ALBUQUERQUE INTERNATIONAL AIRPORT DRAINAGE AND TRAFFIC CIRCULATION STUDY

ALBUQUERQUE
NEW MEXICO
1706

NOV 1 8 1981

Tom Mann Associates, Inc.

811 DALLAS, N E • ALBUQUERQUE • NEW MEXICO • 87110

ENGINEERS

Tom Mann Associates, Inc.
Engineers
811 DALLAS, N.E. • ALBUQUERQUE • NEW MEXICO • 87110 • 505-265-5611

10511
November 18, 1981

Mr. Robert Gill
Aviation Supervisor
Albuquerque International Airport
P. O. Box 9022
Albuquerque, New Mexico 87119

Re: Albuquerque International Airport
Traffic Circulation and Drainage Study

Dear Bob:

Transmitted herewith are five (5) copies of the "Albuquerque International Airport Traffic Circulation and Drainage Study" for preliminary review as stipulated by the A/E Services Agreement 81-13. Please distribute copies to the appropriate reviewing departments.

Due to the multitude of data presented in the report, the reviewers may request a conference to explain the contents. Such a meeting can be arranged upon request.

If you have any further questions or comments concerning this submittal, do not hesitate to call.

Sincerely,

TOM MANN & ASSOCIATES, INC.

Jeffrey G. Mortensen
Project Engineer

Thomas T. Mann, Jr., P.E.
President

JGM:bb
Encs. (5)

ALBUQUERQUE INTERNATIONAL AIRPORT

DRAINAGE AND TRAFFIC CIRCULATION

STUDY

NOVEMBER 1981

PREPARED FOR: AVIATION DEPARTMENT
CITY OF ALBUQUERQUE
P. O. BOX 9022
ALBUQUERQUE, NEW MEXICO 87119

PREPARED BY: TOM MANN & ASSOCIATES, INC.
811 DALLAS N.E.
ALBUQUERQUE, NEW MEXICO 87110
(505) 265-5611

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APPLICATION FOR ZONE MAP AMENDMENT
AND/OR DEVELOPMENT PLAN

INSTRUCTIONS: PRINT OR TYPE IN BLACK INK ONLY. Use additional sheets, if necessary. Thirty (30) copies (Annexation, forty copies) of all attachments must be submitted with the application and, after folding, shall not exceed 8 1/2 x 14 inches.

Applicant's Name: AVIATION DEPT CITY OF ALBUQUERQUE requests

[ ] Zone Map Amendment [X] Development Plan Review [ ] Annexation
___ Type of Plan

For Lot(s): ____ Block(s): ____ in Subdivision/Addition _____
Presently zoned SU/1 and proposed to be zoned as SU/1 located at:
Street Address (No. If any): International Airport
Between _____ Street and _____ Street

Total Land Area Covered by Application: Acres 5 Square Feet _____

Number of DU's Proposed ____ DU's Per Gross Acre ____ DU's Per Net Acre ____

APPLICANT INFORMATION:

Applicant's Mailing Address: AVIATION DEP Phone: 842-4366
PO BOX 9022 Zip Code: 87119

Applicant's Proprietary Interest in the Property: OWNER

Agent if any: ROBERT GILL Phone: 842-4366
Mailing Address: PO BOX 9022 Zip Code: 87119
Signature of Applicant Robert Gill Date: 8-16-83

****

FOR PLANNING STAFF USE ONLY

Application Received By: Cynthia Borrego Date: 8/17/83

Fee: $ _____ Paid: $ _____ Method of Payment: _____

Cross-Ref.: AX- _____ Z- _____ S- _____ Other: _____

No. of Signs Issued: _____ ZHE or EPC? DRB-83-289

1 Copy each to:
Research Analyst
Applicant
Grapnics

Hearing Date: 8.30.83
Z-
Case Number

Map Number: N-16

Drainage report required prior to site development plan sign-off.

CITY OF ALBUQUERQUE

ALBUQUERQUE, NEW MEXICO

INTER-OFFICE CORRESPONDENCE December 29, 1981 REF. NO._____

TO: Richard S. Heller, City Engineer
R. L. Prate, Assistant Director, Aviation
FROM: Charles M. Easterling, Princ. Asst. City Eng./Hydrology CME

SUBJECT: AMFAC HOTEL DRAINAGE PROBLEMS

The attached memos from Brian Burnett of my staff point up the need to proceed towards a solution to the AMFAC hotel drainage problems. The platting and development actions in the area, both recent and proposed, place the City in a good position to get many old problems resolved at very low City expense, i.e. Craddock, H & H Construction, Budget Rent-A-Car and several others. I recommend that Tom Mann & Associates be retained immediately to provide some temporary and permanent solutions to the area drainage problems and that by doing so we will be in a posture to act in concert with developer construction activities this spring, to the benefit of all. Brian Burnett can coordinate these activities if the answer is "GO".

CME/BB/tsl

cc: C. Rodolph, Director of M.D.D.
Brian G. Burnett, Civil Eng./Hydrology
Tom Mann, Tom Mann & Assoc.

AIA Dr. + Traffic Circulation Study

CITY OF ALBUQUERQUE

ALBUQUERQUE, NEW MEXICO

INTER-OFFICE CORRESPONDENCE December 22, 1981 REF. NO._____

TO: C. M. Easterling, Principal Assistant City Engineer/Hydrology

FROM: Brian G. Burnett, Civil Engineer/Hydrology BGB

SUBJECT: H & H Airport Property

This memo is in response to Carl Rodolph's request for information concerning the City's commitment to protect the H&H Site from surface runoff imminating from the AMFAC Hotel. In a telephone conversation with Mr. Bob Gill, Assistant Airport Director, he indicated that the Airport is planning to pay for any improvements that are designed to alleviate this problem. Bob indicated that a temporary solution may be required until the permanent collection system located in the Girard extension is constructed. We both agreed that the City should proceed with finalizing a scheme.

BGB/fs
9480A

cc: Carl Rodolph, MDD
Bob Gill, Airport

CITY OF ALBUQUERQUE

ALBUQUERQUE, NEW MEXICO

INTER-OFFICE CORRESPONDENCE December 22, 1981 REF. NO._____

TO: R. L. Pratte, Director of Aviation

FROM: Brian G. Burnett, CE/Hydrology BGB

SUBJECT: Airport Development Plan

The following comments are made in reference to the drainage proposals outlined for the referenced plan:

1. Table 3 - The engineer should supply criteria used to establish the curve numbers for the different ground covers. It is the opinion of this office that curve numbers of 54 do not adequately represent Albuquerque runoff characteristics resulting from high intensity storms. Based on past work experience by several engineers in this office, curve numbers less than 65 are not realistic for this area of Albuquerque. Therefore, watersheds utilizing these low curve numbers should be reanalyzed to check the capacity of existing and proposed facilities.

2. Item 10, Page 31 - There are several places along the north boundary that the slope is almost vertical. The stability of this slope is of great concern and a solution should be coordinated between Craddock Dev. and the City at the earliest possible date.

3. Item 2, Page 36 - The report suggests the establishment of a program of maintenance for cleaning catch basins. Because of budget limitations for this type of work, a program may not be feasible at this time.

4. Item 3, Page 36 - This office requests that the engineer prepare a list of advantages and disadvantages for each alternative and then make a recommendation based on these findings.

5. Item 4, Page 36 - This office is currently investigating who is financially responsible for making this connection. Representatives from Craddock and the City have been meeting in an effort to make this determination.

6. Item 9, Page 37 - Runoff generated by this site after the proposed regrading should discharge onto Girard Blvd. at a controlled rate.

7. Item 10, Page 37 - See #2 above.

8. Overall, the traffic and drainage is comprehensive and very well done.

BGB/fs
9482A

cc: Jeff Mortensen, Tom Mann Assoc.
Richard Heller, City Engineer

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I. SUMMARY

This report has been prepared for the City of Albuquerque Aviation Department to study traffic circulation and drainage conditions in and around the Albuquerque International Airport. The report analyzes and discusses existing traffic and drainage conditions. Following the analysis and discussion of existing conditions, traffic and drainage plans are presented which address the problems associated with the existing conditions. The traffic and drainage plans have been integrally developed to ensure compatibility. The plans have been presented in phases to accommodate staged development.

The plans involve the construction of new roadway, the construction of new storm drainage facilities, the modification of existing storm drainage facilities, the reconstruction/widening of existing roadways and the modification of the parking lots. These improvements are discussed in detail in Sections III and IV for the Traffic Plan and Drainage Plan respectively. The recommendations and conclusions for traffic and drainage are combined and presented with respect to Phase in Section V. The implementation of the plans is recommended in phases in stride with development.

II. INTRODUCTION

A. BACKGROUND

In 1939, the Albuquerque International Airport began operations under the name Albuquerque Municipal Airport. The Albuquerque Municipal Airport, owned by the City of Albuquerque, was leased to the New Mexico Airport Corporation which was formed by Trans World Airlines to operate the facility. The City of Albuquerque gave the airport to the United States Government in 1950; however, the City of Albuquerque resumed ownership in December 1962. In October 1963, the Albuquerque Municipal Airport was renamed the Sunport, and in October 1971, became the Albuquerque International Airport.

The Albuquerque International Airport, despite several name and management changes, has served the Albuquerque area since 1939. During these 42 years of service, the Airport facilities have expanded to meet the needs of a growing Albuquerque; likewise, the surrounding lands have developed commercially in stride with the City's growth. Some of this commercial development has taken place on Airport land. For example, several car rental companies and a hotel are current holders of commercial leases with the Airport.

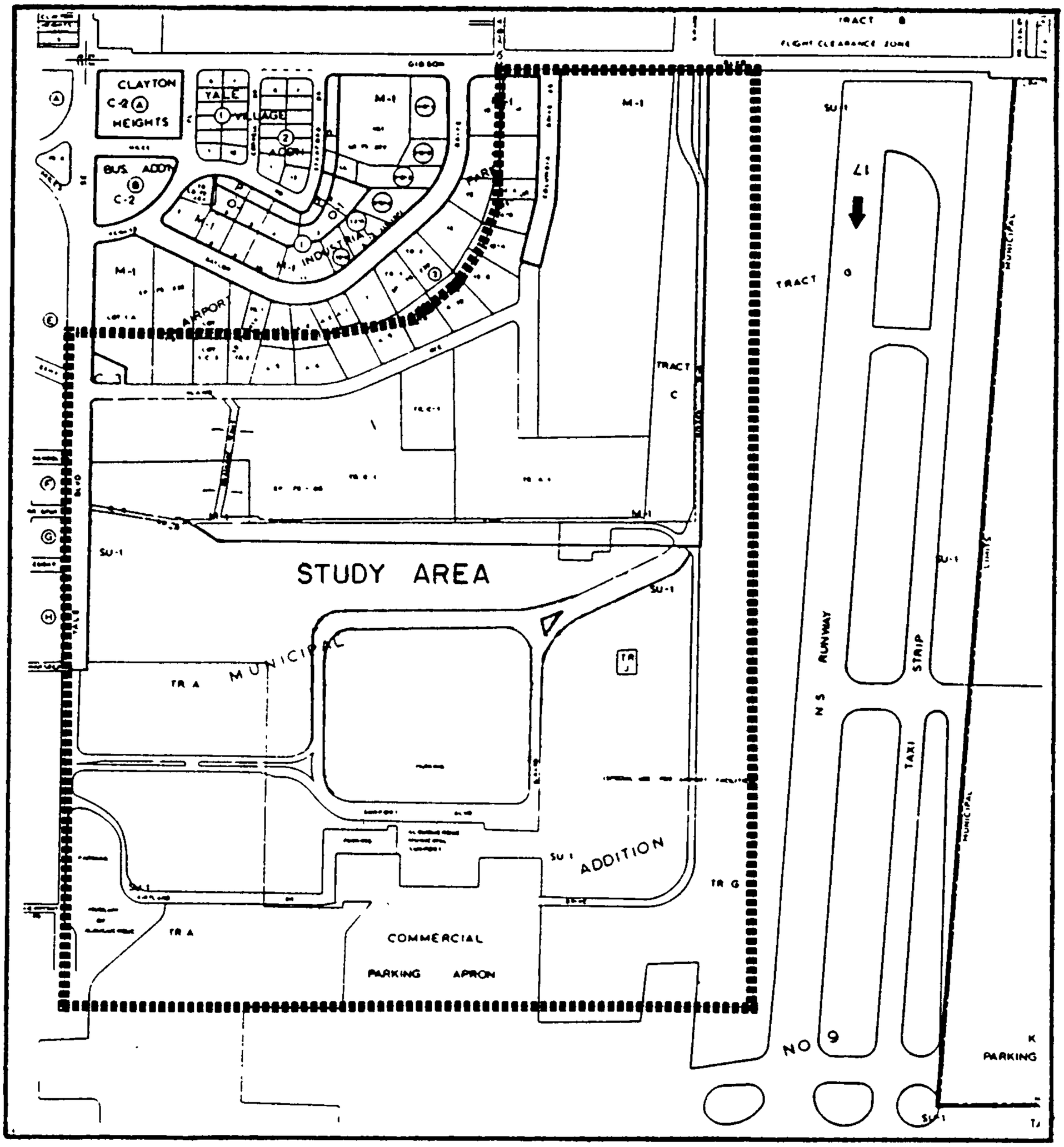
B. PURPOSE AND SCOPE

Although the majority of the airport land and its adjoining parcels are currently developed, vacant land is still available for commercial development. Such development could take the form of aviation facility expansion or development by private

concerns. In either case, careful thought must be given to the planning of any further development in the area. It is the purpose of this report to study the impact of future development on traffic circulation and drainage conditions. The area for which this study is presented is outlined in Figure 1.

The analysis of existing traffic patterns and drainage conditions is contained herein. This analysis includes a discussion of the advantages and/or disadvantages of the existing facilities. Existing traffic patterns, land uses and drainage patterns are identified as part of this discussion.

Several plans for the further development of the Airport area are presented in this report. These plans not only identify areas of future development, but also discuss the merits of the plans along with recommendations for the implementation of the respective plan. The emphasis of the plans is the improvement of traffic circulation in and around the Airport coupled with the adequate control of storm water runoff.



Scale: 1" = 800'

FIGURE 1: VICINITY MAP

III. TRAFFIC CIRCULATION

The purpose of this section is to discuss the existing traffic patterns in the study area and the problems associated with those patterns. Following the discussion of the existing traffic patterns, a plan is presented which addresses existing problems as well as accommodates future expansion and development. Lastly, recommendations are offered which suggest a manner in which the plans can be implemented.

A. EXISTING TRAFFIC PATTERNS

The main flow of traffic enters the Airport area from Yale Boulevard S.E. via Sunport Boulevard S.E. This traffic, for the most part, is forced to pass in front of the Terminal Building. A motorist not wishing to conduct business in that immediate vicinity must continue north to a 'Y' intersection where he or she can bear right and travel north on Girard Boulevard S.E., or can continue straight to 'T' intersection. At that intersection, the motorist must turn left and head back toward either Yale Boulevard or the Terminal Building. Motorists entering the Airport area from Girard Boulevard S.E. must also travel in this circular motion. These patterns which are awkward and inconvenient are depicted graphically by Figure 2.

