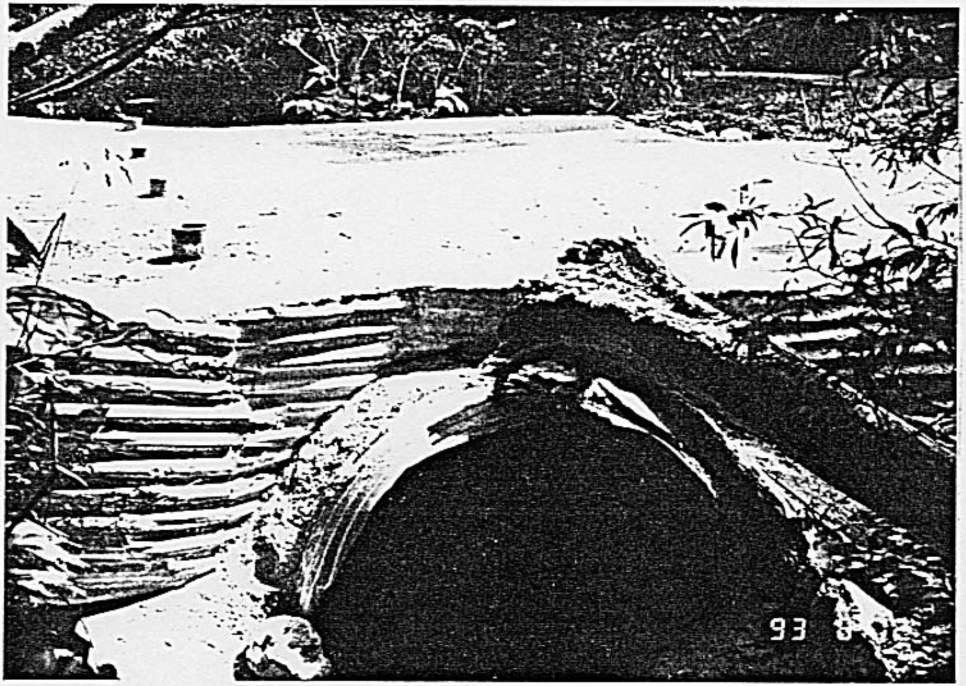
#18: 1-36" CMP Q_{cap} = 70 cfs

Downstream of crossing at Goat Hill Road along Sabino Canyon Park

Dip crossing at Retreat Road along Sabino Canyon Park



Dip crossing at Nogales Street along Sabino Canyon Park



Upstream side of crossing at Nogales Street along Sabino Canyon Park

PERMIT ON FILE WITH PIMA COUNTY

#19: 1-44"W x 36"H CMP Q_{cap} = 48 cfs

PIMA COUNTY
DEPARTMENT OF TRANSPORTATION AND FLOOD CONTROL DISTRICT

JOHN M. BERNAL
DIRECTOR

(602) 740-6410
FAX (602) 620-1933

September 30, 1993

Dear Property Owner:

Pima County Department of Transportation and Flood Control (DOT & FCD) has contracted with CMG Drainage Engineering Inc. to develop standardized access design for properties adjacent to Sabino and Carter Creek. Property owners are currently required to individually hire civil engineers to design access across these streams in conformance with the Pima County Floodplain Management Ordinance #1988-FC2. The goal of the study is to minimize the drainage impact of the access structures while still providing adequate access across the streams to individual parcels. This study may save many property owners the expense of hiring an engineer. CMG and this department welcome any written comments and observations by affected property owners. Comments on access design or related drainage concerns should be sent to:

CMG Drainage Engineering Inc.
P.O. Box 1425
Tucson, AZ 85702

Other questions or comments about this study can be addressed to:

B.J. Davis
DOT & FCD
Public Works Bldg., 4th Floor
201 N. Stone
Tucson, AZ 85701
(740-6350)

Sincerely,

B.J. Davis

B.J. Davis, Hydrologist
Project Manager
Floodplain Management Section

CC: John Wallace
Mary Lou Johnson
Dave Smutzer

Mount Lemmon Realty

MOUNT LEMMON SPECIALIST

P.O. BOX 1 MT. LEMMON, AZ 85619 (602) 576-1333

October 8, 1993

B. J. Davis
DOT & FCD
Public Works Bldg., 4th Floor
201 N. Stone
Tucson, AZ 85701

Bridge design criteria for Summerhaven

Dear B. J. Davis,

This letter is in regards to the county efforts to standardize access across the Sabino Creek in Mt. Lemmon, which I fully support. Before the access plans are finalized, would it be possible for me to review the access plan requirements? I pretty well know where all potential bridges could be constructed and am well acquainted with site specific problems.

The reason I ask for this is that there will be only a few new bridges that will be needed so that I feel that the design requirements will need to be quite specific for these potential sites. The design, it is hoped, will accommodate special limitations imposed by the area where the bridge is to be built. An example is along Turkey Run Road, where there is a relatively deep stream bed together with the narrow road easement. In addition, in this area, the fall of the stream bed is large, so that in a relatively few feet the ability of an upstream bridge to affect a downstream bed is limited to a short distance.