There are various problems associated with the existing traffic patterns described above and shown by Figure 2. These problems are as follows:

VACANT LEASE SPACE

VACANT LEASE SPACE

VACANT LEASE SPACE

BUDGET RENT A CAR

DOLLAR A-DAY CAR RENTAL

F.A.A. FIELD MAINTENANCE

SCALE 1" = 200'

FUEL STORAGE

LOWER LOT - PARKING

K.A.F.B. WELL

AMFAC

AVIS RENT-A-CAR

RAMP

UPPER LOT - PARKING

HERTZ RENT-A-CAR

SUNPORT BLVD. S.E.

NATIONAL CAR RENTAL

TERMINAL BUILDING

EMPLOYEE PARKING

POST OFFICE

EMERGENCY POWER

AIR CARGO

AIR FREIGHT BUILDING

MUSEUM

EWPOINT ROAD

LEGEND

X147 EXISTING CONTOUR

5280 EXISTING SPOT ELEVATION

DIRECTION OF TRAFFIC FLOW

KEYED NOTES

1. BOOTH, PARKING LOT EXIT
2. ACCESS TO UPPER LOT
3. ACCESS TO LOWER LOT
4. A- CHECK-IN
5. SECURITY GATE
6. RENTAL CAR CHECK-IN
7. SINGLE CURB DIVIDER, 8-IN. HEIGHT

FIGURE 2 :
EXISTING TRAFFIC PATTERNS

1. Motorists passing in front of the Terminal Building are confronted with congestion created by the presence of taxis, buses, pedestrians, rental car check-ins (i.e. returns) and general pick-ups and deliveries.
2. Motorists traveling south on Yale Boulevard wishing to reach Girard Boulevard from which they can travel north to Gibson Boulevard can do so only by passing in front of the Terminal Building; hence, adding to the congestion in that area.
3. An alternative to Item 2 is that the motorist traveling south on Yale Boulevard must find a convenient place to turn around or negotiate a 'U'-turn and travel north on Yale Boulevard to the intersection with Gibson Boulevard, thus avoiding the Terminal Building congestion. The motorist can then negotiate a right turn and be heading toward the intersection of Girard Boulevard and Gibson Boulevard. A drawback to this is that the intersection of Yale Boulevard and Gibson Boulevard is extremely congested at the peak times of the day. Also, the route itself is less direct.
4. Motorists leaving the parking lots shown on Figure 2 and wishing to travel north on Girard Boulevard are confronted with the same problems addressed in Items 2 and 3.
5. The intersection of Girard Boulevard with the Service Road is awkward and potentially dangerous. This is because traffic entering and/or leaving the Service Road is in conflict with the traffic merging onto Girard

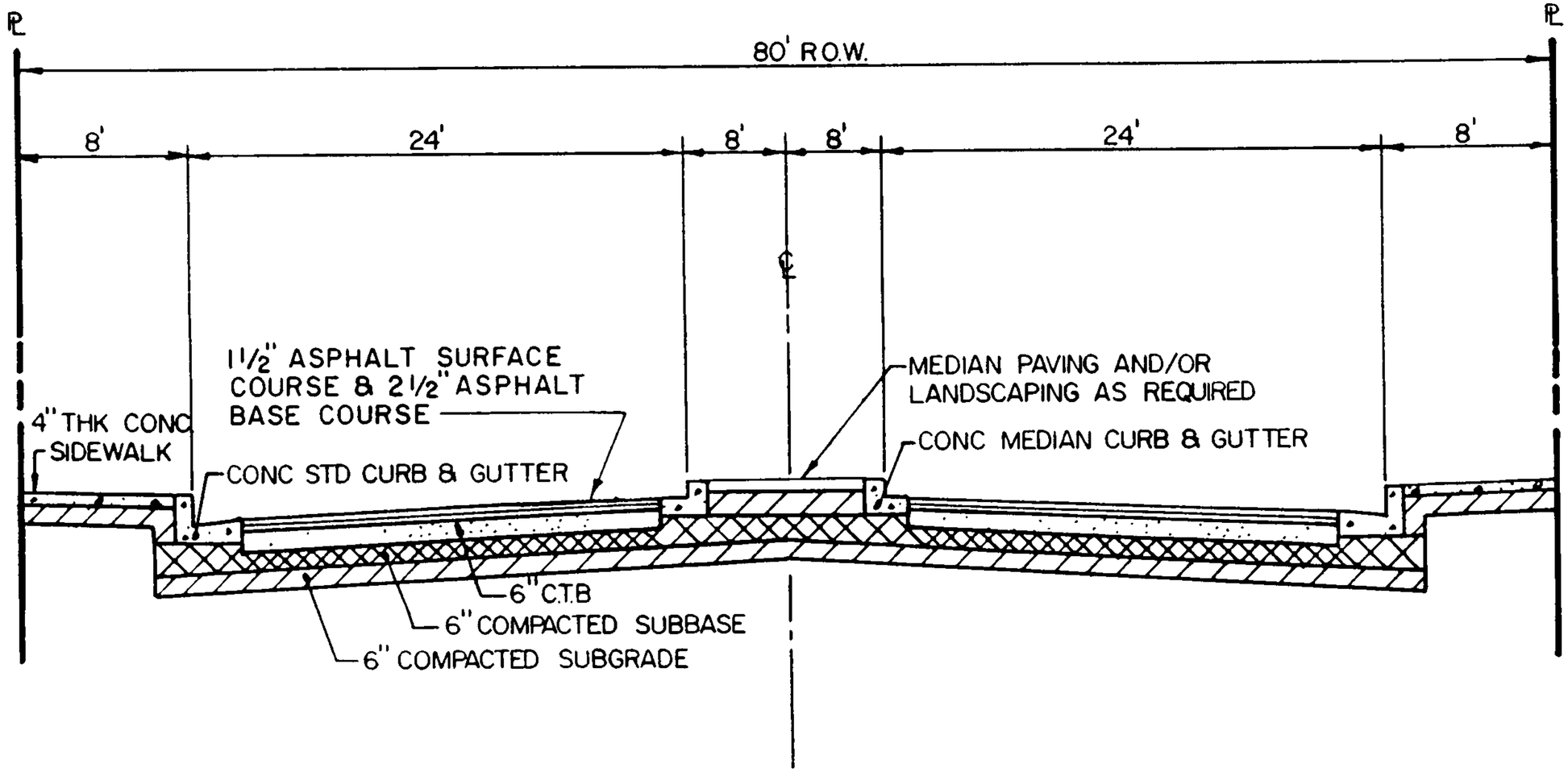
Boulevard from the Terminal area. This crossing motion is conducive to the occurrence of traffic accidents.

The problems listed above may appear insignificant at this time, however, as the growth of the Airport area continues, these problems will magnify and become much more significant to the public.

B. PROPOSED TRAFFIC PLAN

The magnitude of the existing traffic problems in and around the Airport will undoubtedly increase with time in direct proportion to the amount of growth experienced in the Airport area. It is because of this that a Traffic Plan is needed to accommodate the future growth in the Airport area. The plan presented herein is one which can be implemented in phases. The ultimate goal of the plan is to provide convenience for the users. This can be accomplished by (1) reducing congestion and (2) reducing the walking distances between the automobile parking and the Airport Terminal.

Congestion in front of the Terminal can be reduced by providing routes by which non-airport traffic can pass, yet avoid the Terminal area. The construction of a four-lane divided roadway connecting Yale Boulevard to Girard Boulevard will accomplish this. The alignment of this proposed roadway is shown on the Traffic Plan - Phase I, Figure 3. Figure 4 illustrates the typical street section proposed. The construction of this roadway will permit traffic to travel between



TYPICAL STREET SECTION
VERT. SCALE: 1"=60" HORIZ. SCALE: 1"=10'

FIGURE 4: TYPICAL STREET SECTION

Yale Boulevard and Girard Boulevard without passing by the Terminal Building.

The preliminary design of the proposed roadway mentioned above appears in Figure 5. As can be seen, the roadway is typically 64 feet face-to-face. A deviation from this is the portion of the roadway situated between Yale Boulevard and the Lower Parking Lot. Due to prior commitments by the City of Albuquerque, this section is restricted to 60 feet face-to-face. Figure 6 illustrates the approved roadway section for this portion of the proposed roadway. As shown by the preliminary design, a 400-foot transition is proposed to increase the roadway section from 60 feet face-to-face to the typical 64 feet face-to-face. In addition to illustrating the transition in roadway widths, the preliminary design demonstrates that the design/construction of the proposed roadway is feasible from a grade standpoint. Extensive grading and/or the construction of retaining walls will be necessary for the first 1,200 feet. This complication, however, does not present an insurmountable problem. It must be realized, however, that this design is merely preliminary and does not constitute precise engineering.

In a further effort to reduce congestion, minor revisions to the traffic pattern in front of the existing Terminal Building are proposed. These revisions provide for one lane of curb parking closest to the Terminal Building. The next lane to the north is a 'Drop-off/Thru Lane' to permit motorists to park or continue. The next lane to the north is exclusively

a 'Thru Lane'. This lane is bordered on the north by a 6-foot median intended to serve as a sidewalk for crossing pedestrians. A taxi stand is situated due north of the 6-foot median. The next two lanes to the north are 'Thru Lanes' with the northern most permitting entry to the Upper Parking Lot. Close comparison between Figure 2 and Figure 3 reveals that the major differences between the existing and proposed traffic patterns in front of the existing Terminal Building are as follows:

1. The Rental Car check-in area has been eliminated.
2. The single curb dividing the roadway has been removed and replaced by a 6-foot median, thus providing a 'rest stop' for pedestrians laden with baggage or delayed by traffic.
3. Two turnstiles have been located for easy entry into the Upper Parking Lot.

All lanes mentioned above should be a minimum 12 feet in width.

Phase I of the Traffic Plan also considers modifications to the parking lot. These modifications, as shown by Figure 3, require the following changes:

1. The relocation of the existing toll booth from the west side of the Upper Parking Lot to the east side of the lot. This will reduce congestion in front of the Terminal Building.
2. Two (2) exits are provided for the Upper Parking Lot proper in contrast to the one existing exit. This will accommodate the frequent movements of vehicles anticipated from that lot.

3. The parking and circulation pattern of the Upper Parking Lot has been modified to accommodate the two (2) entrances along the south side of the lot. The revisions are minor in that a herringbone pattern is maintained. The proposed orientation of vehicles is a herringbone pattern with 60-degree angle parking. The existing pattern is another variation of the herringbone pattern.
4. The existing entrance to the east side of the Upper Parking Lot has been omitted. The east entrance to the Lower Parking Lot, however, has been maintained.
5. The existing entrance along the north side of the Lower Parking Lot has been revised to accept eastbound traffic. At present, the entrance accepts westbound traffic.

It was pointed out earlier in this section that the existing intersection of Girard Boulevard with the Service Road is potentially dangerous. Because of this, the proposed Traffic Plan - Phase I, recommends the relocation of this intersection. The relocated intersection appears on Figure 3. The new Service Road connects the relocated intersection to the existing Service Road just south of the KAFB well. If security in this area is a concern, the Service Road can be controlled by the installation of an automatic security gate to prohibit unauthorized entry.

The improvements proposed above represent Phase I of the proposed Traffic Plan. Phase II assumes Terminal Building expansion, and hence, extends the traffic pattern revision in front of the Terminal Building. This concept is demonstrated

CITY OF ALBUQUERQUE
MUNICIPAL DEVELOPMENT DEPARTMENT
ENGINEERING DIVISION

FIGURE 5
GIRARD BOULEVARD, S.E. EXTENSION
PRELIMINARY DESIGN
PAVING IMPROVEMENTS PLAN AND PROFILE

APPROVALS	ENGINEER	DATE	ENGINEER	DATE
City Engineer			Liquid Waste	
A C E - Design			Traffic	
A C E - Hydrology			Water	

DRAWING NO. — SHEET 1 OF 1

APPROVED FOR CONSTRUCTION

PRELIMINARY

GIRARD BOULEVARD EXTENSION

VERT SCALE 1"=4'
HORIZ SCALE 1"=20'

EXISTING TOP OF CURB (LEFT) PROFILE

TOP OF LEFT CURB PROFILE

EXISTING GROUND PROFILE @ LEFT CURB

STA. 55+00 · STA. 54+00 · STA. 53+00 · STA. 52+00 · STA. 51+00 · STA. 50+00 · STA. 49+00 · STA. 48+00 · STA. 47+00 · STA. 46+00 · STA. 45+00 · STA. 44+00 · STA. 43+00 · STA. 42+00 · STA. 41+00 · STA. 40+00 · STA. 39+00 · STA. 38+00

CONNECT TO EXIST CURB
STA. 48+40 (RT)

CONSTRUCT VALLEY GUTTERS W/ WHEEL CHAIR RAMPS

NEW SERVICE ROAD

MATCH LINE STA. 34+50

CONSTRUCT 100' TURNOATS W/ 98' MEDIAN OPENING

ENTRANCE TO FAA FIELD MAINTENANCE YARD

STA. 55+45 GIRARD BLVD = ℄ GIBSON BLVD S.E.

GIBSON BLVD.

STA. 53+50 CONNECT TO EXIST MEDIAN

STA. 53+20 (LT) CONNECT TO EXIST CURB

8' SIDEWALK

16' MEDIAN

44'-0"

5310 · 5300 · 5290 · 5280 · 5270

RIGHT CURB · LEFT CURB

EXISTING GROUND PROFILE @ RIGHT CURB

TOP OF RIGHT CURB PROFILE

TOP OF LEFT CURB PROFILE

BREAK LINE

MATCH LINE STA. 34+50

PRELIMINARY

GIRARD BOULEVARD EXTENSION

VERT SCALE 1"=4'
HORIZ SCALE 1"=20'

EXISTING GROUND PROFILE @ LEFT CURB

TOP OF LEFT CURB PROFILE

EXISTING GROUND PROFILE @ RIGHT CURB

TOP OF RIGHT CURB PROFILE

BREAK LINE

STA. 10+00 · STA. 11+00 · STA. 12+00 · STA. 13+00 · STA. 14+00 · STA. 15+00 · STA. 16+00 · STA. 17+00 · STA. 18+00 · STA. 19+00 · STA. 20+00 · STA. 21+00 · STA. 22+00 · STA. 23+00 · STA. 24+00 · STA. 25+00 · STA. 26+00 · STA. 27+00 · STA. 28+00 · STA. 29+00 · STA. 30+00 · STA. 31+00 · STA. 32+00 · STA. 33+00 · STA. 34+00 · STA. 35+00

5290 · 5280 · 5270 · 5260 · 5250

RIGHT CURB · LEFT CURB

MATCH LINE STA. 34+50

PRELIMINARY

GIRARD BOULEVARD EXTENSION

VERT SCALE 1"=4'
HORIZ SCALE 1"=20'

STA. 10+00 · STA. 11+00 · STA. 12+00 · STA. 13+00 · STA. 14+00 · STA. 15+00 · STA. 16+00 · STA. 17+00 · STA. 18+00 · STA. 19+00 · STA. 20+00 · STA. 21+00 · STA. 22+00 · STA. 23+00 · STA. 24+00 · STA. 25+00 · STA. 26+00 · STA. 27+00 · STA. 28+00 · STA. 29+00 · STA. 30+00 · STA. 31+00 · STA. 32+00 · STA. 33+00 · STA. 34+00 · STA. 35+00

5250 · 5260 · 5270 · 5280 · 5290

RIGHT CURB · LEFT CURB

MATCH LINE STA. 34+50

FUEL STORAGE AREA

STA. 34+50

STA. 34+00

STA. 32+00 (INTERSECTION)

STA. 29+84

STA. 30+00

STA. 28+00

STA. 26+00

STA. 24+00

STA. 22+00

STA. 20+00

STA. 15+00

STA. 10+00

CONSTRUCT VALLEY GUTTER & WHEELCHAIR RAMPS

CONSTRUCT VALLEY GUTTER & WHEEL-CHAIR RAMP

CONSTRUCT 100' TURNOAT

CONSTRUCT DRIVE PADS

DOLLAR-A-DAY CAR RENTAL

BUDGET RENT-A-CAR

FAA FIELD MAINTENANCE

8' SIDEWALK

16' MEDIAN

40' MEDIAN OPENING

24' FF

36' SIDE-WALK

STA. 43+10 (INTERSECTION W/ 2' ℄ OFFSET)

ACCESS TO VACANT LEASE LAND

CONSTRUCT 100' TURNOAT

COMPLETE TRANSITION TO 64' FF ROAD SECTION

PAINTED MEDIAN OPENING

STA. 41+00 BEGIN TRANSITION FROM 64' TO 98' ROAD SECTION

PAINTED MEDIAN

STA. 38+00

STA. 40+00

STA. 42+00

STA. 44+00

98' MEDIAN OPENING CONSTRUCT MEDIAN NOSE

AAMCAF HOTEL

DRIVE PAD FOR AMFAC

40' MEDIAN OPENING

CONSTRUCT 100' TURNOAT

STA. 46+00

STA. 48+00

STA. 50+00

STA. 52+00

STA. 54+00

STA. 55+00

STA. 10+00 GIRARD BLVD S.E.