I have concerns for specific sites for which bridges will be needed and the impact the bridge design criteria may have on the property owners. I am concerned that the bridge design criteria would be so restrictive and or expensive to accommodate that it would become impossible to construct access to these properties. The other choice, available to the property owner, is to not build a bridge and park along Turkey Run Road, which is, in some areas, only 30 feet wide.

In addition to the private sector, I am concerned about the progress that will be made prior to winter to restore the existing county bridges to their full potential, particularly in recognition of their inadequate design. For example, one of the two culverts in the main bridge in the village is still plugged from last winter. I feel that if this situation is not corrected, that this main bridge, which is vital to keep open for emergency vehicles, could easily be lost again.

As this bridge has repeated failed every few years at great expense to Pima County, are there any plans to up grade it?

Thanks.

Sincerely,

Bob Zimmerman

Robert T. Zimmerman

cc

Paul Marsh, County Supervisor
CMG Drainage Engineering Inc
Mt. Lemmon Homeowners Association
Mt. Lemmon Fire District

MT. LEMMON HOMEOWNER'S ASSOCIATION
P.O. BOX 699
MT. LEMMON, ARIZONA 85619

B. D. Davis
DOT & FCD
Public Works Bldg. 4th Floor
201 N. Stone
Tucson, Az. 85701-1207

RE: Bridge Design Criteria, Mt. Lemmon, Arizona

Dear Mr. Davis,

Robert Zimmerman responded in a letter dated Oct. 8, 1993 to the letter from your office concerning access to properties adjoining Sabino & Carter Canyon creeks. The Mt. Lemmon Homeowner's Association supports Mr. Zimmerman's request to review the access plans before finalization. We believe Robert Zimmerman, a long time resident who is actively involved in many aspects of this community, would prove a valid and practical source of information and knowledge on specific sites that would be affected by the contemplated changes in county floodplain ordinances.

The Mt. Lemmon Homeowner's Association would appreciate being placed on your mailing list so that we may access the information from your office in a timely manner, and disperse it to our 240 members through our bimonthly publication. The mailing address is on the letterhead.

Sincerely,

Danielle Hayhurst

Danielle Hayhurst
President

cc

Paul Marsh, County Supervisor
CMG Drainage Engineering Inc.

APPENDIX F

NORMAL DEPTH COMPUTATIONS FOR CHANNEL CROSS SECTIONS

1"=200'H

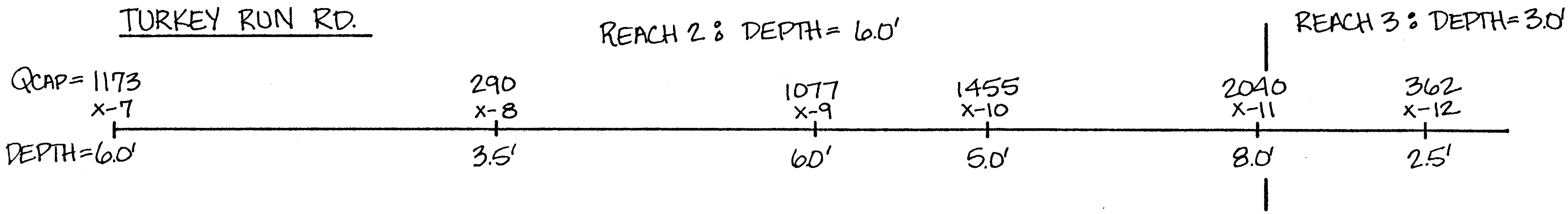
MT. LEMMON - SUMMERHAVEN

CARTER CANYON RD.

REACH 1 : DEPTH = 4.0'

TURKEY RUN RD.

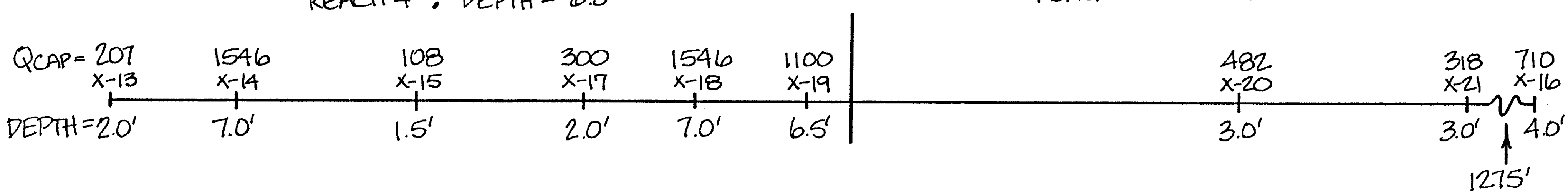
REACH 2 : DEPTH = 6.0'



SABINO CANYON PARKWAY

REACH 4 : DEPTH = 6.0'

REACH 5 : DEPTH = 3.0'