℄ YALE BLVD S.E.

8' SIDEWALK

CONSTRUCT VALLEY GUTTER

℄ WHEELCHAIR RAMPS

PARKING LOT

YALE BLVD.

64'-0"

98'-0"

CONSTRUCT 100' TURNOAT

5310 · 5300 · 5290 · 5280 · 5270

RIGHT CURB · LEFT CURB

MATCH LINE STA. 34+50

REFERENCES

NO.	DATE	REMARKS	BY

MAP NO. · EST. NO. · W.O. NO.

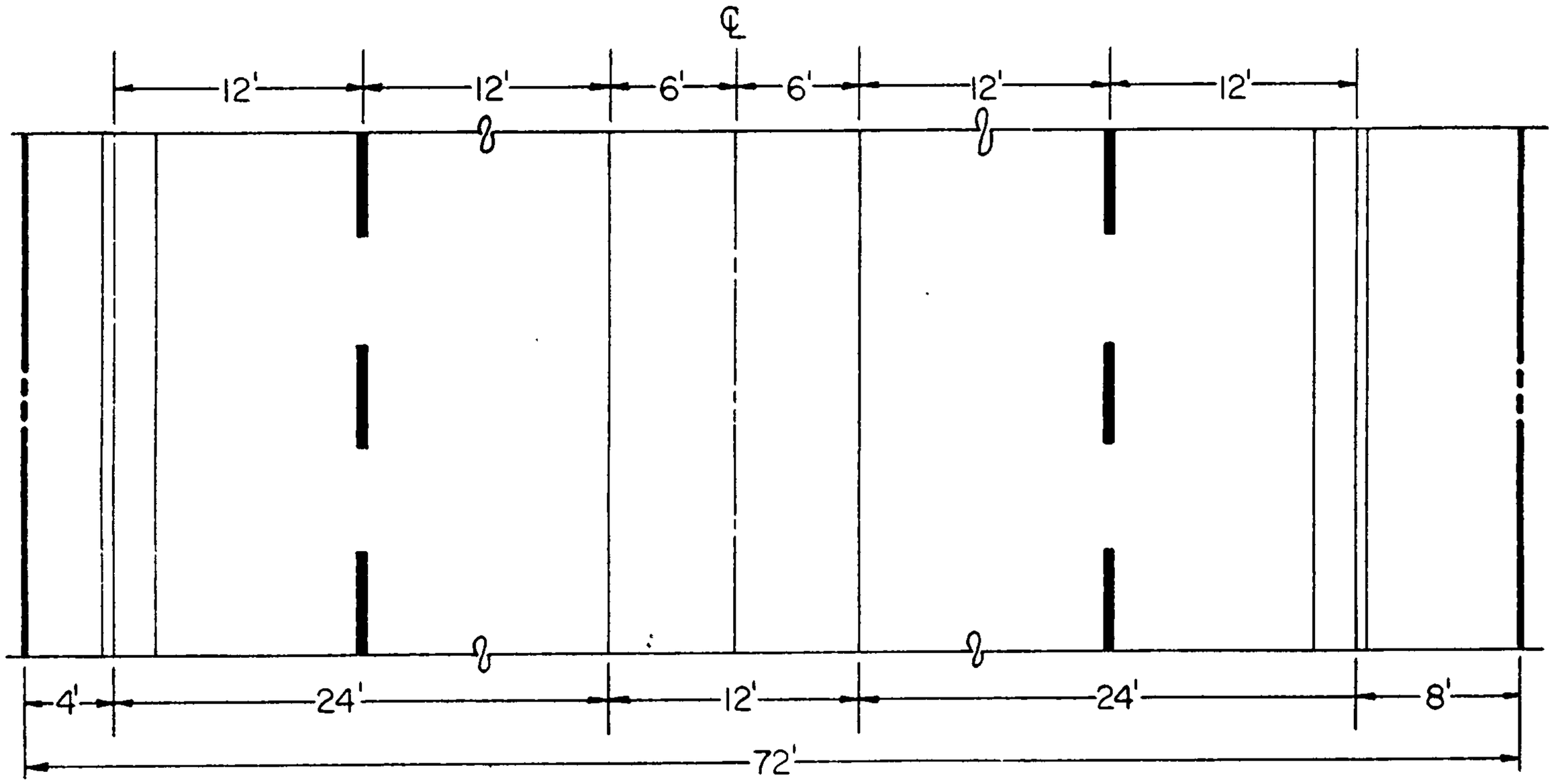
SURVEY INFORMATION

NO.	DATE	FIELD NOTES	BY

BENCH MARKS

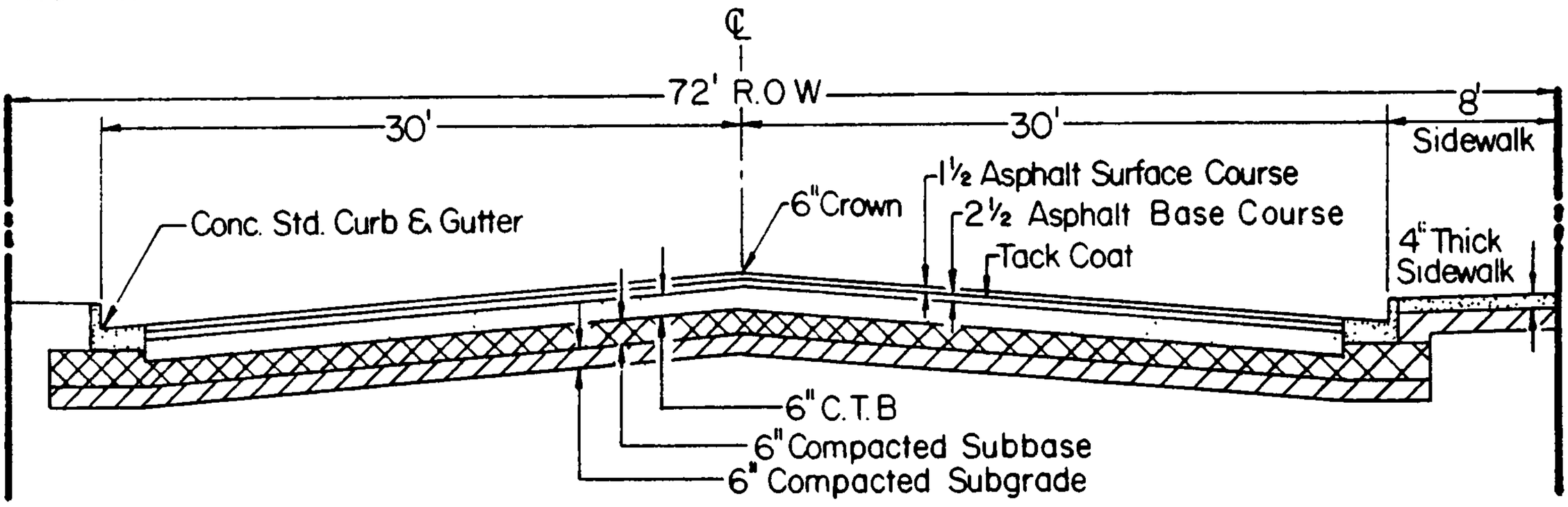
AS BUILT INFORMATION

CONTRACTOR
GRADING BEGUN
STAKED BY
INSPECTORS
FIELD PANEL BY
DATA
SUBSTANTIALLY COMPLETED BY
DATA
RECORDED BY
DATA
NO



NORTH

PLAN
1" = 50'



SECTION
HORIZ.: 1" = 50'
VERT. : 1" = 5'

FIGURE 6: 60' ROADWAY PLAN AND SECTION

GIBSON BLVD. S.E.

YALE BLVD. S.E.

MILES PLACE S.E.

COLUMBIA DRIVE

STANFORD DRIVE

ROAD S.E.

RENARD

BAYLOR DRIVE S.E.

COLUMBIA DRIVE S.E.

FUTURE COMMERCIAL

FUTURE COMMERCIAL

FUTURE COMMERCIAL

FUTURE COMMERCIAL

ALAMO AVENUE S.E.

RANDOLPH ROAD

GIRARD BLVD. S.E.

SIGN NOTES

A	TERMINAL BUILDING LEFT LANE
B	GIRARD BLVD. KEEP RIGHT
C	YALE BLVD. KEEP LEFT
D	TERMINAL BLDG KEEP LEFT — YALE BLVD KEEP RIGHT
E	THRU TRAFFIC STRAIGHT — TERMINAL BLDG RIGHT TURN
F	TERMINAL BLDG. RIGHT TURN
G	YALE BLVD. ←
UP	UPPER PARKING LOT KEEP LEFT
LP	LOWER PARKING LOT KEEP LEFT
S	STOP
Y	YIELD

SCALE 1" = 200'

KEYED NOTES

1	CONSTRUCT 100' TURN LANE
2	CONSTRUCT 98' MEDIAN OPENING
3	PROPOSED ROADWAY; FOR FURTHER DETAIL SEE PRELIMINARY DESIGN
4	CONSTRUCT 8' SIDEWALK
5	TWA CHECK-IN
6	REMOVE EXISTING TURNSTILE AND RELOCATE TO ACCEPT EASTBOUND TRAFFIC
7	ACCESS TO FUTURE COMMERCIAL
8	CONSTRUCT 32' WIDE SERVICE ROAD
9	EXISTING TURNSTILE TO REMAIN
10	CONNECT SERVICE ROAD TO AVIATION FACILITY EXPANSION
11	EXIT FROM LOWER PARKING LOT
12	REMOVE EXISTING TOLL BOOTH
13	EXIT FROM UPPER LOT
14	CONSTRUCT NEW 2 LANE TOLL BOOTH
15	90° PARKING
16	24' WIDE ASILE
17	60° ANGLE PARKING
18	INSTALL TURNSTILE
19	PEDESTRIAN CROSSWALK TO REMAIN
20	PAINTED MEDIAN
21	MEDIAN CURB

MATCH FIGURE 7 b

FIGURE 7a : TRAFFIC PLAN PHASE II

MATCH FIGURE 7a

FUTURE COMMERCIAL

BUDGET RENT-A-CAR

DOLLAR A DAY CAR RENTAL

F.A.A. FIELD MAINTENANCE

FUEL STORAGE

AMFAC

FUTURE COMMERCIAL

LOWER LOT PARKING

UPPER LOT PARKING

FUTURE EXPANSION OF AVIATION FACILITIES

YALE BLVD. S.E.

SUNPORT BLVD. S.E.

TERMINAL BUILDING

POST OFFICE

EMERGENCY POWER

AIR CARGO

AIR FREIGHT BUILDING

MUSEUM

VIEWPOINT ROAD

SCALE

LEGEND

5280 EXISTING CONTOUR

X 122 EXISTING SPOT ELEVATION

DIRECTION OF TRAFFIC FLOW

TRAFFIC CONTROL SIGNS

FIGURE 7b :
TRAFFIC PLAN – PHASE II

by Figure 7. Many of the items discussed for Phase I apply for Phase II. A minor difference between the two phases is that the new Service Road will become shorter in length. In Phase II, however, the new Service Road will provide access to and from the expanded facilities.

Traffic control in both phases will be accomplished by the installation of signs and traffic signals, and the painting of directional markers and lanes on the road surface. Suggested locations for traffic signals and traffic control signs are identified by keyed notes on Figures 3 and 7. Suggested sign headings are also shown on Figures 3 and 7.

A third phase of the proposed Traffic Plan involves the construction of a parking structure on the site of the Upper Parking Lot. Although not illustrated by a figure, one can easily visualize the location of the proposed structure on Figure 3 or Figure 7. The construction of such facility would accomplish two goals. These goals are (1) minimizing the walking distance between automobile parking and the Terminal Building and (2) maximizing the efficiency in which Airport land is utilized. The first goal is self explanatory. The second goal is achieved by freeing more space for commercial development. With the construction of a parking structure, the Lower Parking Lot will no longer be needed, hence, it can become available lease space.

The construction of the parking structure mentioned above will also serve to reduce congestion in front of the Terminal Building by providing overhead pedestrian access between the parking structure and the Terminal Building. The construction

of a pedestrian bridge between the parking structure and Terminal Building will eliminate the need for pedestrians to utilize the pedestrian cross-walks shown on Figures 3 and 7, hence eliminating one of the contributing factors to congestion in front of the Terminal Building. At the time of design of such a structure, the appropriate number of pedestrian bridges needed must be determined. This will be a function of Terminal layout, parking structure layout and anticipated use.

IV. DRAINAGE

The purpose of this section is to review and discuss the existing drainage conditions in the study area. Following the discussion of the existing drainage conditions, two (2) plans are presented. The first plan provides recommendations for solving the existing drainage problems. The second plan demonstrates a generalized treatment of the runoff anticipated from future expansion and development. These two (2) plans coincide with Phase I and II of the Traffic Plan.

A. EXISTING DRAINAGE CONDITIONS

The existing drainage basins and facilities in the Airport area are shown by Figure 8. The drainage basin boundaries were determined from the existing topographic information provided by Koogle & Pouls Engineering and supplemented by numerous field visits. The existing storm drainage facilities, such as catch basins, manholes and culverts, were researched from Airport records, located and identified in the field, and presented in Tables 1 and 2. Table 1 is an inventory of existing catch basins and Table 2 is an inventory of the existing storm drains. These inventories identify the particular facility, quantify the flows carried by the facility and quantify the capacity of the facility.

The existing drainage conditions for the Airport area were analyzed by the Soil Conservation Service (SCS) Curve Number Method. For convenience in analysis and presentation of results, the method was computerized. The output and an explanation

thereof appear in Appendix A. Table 3 presents a summary of the ground cover conditions and corresponding curve numbers for the study area. Figure 9 is the soil map which supports Table 3.

Each drainage basin was analyzed individually to determine the 10- and 100-year peak flows and runoff volumes associated with the basin. A summary of these results appear in Table 4. Table 5 summarizes the 10- and 100-year peak flows and runoff volumes for composite drainage basins. The analysis of the composite drainage basins enabled the determination of the peak flows contributing to the storm drain system. This analysis simulates the construction of composite hydrographs, thereby eliminating the need to calculate basin lag times.

Table 1, as previously stated, is an inventory of existing catch basins. The table describes (1) the type of catch basin (CB); (2) the location of the catch basin, whether in a low-point, sag-point or along a curb line; (3) the peak flows reaching the catch basin; and (4) the capacity of the catch basin. Items 1 and 2 mentioned above were determined in the field. Item 3 was determined from Table 4. Item 4 was determined from the calculations appearing in Appendix B. Comparing Items 3 and 4 enables the evaluation of the catch basin size. In all cases, except those in which insufficient data exists, the catch basins appear adequate. The evaluation assumes, however, that the pipe draining from the catch basin was adequately designed and built, and that no clogging occurs at the inlet of the catch basin. It must be kept in mind, however, that the deficient sizing of the outlet pipe from a catch basin

the deficient sizing of the outlet pipe from a catch basin and/or the clogging of the inlet may cause flooding.

Table 2 is an inventory of the existing storm drains in the study area. The inventory traces the existing system from its uppermost reaches to the Albuquerque Metropolitan Arroyo Control Authority (AMAFCA) Channel located just west of the intersection of Yale Boulevard S.E. and Alamo Boulevard S.E. The inventory analyzes the storm drain system between manholes designated by 'MH' and/or catch basins designated by 'CB'. The inventory describes pipe type, diameter, length and slope, which have been determined by combining as-built records, current survey data and engineering judgment. The contributing flows were determined from Table 5. The pipe flows were obtained by comparing pipe capacity and contributing flow, and selecting the lower value. The pipe capacity, flow depth and flow velocity were determined by the Hazen-Williams equation for uniform culvert flow. The Hazen-Williams equation for uniform flow is as follows:

$$Q = 1.318\ ACR^{0.63}s^{0.54}$$

where Q = flow in cfs,

A = flow cross-sectional area in square feet

C = Hazen-Williams friction coefficient, dimensionless

R = hydraulic radius, in feet, which is the ratio of the flow corss-sectional area to the wetted perimeter of the flow cross section

s = energy grade line slope (which is assumed equal to the slope of the pipe) expressed as a decimal

Values of 'C' used in this analysis are 120 for reinforced concrete pipe (RCP) and 80 for corrugated metal pipe (CMP). These 'C' values assume that all culverts are 50 years in age.

The analysis of the storm drain system in Yale Boulevard S.E. assumes flow in only one-half (½) of the street for collection by catch basins. The analysis of the storm drain system in Alamo Avenue S.E. assumes equal flow in either gutter. Combining the flows from these two streets, with the flows from the Airport proper, was achieved by merely adding peak flows. This not only simplified the analysis, but added safety as well. The actual flows anticipated at the junctions will be somewhat less than those estimated by the technique described above due to basin lagging.

The majority of storm water runoff generated in the Airport area, if not intercepted by the existing storm drainage systems, flows in the streets. Because of this, the existing streets were analyzed at several locations to see if the anticipated flows were contained within the street. This was accomplished by using the Manning equation:

$$Q = \frac{1.49}{n} AR^{2/3} s^{1/2}$$

where Q = flow in cfs

n = Manning roughness coefficient (assumed to be 0.020 for this study), dimensionless.

A = Flow cross-sectional area in square feet.

R = Hydraulic radius, in feet, which is the ratio of the flow cross-sectional area to the wetted perimeter of the flow cross section.

s = Energy grade line slope (which is assumed equal to the slope of the street) expressed as a decimal.

The results of this analysis are shown in Table 6. In all cases the anticipated flows stay well within the paved street.

As illustrated by Figure 8, Basins 1, 2, 3, 4, 5, 6, 7, 8, 11, 12, 13 and 14 drain to an AMAFCA channel. The dimensions and capacity of this channel are shown by Figure 10. Basins 9 and 10, however, discharge approximately 2.2 cfs into a small arroyo located on undeveloped Airport land situated on the west side of Yale Boulevard approximately 750 feet north of Viewpoint Road.

There are several problem areas with respect to the existing drainage conditions. These problem areas are summarized as follows:

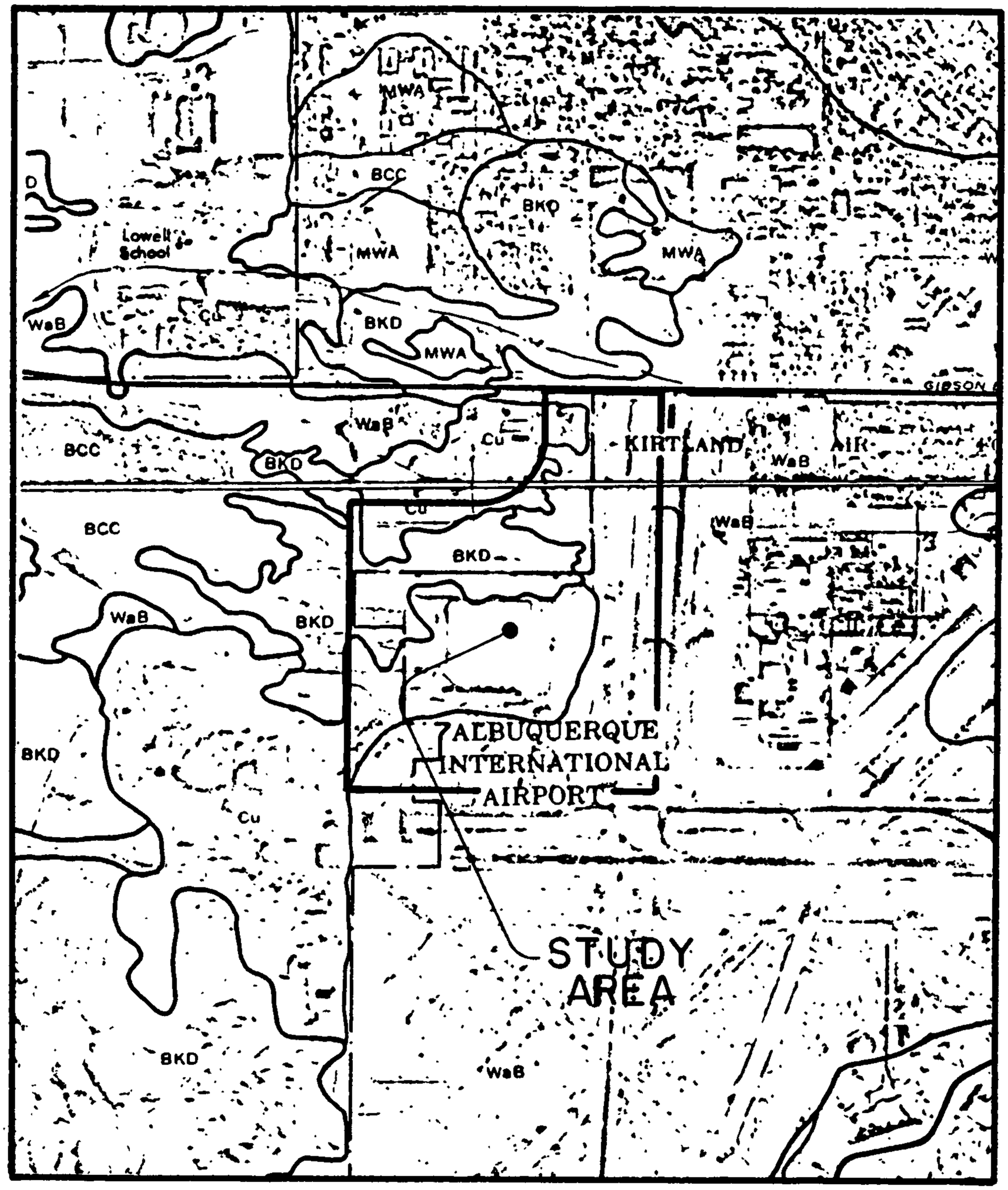
1. The portion of Basin 2 leased by Avis Rent-A-Car drains westerly toward CB-11, however, an existing block wall interrupts this flow. Because of this, storm water runoff has undermined the wall at the location denoted by keyed Note 3. A 6-inch drain line has been installed in the corner, but does not appear to be functioning adequately. The sand and gravel mobilized by the undermining of the wall have been transported to CB-11 and have significantly clogged the grate.

On East side of parking lots.

2. CB-4, CB-5 and CB-8 have been found to be full of debris. This debris ranges from tumbleweeds to beer cans which obviously obstruct the flow into the respective catch bains.

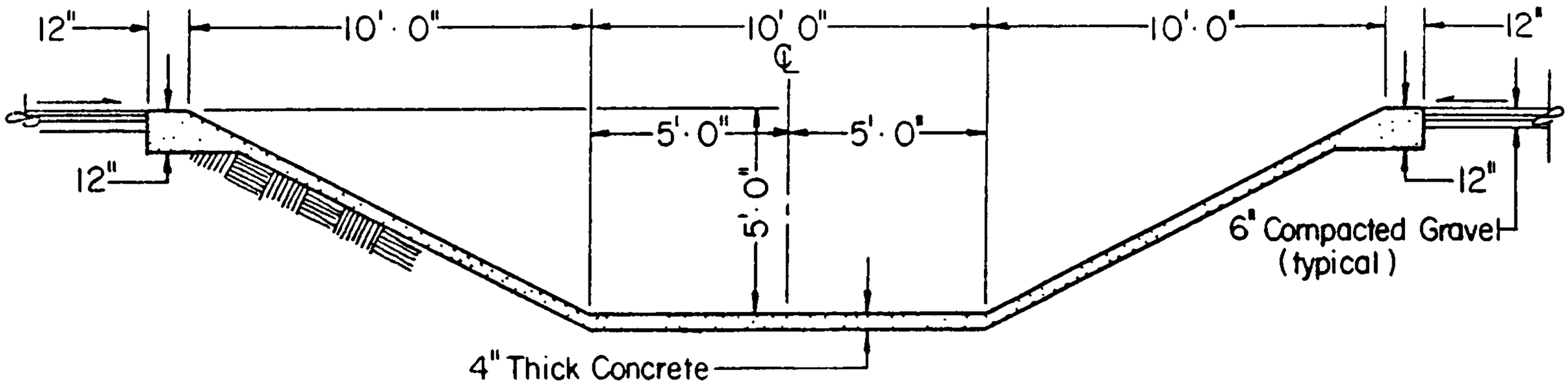
near terminal building

3. CB-3 is located in Basin 10. Due to the very low curve number associated with this basin, runoff is believed to be negligible for both the 10- and 100-year storms.



Scale: 1" = 2,000'

FIGURE 9: SOIL MAP FOR STUDY AREA



SECTION

CHANNEL ANALYSIS:

By Manning Equation, $Q = \frac{1.49}{n} AR^{2/3}s^{1/2}$,

where $n = 0.015$

$A = 100$ sf

$P = 32.36'$

$R = \frac{A}{P} = 3.09'$

$s = 0.0144$ (min.)

$Q_{Max} = \frac{1.49}{0.015}(100)(3.09)^{2/3}(0.0144)^{1/2} = 2522$ cfs

$Q_{100,\ dev.} = 330.8$ cfs $\Rightarrow$ d = 1.73'

FIGURE 10: AMAFCA CHANNEL SECTION AND ANALYSIS

Despite this, a catch basin, CB-3, has been located at the low point of the basin. High grass was observed partially choking the inlet to the catch basin. The choking effect of the high grass could be a problem if the landscaped area experienced an irrigation failure.

4. The slope of the 27-inch RCP between MH-23 and MH-25 is relatively flat, about one percent (1%). Due to the flatness of the slope, this portion of the storm drain system does not have the capacity to carry the 100-year peak flow reaching that point. Because of this, the runoff reaching CB-9 and CB-10, if exceeding 37 cfs, will "back-up" and overflow into Basin 7. The analysis formed in Table 2 demonstrates that the storm drain facilities in Basin 7 can handle this overflow. The problem mentioned above is only anticipated for the 100-year peak flow. The results of Table 2 demonstrate that this portion of the storm drain system can adequately handle the 10-year peak flow.

5. The rim and cover for MH-26 are broken. It appears that they have been sheared from the uppermost concrete barrel section of the manhole. The lack of an adequate cover on this manhole could permit large objects to enter the storm drain system; hence significantly reducing the capacity of the system.

6. The grate for CB-17 is missing. The lack of grate on this catch basin could permit large objects to enter the storm

drain system; hence, significantly reducing the capacity of the system.

7. The 35-foot drainage and public utility easement between the outlet of the 36-inch RCP and the inlet to the 60-inch RCP is intended to serve as a drainage channel or swale. This area is really a pond which must fill to a depth of approximately two feet (2') before it will spill into the 60-inch pipe. Because of this, the water which lies below the invert of the 60-inch pipe will not drain, but rather be left to infiltrate and/or evaporate. In general, the area is an eyesore.

South of Caddell Develop.

8. Basins 9 and 10, as previously mentioned, discharge via a 30-inch CMP culvert into a small arroyo. This point of discharge does not lie within a drainage easement, nor does it connect to a conveyance structure by which runoff can reach an outfall.

West of Terminal Building

9. Two (2) existing 24-inch RCP highway culverts are located near the F.A.A. Field Maintenance yard. These culverts, shown on Figure 8, discharge onto undeveloped non-Airport land where neither a drainage easement nor a drainage conveyance structure exist.

Basin B

10. A soil erosion problem exists along the north boundary of the Airport land. This problem is most severe along the north side of Budget Rent-A-Car where a 1:1 slope was created to construct that facility. This 1:1 slope appears to be unstable. The slope is less severe, approximately 3.5:1, along the north side of the F.A.A. Field Maintenance yard. This slope is being eroded by runoff

generated in the F.A.A. Field Maintenance yard and allowed to discharge to the north. This runoff discharges onto non-Airport land.

B. PROPOSED DRAINAGE CONDITIONS

The proposed Drainage Plan for the Airport area has been developed by assuming future land uses. The Drainage Plan, shown by Figure 11, illustrates the fully developed state. Drainage basin boundaries were determined on the basis of existing topographic information and anticipated land use. Existing and proposed storm drainage facilities are also shown on Figure 11.

See comment relating to previous section.

The proposed drainage conditions for the Airport area were analyzed by the SCS Curve Number method. Ground cover conditions and curve numbers were obtained from Table 3. Areas denoted as "FUTURE COMMERCIAL" were assumed to be 100 percent impervious (i.e. CN = 95).

Each drainage basin was analyzed individually to determine the 10- and 100-year peak flows and runoff volumes associated with the basin. A summary of these results appear in Table 7. Table 7 also summarizes the 10- and 100-year peak flows and runoff volumes for composite drainage basins. The analysis of the composite drainage basins enabled the determination of the peak flows contributing to the storm drain system. The basis for this analysis was discussed earlier in this section.