075

Mt. Lemmon - Summerhaven

#92047-4

Cross-section #	Wash	slope	Depth	bottom-width	side-slope	Capacity	Velocity	Comments
1	Carter Canyon	0.13 ft/ft	4.0 ft.	9 ft.	~1:1.3	739 cfs	12.02 fps	
2	Carter Canyon	0.13	4.1	8	~1:0.6	1021	16.61	
3	Carter Canyon	0.13	6.0	4	~1:0.75	1212	18.08	
6	Carter Canyon	0.13	4.0	6	~1:0.80	599	15.44	
7	Turkey Run	0.10	6.0	4	2:1	1173	16.30	
8	Turkey Run	0.10	3.5	4	~1:1.2	290	11.82	upstream of culvert, capacity reduced by deposition
9	Turkey Run	0.10	6.0	5	1:1	1077	16.31	
10	Turkey Run	0.10	5.0	8	2:1	1455	16.17	
11	Turkey Run	0.10	8.0	6	0:1 (rd.)	2040	18.15	
12	Turkey Run	0.10	2.5	8	2:1	362	11.13	breakout area
13	Sabino	0.06	2.0	10	2:1 (rd.)	207	7.96	
14	Sabino	0.06	7.0	8	1:1	1546	14.72	
15	Sabino	0.06	1.5	8	2:1 (rd.)	108	6.55	upstream of culvert, capacity reduced by deposition
16	Sabino	0.06	4.0	8	2:1	710	11.09	" "
17	Sabino	0.06	2.0	10	10:1 (rd)	300	7.14	upstream of culvert, capacity reduced by deposition
18	Sabino	0.06	7.0	8	1:1	1546	14.72	

Mt. Lemmon-Summerhaven

#92047-4

Cross-section #	Wash	slope	Depth	bottom-width	Side-slope	Capacity	velocity	Comments
19	Sabino	0.06 ft/ft	6.5 ft.	6 ft	1:1	1100 cfs	13.54 fps	
20	Sabino	0.06	3.0	4	5:1	482	8.46	upstream of culvert, capacity reduced by deposition
21	Sabino	0.06	3.0	8	1:1	318	9.64	

APPENDIX G

CAPACITY COMPUTATIONS FOR EXISTING DRAINAGE CULVERTS

Mt. Lemmon - Summerhaven

#92041-4

Culvert #	Address	Type	~~HW~~ elevation	Capacity	Comments
1	11000 Carter Canyon	bridge	—		See photos
2	11113 Carter Canyon	36" CMP	3.0'	35 cfs	
3	11119 Carter Canyon	36" CMP	—	—	driveway washed out
4	Blk. 30 Lots 25 & 26 Carter C.	36" CMP	3.0'	35	
5	11149 Carter Canyon	2-30" CMP	5.0	90	
6	11179 Carter Canyon	36" CMP	3.0'	35	
7	E. of 11179 Carter C	36" CMP	3.0'	35	
8	11229 Carter C. N to S side of road	42" CMP	5.5	86	
9	10950 Turkey Run	36" CMP + 24" CMP	7.5	80+39 119	
10	Turkey Run & Guthrie Rd	36" CMP	3.0	35	
11	? Turkey Run	2-42" CMP's	7.5	212	
12	11122 Turkey Run	2-42"H x 60" W CMP's	5.5	224	
13	Turkey Run to Sabino	1-36" CMP		—	Blocked with sediment
14	1st Kimball Springs Crossing	2-4			

Post-it® Fax Note 7671
Date
# of pages ▶
To
From
Co./Dept.
Co.
Phone # #1-6
Phone # See F
Fax #
Fax #

079

#92047-4

Mt Lemmon - Summerhaven

Culvert #	Address	Type	Hw elevation	Capacity
15	2nd Kimball Springs Crossing	33" CMP 2-24" CMP	4.7' ? botm	50+60 = 110
16	3rd Kimball Springs crossing	2-36" CMP	6.0	140
17	12824 Sabino	2-18" CMP	—	see photos for inlet
18	Goat Hill & Sabino	1-36" CMP	6.0	70
19	Nogales St. & Sabino	36" H x 44" W	4.0	48
20	N. Middle Sabino & Sabino	2-36" H x 54" W	4.5	100

APPENDIX H

DRIVEWAY CROSSING DESIGN CHECKLIST

081

Driveway Crossing Design Checklist

1. A field inspection of the proposed crossing site must be made to determine if the standard design for the culvert crossing structure is appropriate for that location or if a concrete ford can be used as an alternative. Additionally, this field inspection should identify if there are any other viable locations for providing the needed access that would eliminate the need for a new crossing installation.

2. Determine the channel width/depth ratio.
   a. If W/D is ≥ 10 and the average channel depth is 3 feet or less, then a concrete ford may be used for this crossing site.
   b. If W/D is ≤ 10 or the channel depth is >3 feet, then the standard design for a culvert crossing should be used.

3. Determine the location of the nearest existing downstream culvert. If <50 feet, determine if proposed culvert structure can be moved to reduce the threat that the downstream culvert will cause sediment deposition which may affect the hydraulic capacity of the proposed culvert.