The Drainage Plan is designed to be implemented in phases. These phases coincide with Phases I and II of the previously discussed Traffic Plan. Both phases of the Drainage Plan

recommend the construction of catch basins and storm drain lines. Table 8 is a summary of the proposed catch basins in Phase I. Table 9 is a summary of the proposed catch basins in Phase II. Tables 10 and 11 summarize storm drain construction for Phases I and II, respectively. Catch basins have been sized in accordance with the calculations appearing in Appendix B. Proposed and existing storm drains have been analyzed for the fully developed condition by the Hazen-Williams equation which was discussed earlier in this section. 'C' values were estimated conservatively as 120. Design flows in all phases are based on the 100-year peak flow.

The ability of the streets to carry runoff has been analyzed for the fully developed condition. This analysis utilizes the Manning equation as discussed earlier in this section. As shown by the results of the analysis, which appear in Table 12, the 100-year peak flows for the fully developed condition will stay within the limits of paving of each street. It must be kept in mind, however, that this analysis does not account for the presence of catch basins and storm drains.

Phase I of the Drainage Plan consists of the following items:

1. Construct CB-70, a single 'D' catch basin, at the low point along the westerly boundary of Avis Rent-A-Car where existing runoff has undermined the wall. This catch basin should be connected to CB-11 with approximately 30 lineal feet of 18-inch RCP installed at a one percent (1%) minimum slope. At the time of connection, the sand and gravel clogging CB-11 should be removed.

OK on E side of parking lot near N boundary of Future Terminal Building Expansion Area

2. CB-4, CB-5 and CB-8 should be thoroughly cleaned and a routine program of maintenance be established to ensure that all of the catch basins in the Airport area remain clean and in good repair.

3. Construct a 24-inch RCP storm drain parallel to the existing 24-inch RCP storm drain between MH-27 and MH-29. An alternate to this, Alternate B, is to abandon the existing 24-inch RCP between MH-27 and CB-17 and replace it with a new 36-inch RCP storm drain. The 36-inch RCP storm drain will connect to MH-30 and lie within the proposed Girard Boulevard extension. These alternates are described in more detail in Table 10.

4. Construct a 42-inch RCP storm drain between the existing 36-inch RCP line and the existing 60-inch RCP line within the 35-foot drainage and public utility easement shown on Figure 11b. This will not only serve to contain the storm water flowing into that easement, but will also eliminate the unsightliness of the easement.

5. Construct catch basins CB-62 through CB-65 at the sag-points in the proposed Girard Boulevard extension within Basin 8. These catch basins are connected by a 24-inch RCP storm drain which flows to the 35-foot drainage and public utility easement and connects to the existing 60-inch RCP line. This connection is shown on Figure 11b and described in more detail in Table 10. In conjunction with the construction of the 24-inch RCP storm drain across Basin 15, an underground drainage easement should be

established. Also, all runoff generated by the development of Basin 15 should be routed into the proposed storm drain system. This can be accomplished by constructing an adequately sized catch basin at the low-point of the site. CB-69 has been shown on Figure 11b merely as an example of such a location.

6. Construct catch basins CB-60 and CB-61 at the sag-points in the proposed Girard Boulevard extension at the inter-face between Basin 5 and Basin 13. These catch basins connect to MH-30 with 18-inch RCP at a two percent (2%) minimum slope.

7. Remove and relocate CB-16 as CB-16R to accommodate the widening of the adjacent roadway.

8. Construct approximately 450 lineal feet of 24-inch RCP storm drain between MH-42 and MH-43, and abandon the existing 30-inch CMP draining west from MH-43. This will enhance the development of the Airport land located north of Viewpoint Road and west of Yale Boulevard as well as eliminating drainage problems with such development.

9. The two (2) existing 24-inch RCP highway culverts located near the F.A.A. Field Maintenance Yard should be abandoned and/or removed. The areas adjacent to these culverts should be graded so that runoff will drain to Girard Boulevard. This will eliminate the discharge of runoff from Airport land onto non-Airport land.

10. The soil erosion problem along the north boundary of the Airport land should be abated. Solutions to this problem

are (1) the construction of retaining wall, (2) covering the slope with an impervious slope covering or (3) constructing a combination of retaining walls and slope covering.

11. Extend the existing storm drain in Alamo Avenue further upstream to accept and convey the storm water runoff anticipated by future development. This will decrease the depth of runoff which flows in Alamo Avenue and reaches the sag-point just east of the intersection with Yale Boulevard. This extension is shown on Figure 11a shown on Figure 11a and described in Table 10.

Phase II of the Traffic Plan concerns the future expansion of the Terminal Building and associated aviation facilities. Likewise, Phase II of the Drainage Plan considers the same expansion. The Drainage Plan shown by Figure 11b considers that the roof, or roofs, of the Terminal Building expansion drain to the street in front of the Terminal, in other words, Basin 4. The future aviation facilities, anticipated to be primarily apron space, will need to be relatively flat. Because of this, a series of catch basins and storm drains have been proposed to handle the anticipated runoff. The proposed storm drain system itself is described in more detail in Table 11.

CB	TYPE	BASIN	LOCATION	CAPACITY (CFS)	Q_{10} (CFS)	Q_{100} (CFS)	EVALUATION	COMMENTS
1	Double "C"	9	Sag-Point	28.8	<0.7	<2.2	Satisfactory	None
2	Double "C"	9	Sag-Point	28.8	<0.7	<2.2	Satisfactory	None
3	5.5' x 1' Weir	10	Lo-Point	18.0	~0.0	~0.0	Satisfactory	Opening partially clogged by high grass.
4	3' x 0.5' Weir	5	Curb-line	3.5	0.3	0.5	Satisfactory	Full of debris. Flows determined by areal proportion of Basin.
5	3' x 0.5' Weir	5	Curb-line	3.5	0.3	0.6	Satisfactory	Full of debris. Flows determined by areal proportion of Basin.
6	3' x 0.5' Weir	4	Curb-line	3.5	0.6	1.1	Satisfactory	Flows determined by areal proportion of Basin.
7	2' x 3' Grate	4	Lo-Point	5.0	0.6	1.0	Satisfactory	Flows determined by areal proportion of Basin.
8	3' x 0.5' Weir	4	Curb-line	3.5	0.7	1.2	Satisfactory	Full of debris. Flows determined by areal proportion of Basin.
9	7' x 0.5' Weir	6	Sag-Point	16.4	<11.4	<19.2	Satisfactory	Capacity estimated by orifice analysis assuming C_v = 0.9 & flooding to top-of-curb. Flow restricted @ CB-9 & CB-10 by reduced capacity of adjoining storm drain. Overflow spills into Basin 7.

TABLE 1: INVENTORY OF EXISTING CATCH BASINS

CB	TYPE	BASIN	LOCATION	CAPACITY (CFS)	Q_{10} (CFS)	Q_{100} (CFS)	EVALUATION	COMMENTS
10	7' x 0.5' Weir	6	Sag-Point	16.4	<11.4	<19.2	Satisfactory	Capacity estimated by orifice analysis assuming C_v = 0.9 & flooding to top-of-curb. Flow restricted @ CB-9 & CB-10 by reduced capacity of adjoining storm drain. Overflow spills into Basin 7.
11	Double "C"	1, 2, 4	Sag-Point	28.8	<14.2	<28.6	Satisfactory	Clogged w/sand & gravel.
12	Double "C"	1, 2, 4	Sag-Point	28.8	<14.2	<28.6	Satisfactory	None
13	Single "C"	1	Curb-line	5.0	<1.7	<6.7	Satisfactory	Remaining discharge flows to CB-15.
14	?	1	?	?	?	?	None	Can not locate.
15	2 Double "C"'s	5, 13	Sag-Point	57.6	6.7	13.7	Satisfactory	None
16	2 Double "C"'s	7	Lo-Point	57.6	19.1	34.9	Satisfactory	None
17	Single "D"	8	Slope	?	?	?	None	Grate missing. Serves as junction box.
18	2' x 2' Grate	12	Lo-Point	7.6	3.2	5.4	Satisfactory	None
19	3' x 3' Grate	3	Lo-Point	12.0	4.0	6.8	Satisfactory	Flows determined by areal proportion of Basin.

TABLE 1: INVENTORY OF EXISTING CATCH BASINS (continued)

CB	TYPE	BASIN	LOCATION	CAPACITY (CFS)	Q_{10} (CFS)	Q_{100} (CFS)	EVALUATION	COMMENTS
20	2' x 2' Grate	1A	Lo-Point	3.4	0.9	1.5	Satisfactory	Flows determined by areal proportion of Basin.
21	2' x 3' Grate	1A	Lo-Point	5.0	1.5	2.6	Satisfactory	Flows determined by areal proportion of Basin.
22	2' x 3' Grate	1A	Lo-Point	5.0	0.7	1.2	Satisfactory	Flows determined by areal proportion of Basin.
23	2' x 3' Grate	3	Lo-Point	5.0	0.7	1.1	Satisfactory	Flows determined by areal proportion of Basin.
24	Grate	3	Lo-Point	?	<13.7	<23.4	None	Size unknown; located behind security gate.
25	Grate	3	Lo-Point	?	<13.7	<23.4	None	Size unknown; located behind security gate.
26	Double "C"	11	Curb-line	10.0	<9.8	<22.1	Satisfactory	Collects portions of curb line flows. Some overflow expected.
27	Double "C"	11	Curb-line	10.0	<9.8	<22.1	Satisfactory	Collects portions of curb line flows. Some overflow expected.
28	Double "C"	11	Curb-line	10.0	<9.8	<22.1	Satisfactory	Collects portions of curb line flows. Some overflow expected.
29	Double "C"	11	Curb-line	10.0	<9.8	<22.1	Satisfactory	Collects portions of curb line flows. Some overflow expected.
30	Double "C"	11	Curb-line	10.0	<9.8	<22.1	Satisfactory	Collects portions of curb line flows. Some overflow expected.

TABLE 1: INVENTORY OF EXISTING CATCH BASINS (continued)

CB	TYPE	BASIN	LOCATION	CAPACITY (CFS)	Q_{10} (CFS)	Q_{100} (CFS)	EVALUATION	COMMENTS
31	Double "C"	11	Curb-line	10.0	<9.8	<22.1	Satisfactory	Collects portions of curb line flows. Some overflow expected.
32	2' x 24' Cattle Guard	11	Transverse to Traffic	30.0	<9.8	<22.1	Satisfactory	Collect runoff not collected by upstream catch basins.
33	2' x 36' Cattle Guard	11	Transverse to Traffic	45.0	<9.8	<22.1	Satisfactory	Collect runoff not collected by upstream catch basins.
34	Double "C"	14	Curb-line	10.0	<20.4	<52.5	Satisfactory	Collects portions of curb line flows. Some overflow expected.
35	Double "C"	14	Curb-line	10.0	<20.4	<52.5	Satisfactory	Collects portions of curb line flows. Some overflow expected.
36	Double "C"	14	Curb-line	10.0	<20.4	<52.5	Satisfactory	Collects portions of curb line flows. Some overflow expected.
37	Double "C"	14	Curb-line	10.0	<20.4	<52.5	Satisfactory	Collects portions of curb line flows. Some overflow expected.
38	Double "C"	14	Curb-line	10.0	<20.4	<52.5	Satisfactory	Collects portions of curb line flows. Some overflow expected.
39	Double "C"	14	Curb-line	10.0	<20.4	<52.5	Satisfactory	Collects portions of curb line flows. Some overflow expected.
40	2 Double "C"'s	14	Sag-Point	57.6	<20.4	<52.5	Satisfactory	Collect runoff not collected by upstream catch basins.

TABLE 1: INVENTORY OF EXISTING CATCH BASINS (continued)

CB	TYPE	BASIN	LOCATION	CAPACITY (CFS)	Q_{10} (CFS)	Q_{100} (CFS)	EVALUATION	COMMENTS
41	Double "C"	14	Sag-Point	28.8	<	<	Satisfactory	Collect runoff not collected by upstream catch basins.
42	Single "D"	14	Lo-Point	14.4	0.8	1.4	Satisfactory	Located on private property. Flows estimated.
43	Single "D"	14	Lo-Point	5.0	0.7	1.2	Satisfactory	Located on private property. Flows estimated.

TABLE 1: INVENTORY OF EXISTING CATCH BASINS (continued)

FROM	TO	PIPE DIA. (IN)	TYPE OF PIPE	APPROX. LENGTH (FT)	SLOPE (FT/FT)	EXIST. CAPACITY (CFS)	CONTRIB. FLOW (CFS)		PIPE FLOW (CFS)		ACTUAL DEPTH (FT)		FLOW VELOCITY (FPS)		CONTRIBUTING FLOW
							Q_{10}	Q_{100}	Q_{10}	Q_{100}	10	100	10	100	
MH-19	MH-20	24	RCP	220	0.02[e]	42	17.5	29.8	17.5	29.8	0.9	1.3	12.0	13.5	Basins 1A & 3
MH-20	MH-21	24	RCP	230	0.022[s]	44	19.8	34.0	19.8	34.0	1.0	1.4	13.0	14.5	Basins 1A, 3, Part 4 & Part 5
MH-22	MH-21	18	RCP	250	0.025[s]	22	1.6	2.7	1.6	2.7	0.3	0.4	7.0	8.2	Part of Basin 5
MH-21	MH-23	24	RCP	240	0.046[s]	65	21.2	36.5	21.2	36.5	0.8	1.1	17.7	20.3	Basins 1A, 3, Part 4 & Part 5
MH-23	MH-25	27	RCP	280	0.009[s]	37	30.6	52.0	30.6	37.0	1.6	2.1	9.7	9.6	Basins 1A, 3, 4, Part 5 & Part 6
MH-24	MH-25	24	RCP	420	0.022[s]	44	10.2	25.4	10.2	25.4	0.7	1.1	10.9	13.7	Basins 1, 2 & 4
MH-25	MH-30	36	RCP	550	0.033[s]	158	30.7	63.4	30.7	63.4	0.9	1.4	16.7	20.1	Basins 1, 2, 3, 4, Part 5 & Part 6
MH-30	CB-17	36	RCP	90	0.08[s]	255	43.3	90.0	43.3	90.0	0.9	1.3	25.9	31.5	Basins 1, 2, 3, 4, 5, 6, 7 & 13
MH-27	MH-26	24	RCP	285	0.022[e]	44	1.0	5.0	1.0	5.0	0.2	0.5	5.6	8.9	Part of Basin 1 (Flows estimated)
MH-26	MH-28	24	RCP	420	0.022[s]	44	1.0	5.0	1.0	5.0	0.2	0.5	5.6	8.9	Part of Basin 1 (Flows estimated)

TABLE 2: INVENTORY OF EXISTING STORM DRAINS

FROM	TO	PIPE DIA. (IN)	TYPE OF PIPE	APPROX. LENGTH (FT)	SLOPE (FT/FT)	EXIST. CAPACITY (CFS)	CONTRIB. FLOW (CFS)		PIPE FLOW (CFS)		ACTUAL DEPTH (FT)		FLOW VELOCITY (FPS)		CONTRIBUTING FLOW
							Q_{10}	Q_{100}	Q_{10}	Q_{100}	10	100	10	100	
CB-18	MH-28	12	RCP	230	0.02[s]	7	3.2	5.4	3.2	5.4	0.5	0.7	8.0	9.0	Basin 12
MH-28	MH-29	24	RCP	60	0.022[s]	44	4.2	10.4	4.2	10.4	0.4	0.7	8.4	10.9	Basin 12 & Part 1
MH-29	CB-17	24	RCP	360	0.066[s]	79	4.2	10.4	4.2	10.4	0.3	0.5	13.1	16.8	Basin 12 & Part 1
CB-17	35' D & P.U.E.	36	RCP	350	0.038[s]	170	46.4	95.9	46.4	95.9	1.1	1.7	19.7	23.6	Basins 1, 2, 3, 4, 5, 6, 7, 12 & 13
35' D & P.U.E.	MH-31	60	RCP	430	0.028[e]	555	32.4	82.6	32.4	82.6	0.8	1.3	15.1	19.6	Basins 1, 2, 3, 4, 5, 6, 7, 8, 12 & 13
MH-31	MH-33	60	RCP	70	0.028[e]	555	32.4	82.6	32.4	82.6	0.8	1.3	15.1	19.6	Basins 1, 2, 3, 4, 5, 6, 7, 8, 12 & 13
MH-32	MH-33	24	RCP	375	0.039[a]	60	20.0	20.0	20.0	20.0	0.8	0.8	16.3	16.3	Basin 14
MH-33	MH-34	60	RCP	225	0.040[a]	672	52.4	102.6	52.4	102.6	1.0	1.4	19.8	23.9	Basins 1, 2, 3, 4, 5, 6, 7, 8, 12, 13 & Part 14
MH-34	MH-35	60	RCP	260	0.040[a]	672	52.4	115.1	52.4	115.1	1.0	1.4	19.8	24.7	Basins 1, 2, 3, 4, 5, 6, 7, 8, 12, 13 & Part 14

TABLE 2: INVENTORY OF EXISTING STORM DRAINS (continued)

FROM	TO	PIPE DIA. (IN)	TYPE OF PIPE	APPROX. LENGTH (FT)	SLOPE (FT/FT)	EXIST. CAPACITY (CFS)	CONTRIB. FLOW (CFS)		PIPE FLOW (CFS)		ACTUAL DEPTH (FT)		FLOW VELOCITY (FPS)		CONTRIBUTING FLOW
							Q_{10}	Q_{100}	Q_{10}	Q_{100}	10	100	10	100	
MH-35	MH-36	60	RCP	60	0.040[a]	672	52.4	115.1	52.4	115.1	1.0	1.4	19.8	24.7	Basins 1, 2, 3, 4, 5, 6, 7, 8, 12, 13 & Part 14
MH-36	MH-37	60	RCP	114	0.036[a]	635	52.4÷2 =26.2	115.1÷2 =57.6	26.2	57.6	0.7	1.0	16.0	20.3	Basins 1, 2, 3, 4, 5, 6, 7, 8, 12, 13 & Part 14
MH-42	MH-41	24	RCP	445	0.067[a]	80	10.0	10.0	10.0	10.0	0.5	0.5	16.7	16.7	Basin 11
MH-41	MH-40	30	RCP	460	0.044[a]	114	10.0	20.0	10.0	20.0	0.5	0.7	13.9	16.8	Basin 11
MH-40	MH-39	30	RCP	290	0.062[a]	138	10.0	22.1	10.0	22.1	0.5	0.7	15.9	19.9	Basin 11
MH-39	MH-37	36	RCP	40	0.062[e]	222	10.0	22.1	10.0	22.1	0.4	0.6	15.6	19.9	Basin 11
MH-37	MH-38	60	RCP	70	0.029[s]	565	36.2	79.7	36.2	79.7	0.9	1.3	15.1	19.6	Basins 1, 2, 3, 4, 5, 6, 7, 8, 11, 12, 13 & 14
MH-36	MH-38	2-42	CMP	184	0.029[s]	2x147=294	52.4÷2 =26.2	115.1÷2 =57.6	26.2÷2 =13.1	57.6÷2 =28.8	0.7	1.1	9.2	11.4	Basins 1, 2, 3, 4, 5, 6, 7, 8, 12, 13 & 14
MH-38	AMAFCA Channel	2-54	RCP	80	0.036[a]	2x481=962	67.0	165.2	67 ÷2 =33.5	165.2÷2 =82.6	0.8	1.3	16.9	21.8	Basins 1, 2, 3, 5, 6, 7, 8, 12, 13 & 14

RCP = Reinforced concrete pipe
CMP = Corregated metal pipe
e = estimated
a = determined from As-Built records
s = determined from survey data

TABLE 2: INVENTORY OF EXISTING STORM DRAINS (continued)

DESCRIPTION	SOIL UNIT	HYDROLOGIC SOIL GROUP	CONDITION	CURVE NUMBER (CN)
Native Grass	WaB	B	Fair	70
Native Grass	BKD; Cu	A	Fair	54
S. W. Landscaping	WaB	B	Fair	70
S. W. Landscaping	BKD; Cu	A	Fair	54
Landscaping	BKD; Cu	A	Good	39
Gravel/Dirt	WaB	B	N/A	84
Impervious Area	N/A	N/A	N/A	95

TABLE 3: SUMMARY OF GROUND COVER CONDITIONS

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN RUNOFF SUMMARY
CONDITION: EXISTING

DESCRIPTION	AREA (ac)	Q-100 (cfs)	V-100 (cf.)	Q-10 (cfs)	V-10 (cf.)
1A	2.00	5.3	12,800	3.1	7,600
1	13.70	6.7	16,900	1.7	4,200
2	3.90	10.7	25,000	6.4	14,900
3	10.00	23.4	61,300	13.7	35,900
4	6.30	14.5	34,000	8.0	18,900
5	4.60	10.3	24,300	5.7	13,300
6	7.00	19.2	44,900	11.4	26,900
7	11.50	24.1	56,500	12.8	30,100
8A	2.20	0.1	300	0.0	0
8	24.50	3.9	9,100	0.1	200
9	2.60	2.2	5,200	0.7	1,800
10	1.60	0.0	0	0.2	600
11A	8.40	19.2	44,900	10.6	24,900
11B	4.40	1.1	2,600	0.1	200
11	17.10	22.1	54,700	9.8	24,300
12	2.00	5.4	12,800	3.2	7,600
13	5.60	3.4	8,400	1.0	2,400
14A	2.00	1.5	3,500	0.4	1,100
14	56.50	52.5	138,200	20.4	53,900

TABLE 4: EXISTING DRAINAGE BASIN RUNOFF SUMMARY

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN RUNOFF SUMMARY
CONDITION: EXISTING

DESCRIPTION	AREA (ac)	Q-100 (cfs)	V-100 (cf.)	Q-10 (cfs)	V-10 (cf.)
1A & 3	12.00	29.8	74,100	17.5	43,600
1A; 3; PART 4; & PART 5	14.50	34.0	87,500	19.8	51,000
PART OF 5	1.00	2.7	6,400	1.6	3,800
1A; 3; PART 4; & PART 5	15.50	36.5	93,900	21.2	54,800
1A; 3; PART 4; PART 5; & 6	22.50	52.0	138,900	30.6	81,600
1; 2 & 4	23.90	25.4	62,700	10.2	25,300
1; 2; 3; 4; PART 5; & 6	41.90	63.4	163,400	30.7	79,100
1; 2; 3; 4; 5; 6; 7 & 13	62.60	90.0	240,400	43.3	115,600
12	2.00	5.4	12,800	3.2	7,600
1; 2; 3; 4; 5; 6; 7; 12 & 13	64.60	95.9	252,300	46.4	122,200
1; 2; 3; 4; 5; 6; 7; 8; 12 & 13	89.10	82.6	220,500	32.4	86,500

TABLE 5: COMPOSITE (EXISTING) DRAINAGE BASIN RUNOFF SUMMARY

SECTION	TYPE	ANALYSIS	CROWN (IN)	WIDTH (FT)	CURB HEIGHT (IN)	MINIMUM SLOPE (FT/FT)	CAPACITY (CFS)	BASIN	ACTUAL FLOW		DEPTH @ FLOW LINE	
									Q_{10} (CFS)	Q_{100} (CFS)	Q_{10} (FT)	Q_{100} (FT)
A-A	Median	1/2 width	8	36	8	0.035	78	11	10.6	22.1	0.3	0.5
B-B	No Median	Full width	~0	24	8	0.04	176	5	5.7	10.3	0.1	0.1
C-C	No Median	Full width	~0	24	8	0.01	88	1	1.7	6.7	0.1	0.15
D-D	No Median	Full width	6	44	8	0.02	102	4	8.0	14.5	0.3	0.35
E-E	No Median	Full width	6	44	8	0.04	145	14	20.4	52.5	0.35	0.50

TABLE 6: STREET ANALYSIS POINTS - EXISTING CONDITION

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN RUNOFF SUMMARY
CONDITION: DEVELOPED

DESCRIPTION	AREA (ac)	Q-100 (cfs)	V-100 (cf.)	Q-10 (cfs)	V-10 (cf.)
1	7.60	9.8	24,200	4.3	10,700
2	25.00	61.5	160,600	36.7	96,100
4	6.00	15.7	38,500	9.4	23,000
5	5.80	7.8	18,400	3.4	8,100
6	7.00	19.2	44,900	11.4	26,900
7	10.30	20.9	49,000	11.0	25,800
8A	2.20	0.1	300	0.0	0
8	12.80	18.6	43,700	8.5	20,000
9	2.60	2.2	5,200	0.7	1,800
10	1.60	0.0	0	0.2	600
11A	8.40	19.2	44,900	10.6	24,900
11B	4.40	1.1	2,600	0.1	200
11	17.10	22.1	54,700	9.8	24,300
12	2.00	5.4	12,800	3.2	7,600
13	7.90	19.5	48,000	11.4	28,000
14	54.50	125.4	314,000	71.6	179,200
15	10.00	27.4	64,200	16.4	38,400
1; 2; 4; 5; 6; 7; 12 & 13	71.60	147.0	386,700	81.6	214,800
1; 2; 4; 5; 6; 7; 8; 12; 13 & 15	94.40	183.3	489,400	100.1	267,300

TABLE 7: PROPOSED DRAINAGE BASIN
RUNOFF SUMMARY

CB	TYPE	BASIN	LOCATION	CAPACITY (CFS)	Q_{10} (CFS)	Q_{100} (CFS)	COMMENTS
16R	2 Double "C"'s	7	Lo-point	57.6	11.0	20.9	Remove and reconstruct.
60	Double "C"	5 & 13	Sag-point	28.8	<16	<30	Construct in conjunction with Girard Boulevard extension.
61	Double "C"	5 & 13	Sag-point	28.8	<16	<30	Construct in conjunction with Girard Boulevard extension.
62	Double "C"	8	Sag-point	28.8	<8.5	<18.6	Construct in conjunction with Girard Boulevard extension.
63	Double "C"	8	Sag-point	28.8	<8.5	<18.6	Construct in conjunction with Girard Boulevard extension.
64	Single "C"	8	Sag-point	14.4	<8.5	<18.6	Construct in conjunction with Girard Boulevard extension.
65	Single "C"	8	Sag-point	14.4	<8.5	<18.6	Construct in conjunction with Girard Boulevard extension.
69	?	15	Lo-point	?	16.4	27.4	To be sized and installed at time of development.
70	Single "D"	4	Lo-point	14.4	6.4	10.7	Construct in Alamo Avenue when funds are available.
71	Double "C"	14	Curb-line	10	<71.6	<125.4	Construct in Alamo Avenue when funds are available.

TABLE 8: CATCH BASIN SUMMARY - PHASE I

CB	TYPE	BASIN	LOCATION	CAPACITY (CFS)	Q_{10} (CFS)	Q_{100} (CFS)	COMMENTS
72	Double "C"	14	Curb-line	10	<71.6	<125.4	Construct in Alamo Avenue when when funds are available.
73	Double "C"	14	Curb-line	10	<71.6	<125.4	Construct in Alamo Avenue when when funds are available.
74	Double "C"	14	Curb-line	10	<71.6	<125.4	Construct in Alamo Avenue when when funds are available.

TABLE 8: CATCH BASIN SUMMARY - PHASE I (continued)

CB	TYPE	BASIN	LOCATION	CAPACITY (CFS)	Q_{10}* (CFS)	Q_{100}* (CFS)	COMMENTS
21R	2' x 3' Grate	2	Lo-point	11.4	6	10	Remove and reconstruct.
22R	2' x 3' Grate	2	Lo-point	11.4	3	6	Remove and reconstruct.
66	2' x 3' Grate	2	Lo-point	11.4	4	6	New construction.
67	2' x 3' Grate	2	Lo-point	11.4	3	6	New construction.
68	2' x 3' Grate	2	Lo-point	11.4	4	6	New construction.

* Flows estimated by areal proportion of Basin 2.

TABLE 9: CATCH BASIN SUMMARY - PHASE II

FROM	TO	PIPE DIA. 1 (IN)	TYPE OF PIPE	APPROX. LENGTH (FT)	SLOPE (FT/FT)	CAPACITY (CFS)	CONTRIB. FLOW (CFS)		PIPE FLOW (CFS)		FLOW DEPTH (FT)		FLOW VELOCITY (FPS)		COMMENTS
							Q_{10}	Q_{100}	Q_{10}	Q_{100}	10	100	10	100	
ALTERNATE A:															
MH-27	MH-26	24	RCP	285	0.022[e]	44	37	62	37÷2=19	62÷2=31	1.0	1.3	12.8	14.2	Construct new 24" RCP parallel to exist. 24" RCP; Provide Junction Structure at MH.
MH-26	MH-28	24	RCP	420	0.022[s]	44	37	62	19	31	1.0	1.3	12.8	14.2	Construct new 24" RCP parallel to exist. 24" RCP; Provide Junction Structure at MH.
MH-28	MH-29	24	RCP	60	0.022[s]	44	40	67	40÷2=20	67÷2=34	1.0	1.3	12.8	15.6	Construct new 24" RCP parallel to exist. 24" RCP; Provide Junction Structure at MH.
MH-29	CB-17	24	RCP	360	0.066[s]	79	40	67	40	67	1.0	1.5	24.0	26.5	Construct Junction Structure at MH-29. Use existing 24" RCP only.

TABLE 10: STORM DRAIN SUMMARY - PHASE I
(analyzed for full development)

RCP = Reinforced concrete pipe
CMP = Corregated metal pipe

e = estimated
a = determined from As-Built records
s = determined from survey data
p = proposed

FROM	TO	PIPE DIA. 1 (IN)	TYPE OF PIPE	APPROX. LENGTH (FT)	SLOPE (FT/FT)	CAPACITY (CFS)	CONTRIB. FLOW (CFS)		PIPE FLOW (CFS)		FLOW DEPTH (FT)		FLOW VELOCITY (FPS)		COMMENTS
							Q_{10}	Q_{100}	Q_{10}	Q_{100}	10	100	10	100	
ALTERNATE B:															
MH-27R	MH-26R	36	RCP	330	0.022[P]	127	37	62	37	62	1.1	1.5	14.9	17.0	Construct new storm drain to replace 24" RCP. Abandon 24" RCP in place.
MH-26R	MH-28R	36	RCP	400	0.022[P]	127	37	62	37	62	1.1	1.5	14.9	17.0	Construct new storm drain to replace 24" RCP. Abandon 24" RCP in place. Connect CB-18 at MH-28R.
MH-28R	MH-30	36	RCP	350	0.022[P]	127	40	67	40	67	1.2	1.6	15.2	17.3	Construct new storm drain to replace 24" RCP. Abandon 24" RCP in place. Connect to MH-30.