4. Determine the location of the nearest upstream culvert. If <30 feet, determine if the proposed culvert structure can be moved to reduce the threat of scouring turbulence caused by overflow or diversions at upstream culvert.

5. Supply owner or builder with design specifications for standardized culvert installation or concrete ford, whichever applies to the site and stream segment.

6. Inform owner or builder of equivalent alternative structure listed on the plan detail sheets.

Carter Canyon Wash Design Recommendations

The average channel depth along Carter Canyon Wash is about 4 to 4.5 feet, and the bottomwidth ranges from 6 to 8 feet. The selected culvert structure for this average channel geometry is a single 64" x 43" corrugated metal arch pipe. The corregation should be 2-2/3" x 1/2". The H2O load capacity for this pipe is 32,000 lbs. per axle. This pipe culvert has a capacity of about 90 cfs at a headwater depth of 4 feet. The surface profile for the driveway crossing has a minimum depression depth of 6 inches. Larger depressions may be provided (up to 1.0 foot) where the natural channel depth exceeds 4.5 feet.

A typical cross section of the culvert and headwall face is provided on Figure 4 of this report. All standard Pima County requirements shall apply.

Turkey Run Road Wash Design Recommendations

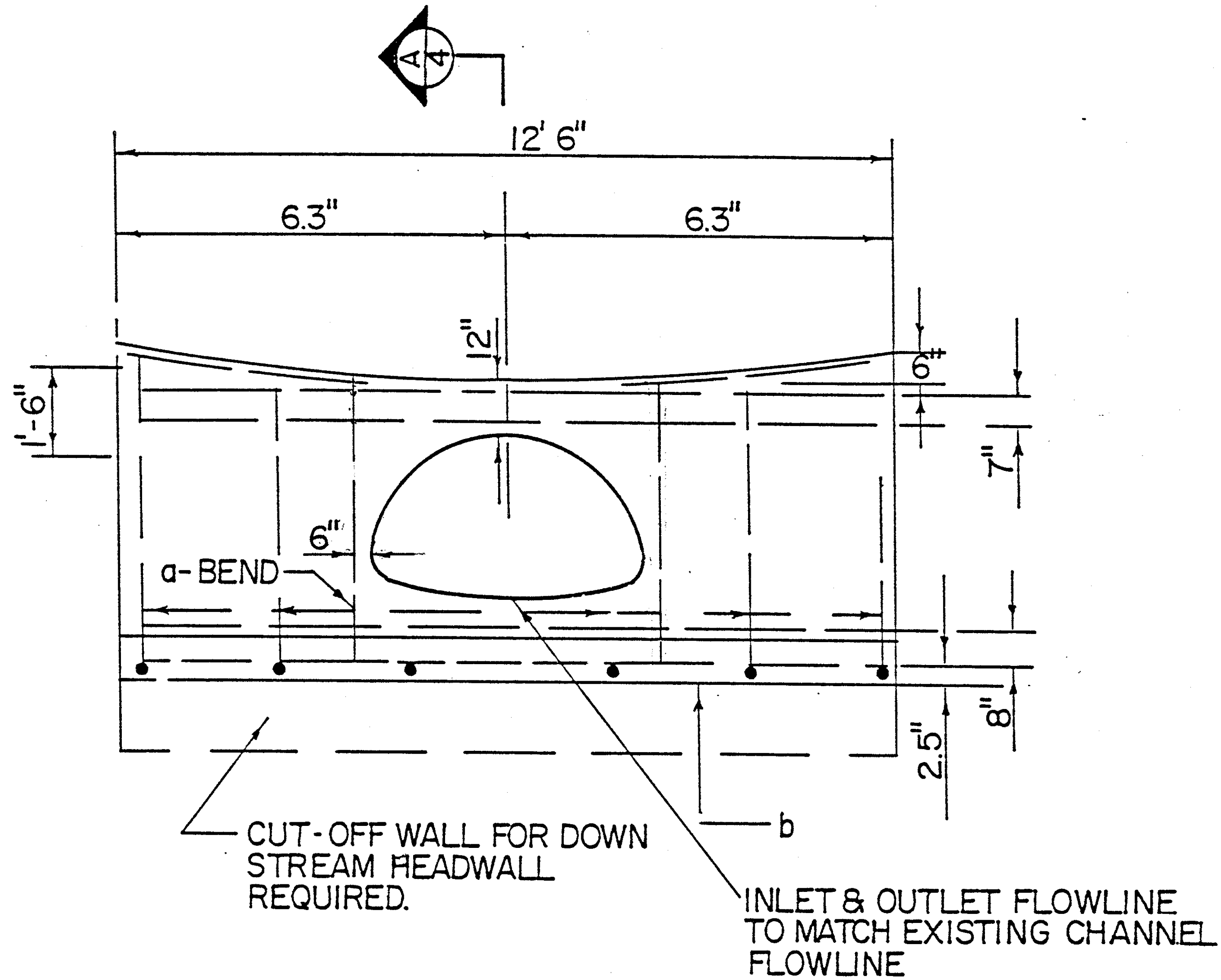
The average channel depth along Turkey Run Wash is about 5 to 6 feet, and the bottomwidth ranges from 6 to 10 feet. The selected culvert structure for this average channel geometry is one 71" x 47" corrugated metal arch pipe. The corregation should be 2-2/3" x 1/2". The H2O load capacity for this pipe is 32,000 lbs. per axle. This pipe culvert has a capacity of about 155 cfs at a headwater depth of 6 feet. The surface profile for the driveway crossing has a minimum depression depth of 1 foot. Larger depressions may be provided (up to 2.0 feet) where the natural channel depth exceeds 6.0 feet.