TABLE 10: STORM DRAIN SUMMARY - PHASE I (continued)
(analyzed for full development)

RCP = Reinforced concrete pipe
CMP = Corregated metal pipe

e = estimated
a = determined from As-Built records
s = determined from survey data
p = proposed

FROM	TO	PIPE DIA. 1 (IN)	TYPE OF PIPE	APPROX. LENGTH (FT)	SLOPE (FT/FT)	CAPACITY (CFS)	CONTRIB. FLOW (CFS)		PIPE FLOW (CFS)		FLOW DEPTH (FT)		FLOW VELOCITY (FPS)		COMMENTS
							Q_{10}	Q_{100}	Q_{10}	Q_{100}	10	100	10	100	
MH-50	MH-51	24	RCP	270	0.01[P]	29	<8.5	<18.6	<8.5	<18.6	0.8	1.2	7.6	9.2	New construction.
MH-51	MH-55	24	RCP	380	0.04[P]	60	8.5	18.6	8.5	18.6	0.5	0.8	13.1	16.2	New construction; connect to exist. 60" RCP.
CB-17	MH-53	36	RCP	350	0.038[P]	170	81.6	147	81.6	147	1.5	2.3	22.7	25.4	Exist. storm drain; connect to new storm drain.
MH-53	MH-54	42	RCP	200	0.02[P]	181	81.6	147	81.6	147	1.7	2.5	17.4	19.7	New construction.
MH-54	MH-55	42	RCP	45	0.02[P]	181	81.6	147	81.6	147	1.7	2.5	17.4	19.7	New construction; connect to exist. 60" RCP
MH-55	MH-31	60	RCP	430	0.028[e]	555	100.1	183.3	100.1	183.3	1.5	2.0	20.6	24.2	Existing.
MH-31	MH-33	60	RCP	70	0.028[e]	555	100.1	183.3	100.1	183.3	1.5	2.0	20.6	24.2	Existing.

TABLE 10: STORM DRAIN SUMMARY - PHASE I (continued)
(analyzed for full development)

FROM	TO	PIPE DIA. 1 (IN)	TYPE OF PIPE	APPROX. LENGTH (FT)	SLOPE (FT/FT)	CAPACITY (CFS)	CONTRIB. FLOW (CFS)		PIPE FLOW (CFS)		FLOW DEPTH (FT)		FLOW VELOCITY (FPS)		COMMENTS
							Q_{10}	Q_{100}	Q_{10}	Q_{100}	10	100	10	100	
MH-56	MH-57	24	RCP		0.04[P]	60	20	20	20	20	0.8	0.8	16.3	16.3	New construction.
MH-57	MH-32	24	RCP		0.04[P]	60	40	40	40	40	1.2	1.2	19.4	19.4	New construction.
MH-32	MH-33	24	RCP	375	0.039[a]	60	60	60	60	60	2	2	20.0	20.0	Existing.
MH-33	MH-34	60	RCP	225	0.040[a]	672	171.7	243.3	171.7	243.3	1.8	2.2	27.0	30.0	Existing.
MH-34	MH-35	60	RCP	260	0.040[a]	672	171.7	263.3	171.7	263.3	1.8	2.3	27.0	30.7	Existing.
MH-35	MH-36	60	RCP	60	0.040[a]	672	171.7	308.7	171.7	308.7	1.8	2.5	27.0	31.8	Existing.
MH-36	MH-37	60	RCP	114	0.036[a]	635	171.7÷ 2=85.8	308.7÷2 =154.4	85.8	154.4	1.2	1.7	21.4	25.6	Existing
MH-43	MH-42	24	RCP	450	0.04[P]	60	0.7	2.2	0.7	2.2	0.2	0.3	6.4	9.0	New construction.
MH-42	MH-41	24	RCP	445	0.067[a]	80	10.7	12.2	10.7	12.2	0.5	0.5	16.7	17.7	Existing.

TABLE 10: STORM DRAIN SUMMARY - PHASE I (continued)
(analyzed for full development)

FROM	TO	PIPE DIA. 1 (IN)	TYPE OF PIPE	APPROX. LENGTH (FT)	SLOPE (FT/FT)	CAPACITY (CFS)	CONTRIB. FLOW (CFS)		PIPE FLOW (CFS)		FLOW DEPTH (FT)		FLOW VELOCITY (FPS)		COMMENTS
							Q_{10}	Q_{100}	Q_{10}	Q_{100}	10	100	10	100	
MH-41	MH-40	30	RCP	460	0.044[a]	114	10.7	22.2	10.7	22.2	0.5	0.8	14	17.4	Existing.
MH-40	MH-39	30	RCP	290	0.062[a]	138	10.7	24.3	10.7	24.3	0.5	0.7	16	20.3	Existing.
MH-39	MH-37	36	RCP	40	0.062[e]	222	10.7	24.3	10.7	24.3	0.5	0.7	16	20	Existing.
MH-37	MH-38	60	RCP	70	0.029[s]	565	96.5	178.7	96.5	178.7	1.4	2	20.4	24.5	Existing.
MH-36	MH-38	2-42	CMP	184	0.029[s]	2x147=294	171.7÷ 2=85.8	308.7÷ 2=154.4	85.8÷2 =42.9	154.4÷2 =77.2	1.3	1.9	12.7	14.7	Existing.
MH-38	AMAFCA Channel	2-54	RCP	80	0.036[a]	2x481=962	182.4	333	182.4÷2 =91.2	333÷2 =166.5	1.4	1.9	22.3	26.2	Existing.

TABLE 10: STORM DRAIN SUMMARY - PHASE I (continued)
(analyzed for full development)

RCP = Reinforced concrete pipe
CMP = Corregated metal pipe

e = estimated
a = determined from As-Built records
s = determined from survey data
p = proposed

FROM	TO	PIPE DIA. 1 (IN)	TYPE OF PIPE	APPROX. LENGTH (FT)	SLOPE (FT/FT)	CAPACITY (CFS)	CONTRIB. FLOW (CFS)		PIPE FLOW (CFS)		FLOW DEPTH (FT)		FLOW VELOCITY (FPS)		COMMENTS
							Q_{10}	Q_{100}	Q_{10}	Q_{100}	10	100	10	100	
CB-19	CB-20	24	RCP	330	0.01^{p}	29	<17	<28	<17	<28	<1.2	<1.8	<9	<9.5	Relay section of storm drain.
CB-20	MH-52	24	RCP	185	0.01^{p}	29	<17	<28	<17	<28	<1.2	<1.8	<9	<9.5	Construct new section of storm drain.
MH-18	MH-52	24	RCP	340	0.01^{p}	29	<17	<28	<17	<28	<1.2	<1.8	<9	<9.5	Relay section of storm drain.
MH-52	CB-21R	24	RCP	100	0.01^{p}	29	17	28	17	28	1.2	1.8	9	9.5	Construct new section of storm drain; relocate CB-22.
CB-21R	CB-22R	30	RCP	200	0.01^{p}	51	23	38	23	38	1.2	1.7	9.7	10.8	Construct new section of storm drain; relocate CB-22.
CB-22R	CB-66	30	RCP	200	0.01^{p}	51	26	44	26	44	1.3	1.9	10.0	11.0	Construct new section of storm drain.
CB-66	CB-67	30	RCP	200	0.01^{p}	51	30	50	30	50	1.4	2.2	10.3	11.0	Construct new section of storm drain.
CB-67	CB-68	36	RCP	200	0.01^{p}	83	33	56	33	56	1.4	1.9	10.6	11.9	Construct new section of storm drain.
CB-68	MH-27	36	RCP	320	0.01^{p}	83	37	62	37	62	1.5	2.0	10.9	12.1	Construct new section of storm drain; Connect to Phase I Facility.
	or MH-27R	36	RCP	290	0.01^{p}	83	37	62	37	62	1.5	2.0	10.9	12.1	

TABLE 11: STORM DRAIN CONSTRUCTION - PHASE II (RCP = Reinforced concrete pipe; p = Proposed)

SECTION	TYPE	ANALYSIS	CROWN (IN)	WIDTH (FT)	CURB HEIGHT (IN)	MINIMUM SLOPE (FT/FT)	CAPACITY (CFS)	BASIN	ACTUAL FLOW		DEPTH @ FLOW LINE	
									Q_{10} (CFS)	Q_{100} (CFS)	Q_{10} (FT)	Q_{100} (FT)
A-A	Median	1/2 width	8	36	8	0.035	78	11	10.6	22.1	0.3	0.5
B-B	No Median	Full width	~0	36	8	0.04	268	5	5.7	10.3	0.1	0.1
C-C	No Median	1/2 width	5	24	8	0.01	48	13	11.4	19.5	0.4	0.5
D-D	No Median	Min. width	6	44	8	0.02	102	4	9.4	15.7	0.3	0.4
E-E	No Median	Full width	6	44	8	0.04	145	14	71.6	125.4	0.55	0.65

TABLE 12: STREET ANALYSIS POINTS - DEVELOPED CONDITION

V. RECOMMENDATIONS AND CONCLUSIONS

A. PHASE I

The following recommendations and conclusions are presented for Phase I of the Drainage and Traffic Plans:

1. Construct a four-lane divided roadway connecting Yale Boulevard to Girard Boulevard. The typical street section should be 64 feet face-to-face with a 16-foot face-to-face median symmetric about the street centerline. Due to prior City commitments, the street section will be limited to 60 feet face-to-face with a painted 12-foot wide median for approximately 600 feet east of Yale Boulevard. A 400-foot transition will increase the street width to 64 feet face-to-face at a distance of 1,000 feet east of Yale Boulevard. The construction of this road will require extensive grading and/or the construction of retaining walls along the first 1,000 to 1,200 feet of roadway beginning at Yale Boulevard. The remainder of the roadway extension and widening to Gibson will require minimal grading. These determinations have been made upon review of Figure 5 on which existing and proposed grades are shown.

The construction of new storm drainage facilities and the modification of existing storm drainage facilities should be performed in conjunction with the construction of the roadway discussed above. Details concerning the construction of the storm drainage facilities are described in more detail in Section IV.

2. Revise the traffic pattern in front of the Terminal Building to accommodate six lanes of traffic. This will require eliminating the rental car check-in area, replacing the single curb divider with a 6-foot median, and locating two turnstiles for entry into the Upper Parking Lot. The relocation of existing storm drainage facilities in this area should not be affected by these revisions. Minor reconstruction may be deemed necessary by a detailed re-design of the area, however, the locations of the existing facilities should remain unchanged. Minimum lane width should be 12 feet.

3. Relocate the toll booth from the parking area from the west side to the east side. This will reduce congestion in front of the Terminal Building.

4. Provide entrances to the parking lots as shown by Figure 3.

5. Modify the parking and circulation pattern of the Upper Parking Lot to accommodate the proposed entrances and relocation of the toll booth. A herringbone pattern with 60-degree angle parking is suggested.

6. Abandon the intersection of Girard Boulevard with the Service Road and relocate the intersection as shown on Figure 3. This will reduce the accident potential of the existing intersection.

7. Institute a maintenance program by which the existing storm drains are inspected and maintained on a regular basis.

This will ensure that the storm drainage facilities remain in clean, operating condition and will undergo routine repairs as necessary. Instituting such a program will eliminate many of the small problems with the existing storm drainage system which were mentioned earlier in this report.

8. Construct a catch basin at the low-point along the westerly boundary of Avis Rent-A-Car where existing runoff has undermined the wall. This will solve the current problem until this area is redeveloped in the future.

9. Construct a storm drain parallel to the existing 24-inch RCP storm drain between MH-27 and MH-29. The parallel system should either supplement the existing line with 24-inch RCP or replace it entirely with 36-inch RCP.

10. Construct a 42-inch RCP storm drain between the existing 36-inch RCP line and the existing 60-inch RCP line within the 35-foot drainage and public utility easement. This will contain the storm water flowing into that easement and eliminate the unsightliness of the easement.

11. Connect MH-42 with MH-43 with approximately 450 lineal feet of 24-inch RCP storm drain and abandon the existing 30-inch CMP draining west from MH-43. This will enhance the development of Airport land along with eliminating an existing drainage problem.

In Yale Blvd along Basin 11B

12. Abandon the two existing 24-inch RCP highway culverts and grade the areas draining to those culverts to drain to Girard Boulevard instead. This will eliminate the discharge of runoff from Airport land onto non-Airport land.

13. Correct the soil erosion problem along the north boundary of the Airport land adjacent to Budget Rent-A-Car and the F.A.A. Field Maintenance Yard. This can be accomplished by (1) the construction of retaining wall, (2) covering the slope with an impervious slope covering or (3) constructing a combination of retaining walls and slope covering.

14. Extend the existing storm drain in Alamo Avenue further upstream with 24-inch RCP and Double 'C' catch basins. This is described in more detail in Section IV.

B. PHASE II

Phase II of the Traffic and Drainage Plan concerns the future expansion of the Terminal Building and associated aviation facilities. In consideration of such expansion, the following recommendations and conclusions are presented:

1. Extend the six lanes of roadway in front of the existing Terminal Building. This will provide adequate service to the Terminal Building extension. This, of course, assumes that expansion will take place as shown on Figure 7.

2. Construct a series of catch basins and storm drains to the east of the future Terminal expansion. It is anticipated

that this area will be primarily apron space which will need to be relatively flat. Because of this, a storm drain system will be needed to convey runoff from the area. The proposed storm drain system for Phase II should connect to the Phase I system as shown by Figure 11. This will avoid overloading the existing storm drain system in the parking lot area.

3. Discharge 100 percent of the fully developed runoff into the existing and proposed storm drainage facilities. The analysis of the existing and proposed storm drainage facilities presented earlier demonstrated that the facilities have the capacity to convey in excess of the 100-year peak flow for the anticipated fully developed condition.

C. PHASE III

Recommendations and conclusions for Phase III of the Traffic and Drainage Plan are presented below:

1. Construct a parking structure upon the site of the Upper Parking Lot. This will maximize the efficiency in which Airport land is used, as well as minimizing the walking distance between automobile parking and the Terminal Building. Pedestrian bridges should be provided between the parking structure and the Terminal Building to reduce congestion on the ground.

2. No storm drainage facilities are proposed for Phase III. The parking structure, however, should be designed with

an internal drainage system to convey storm water runoff into the existing storm drain system. The development of the Lower Parking Lot into another commercial use should provide for positive drainage of the area. Because the Lower Parking Lot already freely discharges all runoff, future development should be permitted to do the same. This is because it has been demonstrated that the existing downstream drainage facilities have the capacity to convey the runoff generated by a 100-year frequency storm falling on the study area under fully developed conditions.