A typical cross section of the culvert and headwall face is provided on Figure 5 of this report. All standard Pima County specifications shall apply.

Sabino Creek Design Recommendations

The average channel depth along Sabino Creek varies from 3 to 7 feet, and the bottomwidth ranges from 8 to 10 feet. The selected culvert structure for this average channel geometry is one 71" x 47" corrugated metal arch pipe. The corregation should be 2-2/3" x 1/2". The H2O load capacity for this pipe is 32,000 lbs. per axle. This pipe culvert has a capacity of about 155 cfs at a headwater depth of 6 feet. The surface profile for the driveway crossing has a minimum depression depth of 1 foot. Larger depressions may be provided (up to 2.0 feet) where the natural channel depth exceeds 6 feet.

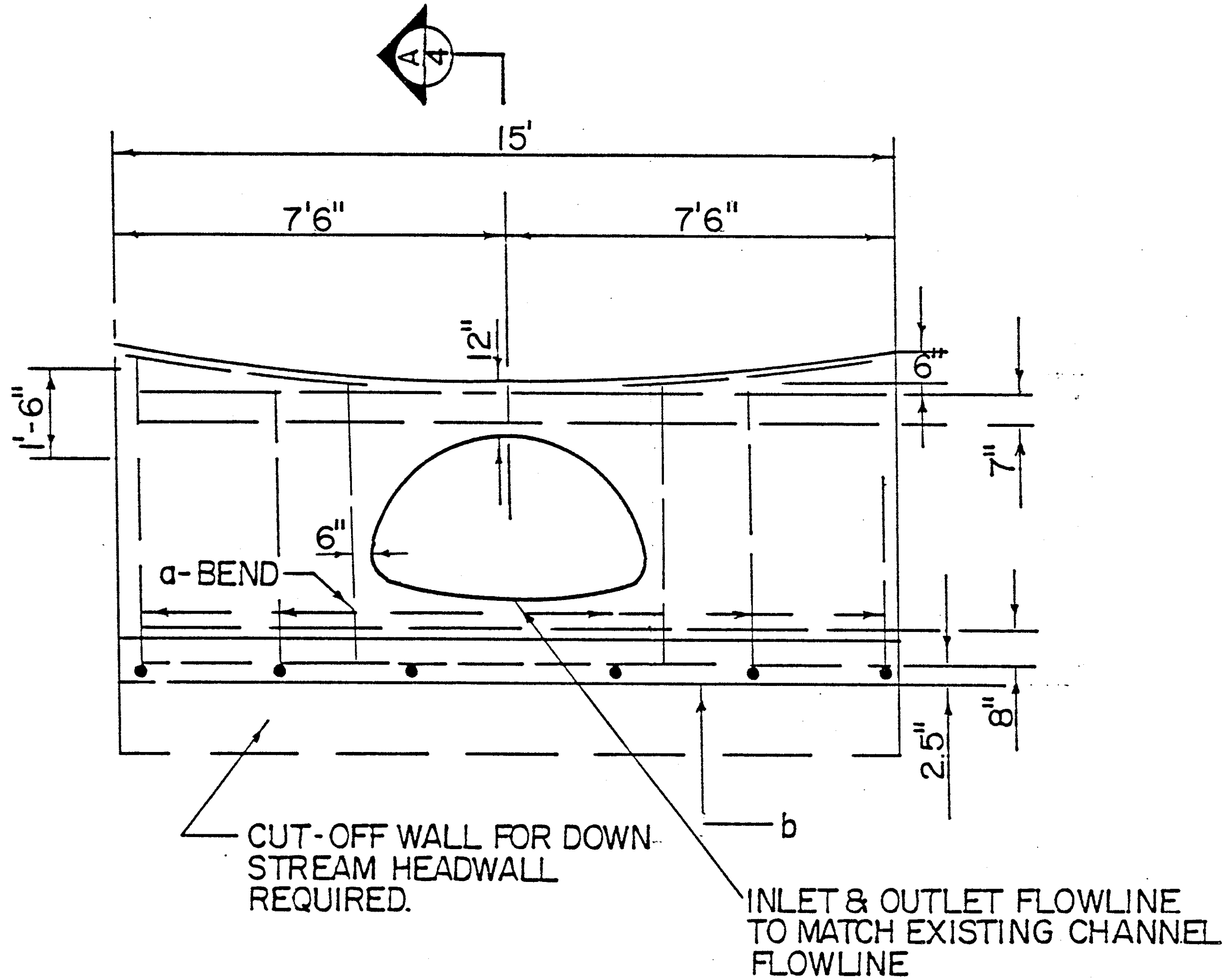
A typical cross section of the culvert and headwall face is provided on Figure 6 of this report. All standard Pima County specifications shall apply.



HEADWALL- CARTER CANYON WASH
1 - 64" x 43" ACMP

<u>STRAIGHT HEADWALL SINGLE PIPE</u>

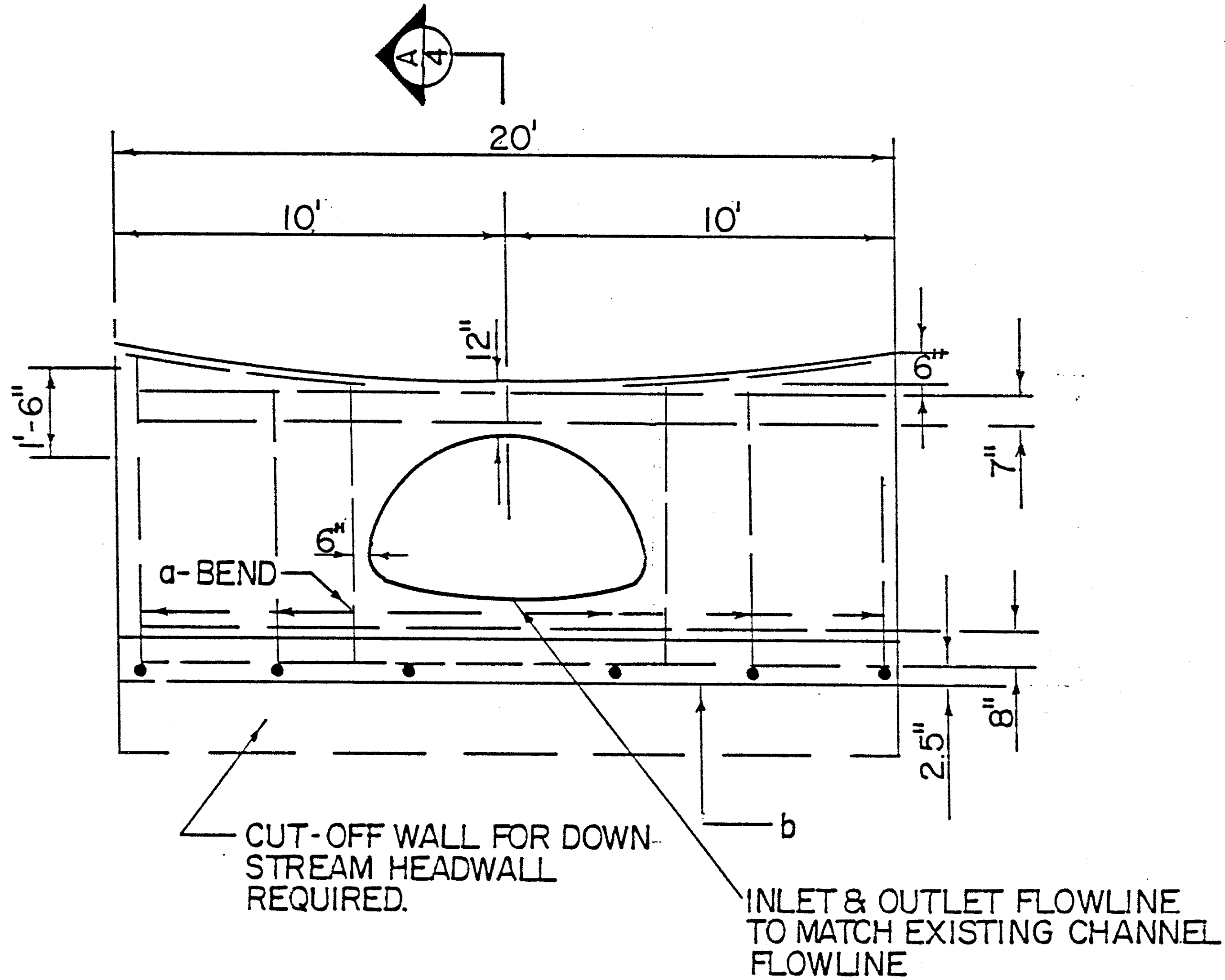
- REFER TO SHEET 4 FOR SECTION THRU HEADWALL AND GENERAL NOTES
- REFER TO SHEET 4 FOR DIMENSION TABLE
- PIPE GAUGE SHALL BE 12 GAUGE WITH 2 2/3" x 1/2" CORREGATIONS, H20 LOAD = 32,000 LBS. /AXLE
- CARTER CANYON WASH 1-64"x 43" ACMP
- ALL REBAR SHALL BE #4 EXCEPT AS NOTED ON SHEET 4



HEADWALL - TURKEY RUN ROAD WASH
1 - 71" x 47" ACMP

STRAIGHT HEADWALL SINGLE PIPE

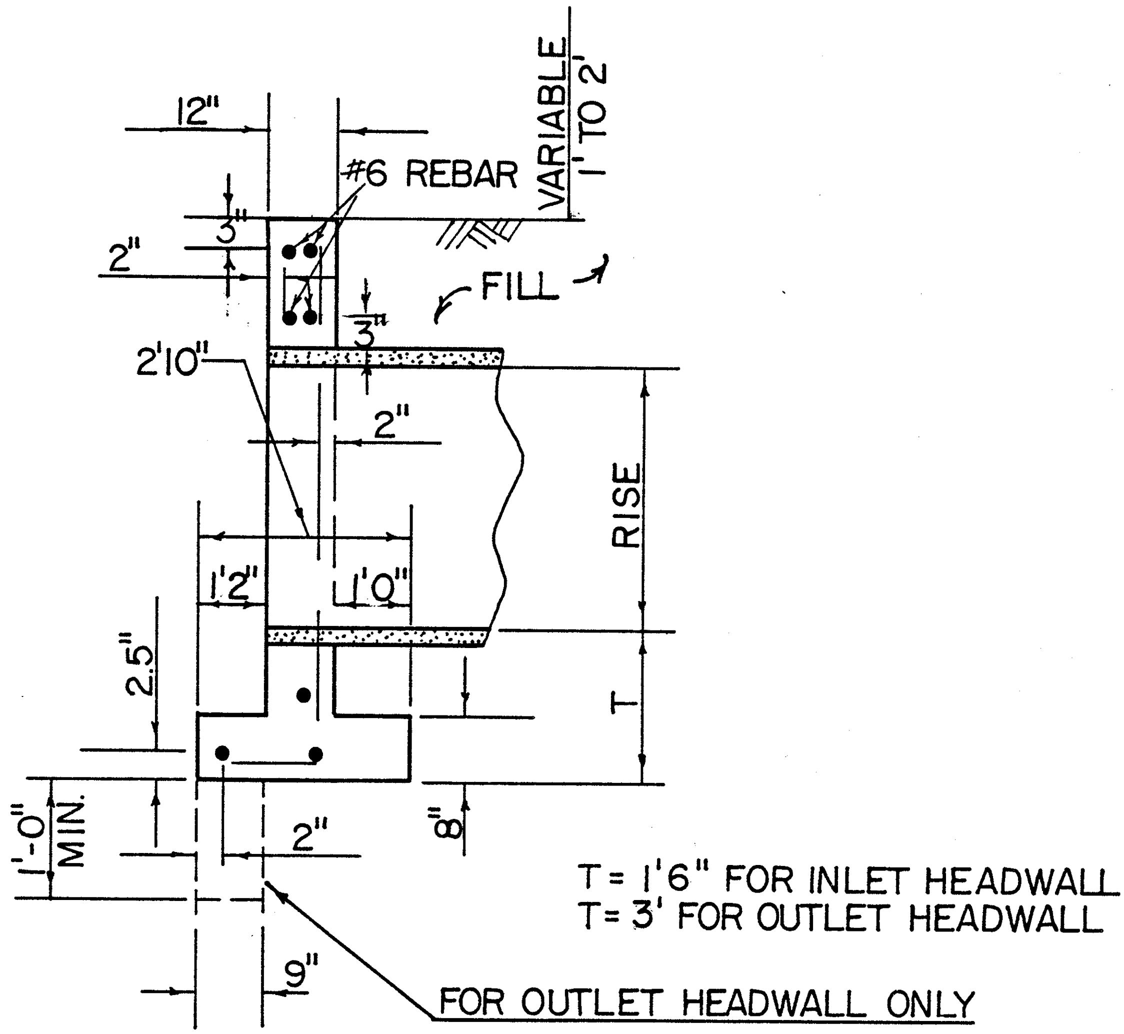
- REFER TO SHEET 4 FOR SECTION THRU HEADWALL AND GENERAL NOTES
- REFER TO SHEET 4 FOR DIMENSION TABLE
- PIPE GAUGE SHALL BE 12 GAUGE WITH 2 2/3" x 1/2" CORREGATIONS, H20 LOAD = 32,000 LBS./AXLE
- ALL REBAR SHALL BE #4 EXCEPT AS NOTED ON SHEET 4



HEADWALL- SABINO CREEK
1 - 71" x 47" ACMP

STRAIGHT HEADWALL SINGLE PIPE

- REFER TO SHEET 4 FOR SECTION THRU HEADWALL AND GENERAL NOTES
- REFER TO SHEET 4 FOR DIMENSION TABLE
- PIPE GAUGE SHALL BE 12 GAUGE WITH 2 2/3" x 1/2" CORREGATIONS, H20 LOAD = 32,000 LBS./AXLE
- ALL REBAR SHALL BE #4 EXCEPT AS NOTED ON SHEET 4

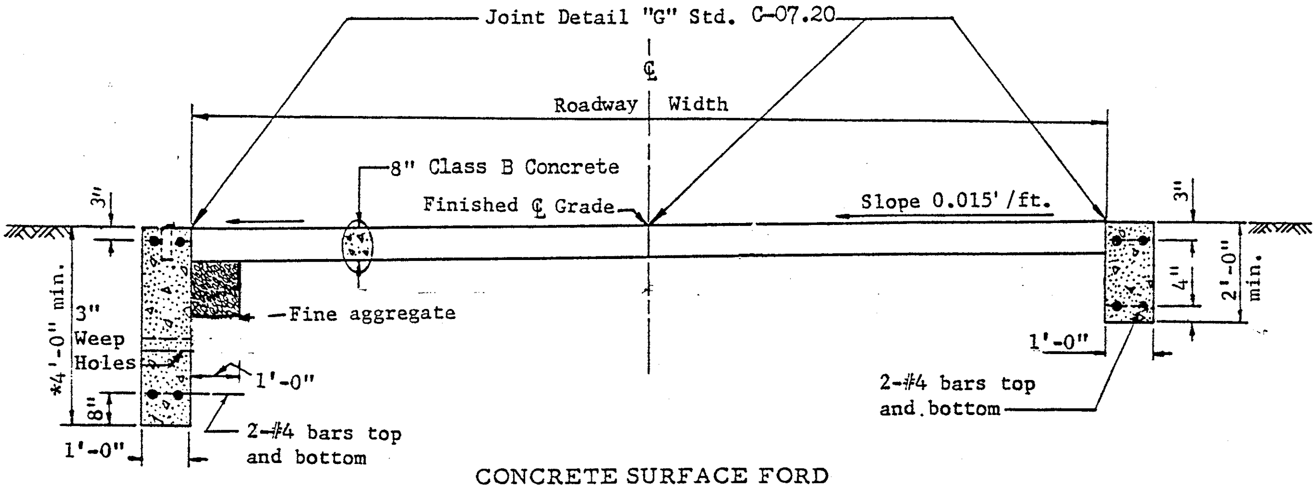


SECTION (A) THROUGH HEADWALL

GENERAL NOTES

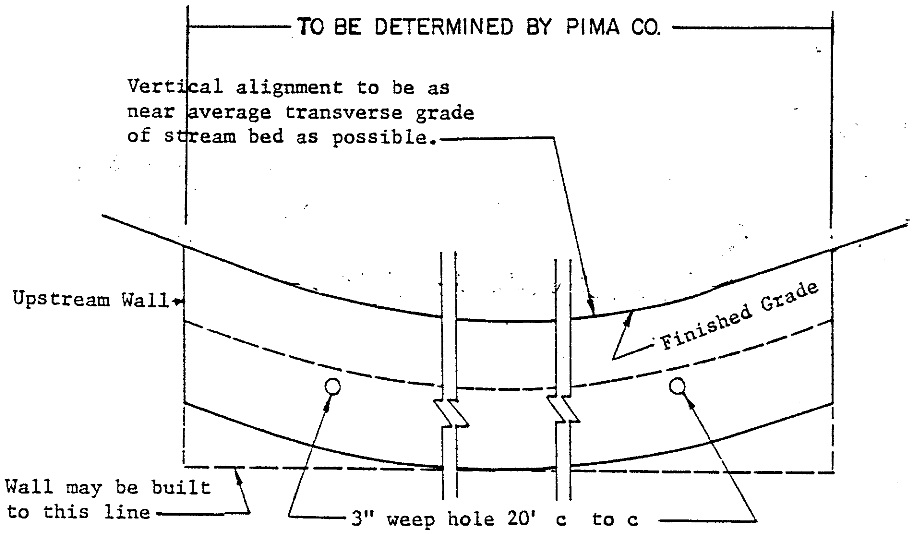
- ALL CONCRETE SHALL BE CLASS "A"
- FILL MATERIAL SHALL BE WELL COMPACTED
- ALL REBAR SHALL BE #4 EXCEPT AS NOTED

089

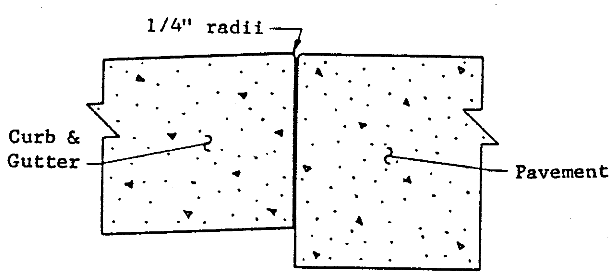


CONCRETE SURFACE FORD
CONCRETE WALLS

*Min. Distance
Below Stream Bed



ELEVATION LOOKING UPSTREAM



JOINT TYPE "G"

CONCRETE FORD DETAILS

LEGEND

- FLOOD DAMAGE & REPAIR SITES
- LOTS WITH FORD CROSSING POTENTIAL
- INSPECTIONS RECOMMENDED
- LOTS WITH CULVERT PERMIT

DETAIL I

D.P. BK. II PG. 35 SHT 2 of 5

091

DETAIL 2

D.P. BK II PG 35 SHT 3 of 5

092