APPENDIX A:

RUNOFF CALCULATIONS

RUNOFF CALCUATIONS:

Storm runoff is determined by the Soil Conservation Service (SCS) Curve Number Method for small watersheds (less than 2,000 acres). This method determines the peak discharge and associated volume of runoff for a given frequency storm falling within the study area. The following criteria were used to quantify runoff:

Watershed Curve Numbers: The runoff curve numbers for the various watershed conditions are listed in Table 3. Using these values, a composite curve number (CN) is computed for an entire watershed.

Time of Concentration: The time of concentration (T_c) is the time it takes for runoff to travel from the most distant part of the watershed area to the point of discharge. Time of concentration is estimated by the Kirpich equation:

$$T_c = 0.00013 \frac{L^{0.77}}{s^{0.385}}$$

where L = hydraulic length of the watershed, in feet,

and s = average slope of the watershed in feet/feet.

Rainfall: The depth of rainfall is determined from the 1973, NOAA ATLAS 2, VOLUME IV. The 100-year, 6-hour precipitation = 2.3", and the 10-year, 6-hour precipitation = 1.55" for the study area.

Direct Runoff: The SCS runoff equation is used to determine the amount of runoff:

$$Q = \frac{(P-0.2S)^2}{(P+0.8S)}$$

$$S = \frac{1000}{CN} - 10$$

where Q = accumulated volume of runoff in inches over the drainage area.

P = accumulated rainfall in inches over the drainage area.

S = Potential maximum retention of water in equivalent inches over the drainage area.

CN = Watershed Curve Number.

Distribution Curve: The distribution curve (DC) is based on the percent of one hour versus 24 hours precipitation and is dependent upon the geographic location of the drainage basin. Based upon Figure 12, the SCS Peak Rates of Discharge for Small Watersheds, DC = 75.

Peak Discharge: The peak discharge is obtained from Figure 13 which plots the cfs/ac/inch of runoff versus T_c for various distribution curves. This is multiplied by the drainage area in acres and the direct runoff in inches to obtain the peak discharge in cfs.

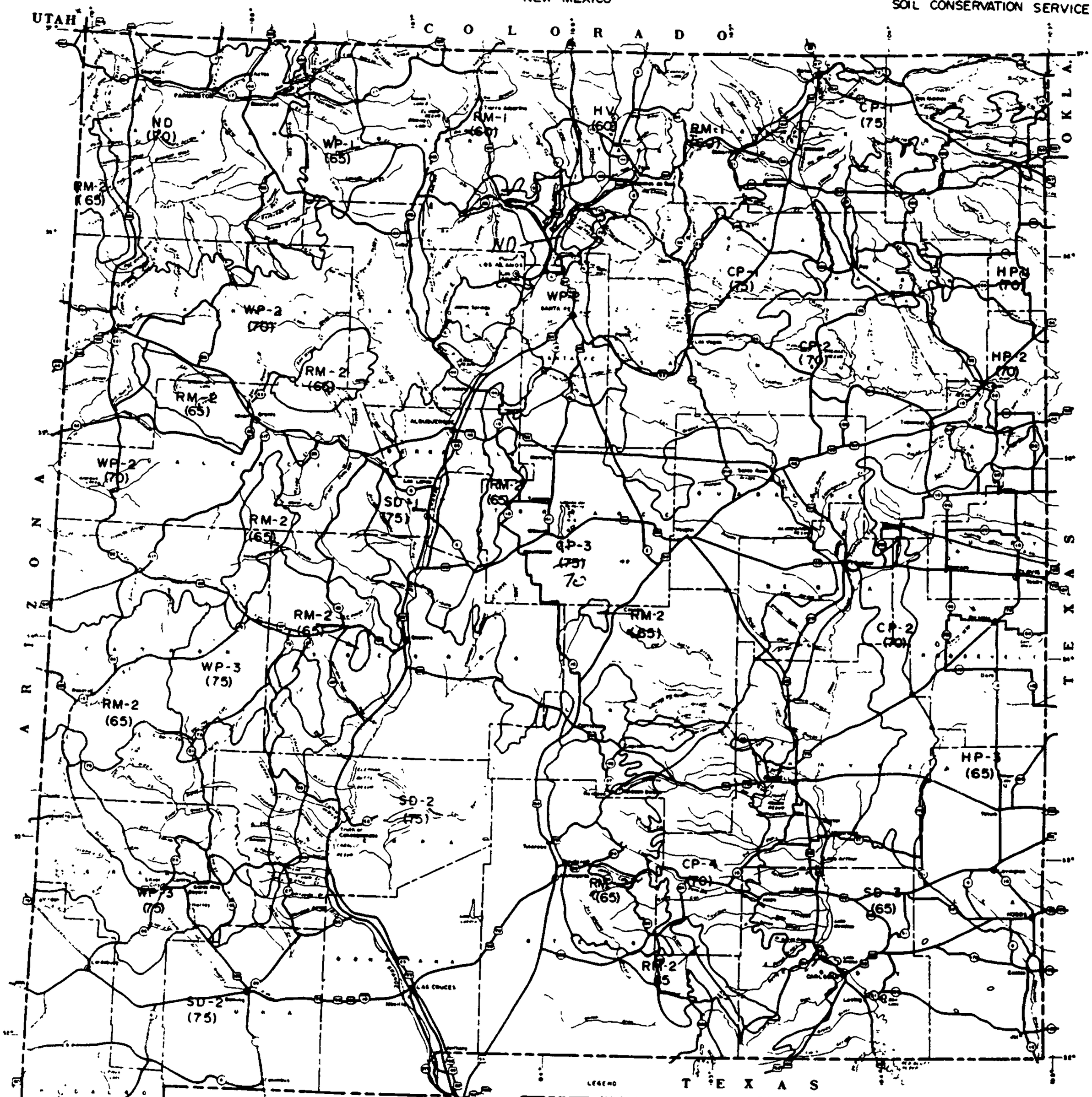
Volume of Runoff: The volume of runoff in acre-feet from a watershed is expressed as the direct runoff in feet multiplied by the watershed area.

As previously mentioned, the SCS Curve Number Method was computerized for ease of calculation and orderly presentation

of results. The results of such analysis follow. Existing and fully developed conditions have been analyzed.

U.S. DEPARTMENT OF AGRICULTURE

SOIL CONSERVATION SERVICE



LEGEND

- State Boundary
- County Boundary
- State Capital
- County Seat (small town)
- Town Less Than 5000 Population
- Town 5000 to 25,000 Population
- Town Over 25,000 Population

Towns of less than 5000 are read from map unless interpolated, or required for orientation purposes

- Lake or Reservoir
- Canal or Ditch
- Large Stream
- Small Stream
- Water Spreading
- Interstate Highway
- Federal Highway
- State Highway

DAILY RAINFALL DISTRIBUTION MAP

BASED ON % - 1 hr vs 24 hr. PRECIPITATION

by

MAJOR AND SUB LAND RESOURCE AREAS

NEW MEXICO

October 1973

SCALE 1 : 1 000 000

ALBERS EQUAL-AREA PROJECTION

GS National Atlas 1:1,000,000 Albers equal-Area projection (1967) used as source base map and adapted for SCS use

% of 1hr/24 hr	MAJOR AND SUB LAND RESOURCE AREAS
60	HV, RM-1
65	HP 3, [illegible], SD 3, WP 1, RM 2
70	HP 1, HP 2, CP 2, CP 3, WP 2, ND, CP 4
75	CP 1, SD 1, SD 2, WP 3

FIGURE 12

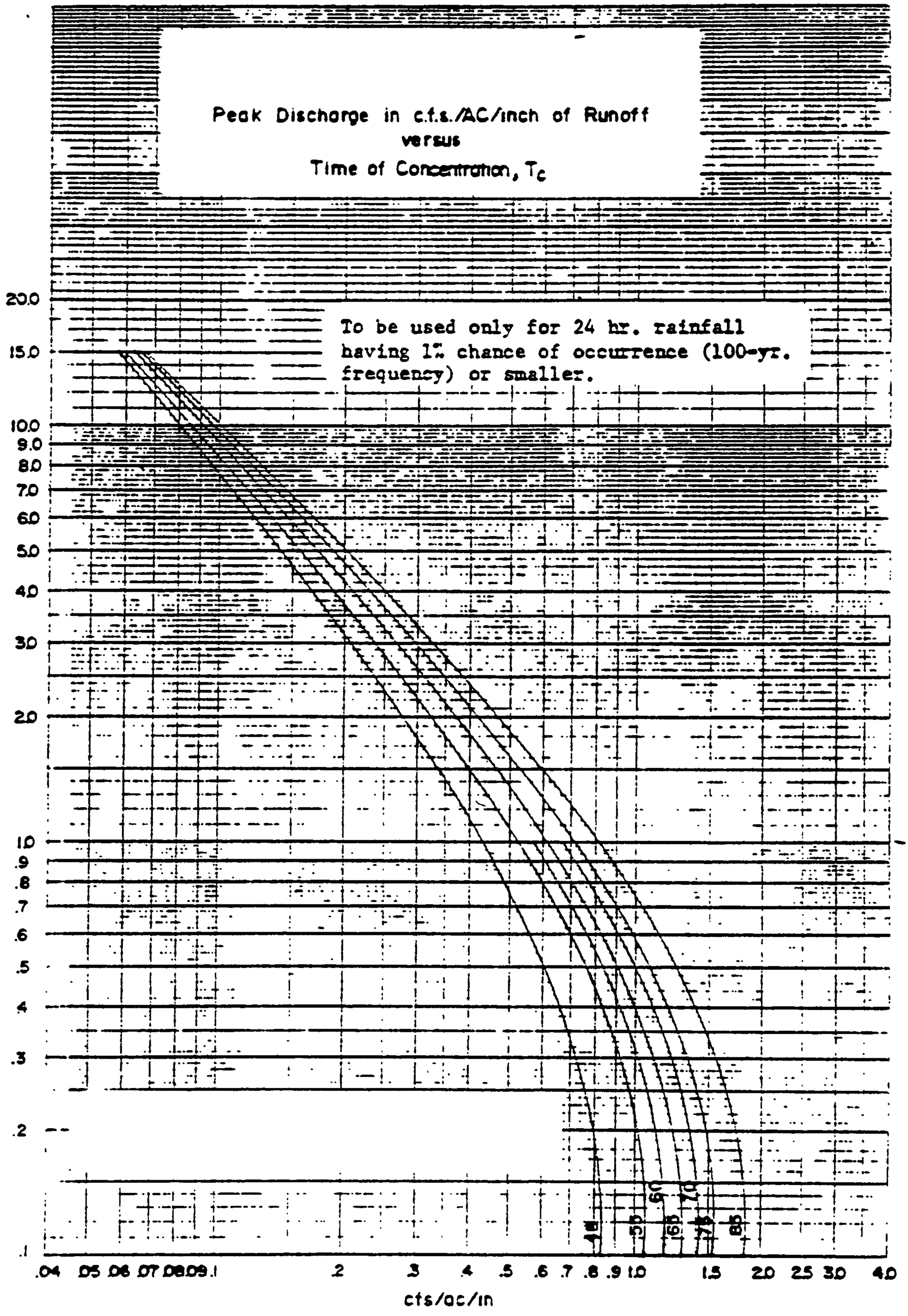


FIGURE 13: PEAK DISCHARGE VS. TIME OF CONCENTRATION

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN RUNOFF SUMMARY
CONDITION: EXISTING

DESCRIPTION	AREA (ac)	Q-100 (cfs)	V-100 (cf.)	Q-10 (cfs)	V-10 (cf.)
1A	2.00	5.3	12,800	3.1	7,600
1	13.70	6.7	16,900	1.7	4,200
2	3.90	10.7	25,000	6.4	14,900
3	10.00	23.4	61,300	13.7	35,900
4	6.30	14.5	34,000	8.0	18,900
5	4.60	10.3	24,300	5.7	13,300
6	7.00	19.2	44,900	11.4	26,900
7	11.50	24.1	56,500	12.8	30,100
8A	2.20	0.1	300	0.0	0
8	24.50	3.9	9,100	0.1	200
9	2.60	2.2	5,200	0.7	1,800
10	1.60	0.0	0	0.2	600
11A	8.40	19.2	44,900	10.6	24,900
11B	4.40	1.1	2,600	0.1	200
11	17.10	22.1	54,700	9.8	24,300
12	2.00	5.4	12,800	3.2	7,600
13	5.60	3.4	8,400	1.0	2,400
14A	2.00	1.5	3,500	0.4	1,100
14	56.50	52.5	138,200	20.4	53,900

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 1A
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
IMPERVIOUS AREA		2.00	95	100 %
	COMPOSITE:	2.00	95	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 470 ft.
DROP: 1 ft.
SLOPE: 0.2 %
TIME OF CONCENTRATION: 0.16 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 1.77 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.51 cfs/ac/in
PEAK DISCHARGE (Q): 5.3 c.f.s.

VOLUME: 12,800 c.f. 0.29ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 1.06 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.51 cfs/ac/in
PEAK DISCHARGE (Q): 3.1 c.f.s.

VOLUME: 7,600 c.f. 0.17ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 1
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
NATIVE GRASS (FAIR)	A	6.60	54	48 %
NATIVE GRASS (FAIR)	B	3.30	70	24 %
IMPERVIOUS AREA		3.80	95	28 %
COMPOSITE:		13.70	69	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 1700 ft.
DROP: 27 ft.
SLOPE: 1.5 %
TIME OF CONCENTRATION: 0.20 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.	FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 2.3 in.	RAINFALL: 1.6 in.
RUNOFF: 0.34 in.	RUNOFF: 0.09 in.
DISTRIBUTION CURVE: 75	DISTRIBUTION CURVE: 75
DISCHARGE: 1.46 cfs/ac/in	DISCHARGE: 1.46 cfs/ac/in
PEAK DISCHARGE (Q): 6.7 c.f.s.	PEAK DISCHARGE (Q): 1.7 c.f.s.
VOLUME: 16,900 c.f. 0.39ac.ft.	VOLUME: 4,200 c.f. 0.10ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 2
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
IMPERVIOUS AREA		3.90	95	100 %
	COMPOSITE:	3.90	95	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 900 ft.
DROP: 26 ft.
SLOPE: 2.8 %
TIME OF CONCENTRATION: 0.10 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 1.77 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 10.7 c.f.s.

VOLUME: 25,000 c.f. 0.57ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 1.06 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 6.4 c.f.s.

VOLUME: 14,900 c.f. 0.34ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 3
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
GRAVEL/DIRT	B	0.80	84	8 %
IMPERVIOUS AREA		9.20	95	92 %
	COMPOSITE:	10.00	94	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 850 ft.
DROP: 2 ft.
SLOPE: 0.2 %
TIME OF CONCENTRATION: 0.26 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 1.69 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.39 cfs/ac/in
PEAK DISCHARGE (Q): 23.4 c.f.s.

VOLUME: 61,300 c.f. 1.41ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.99 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.39 cfs/ac/in
PEAK DISCHARGE (Q): 13.7 c.f.s.

VOLUME: 35,900 c.f. 0.82ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 4
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
S.W. LANDSCAPING	A	0.50	54	8 %
IMPERVIOUS AREA		5.80	95	92 %
	COMPOSITE:	6.30	92	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 750 ft.
DROP: 26 ft.
SLOPE: 3.4 %
TIME OF CONCENTRATION: 0.08 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 1.49 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 14.5 c.f.s.

VOLUME: 34,000 c.f. 0.78ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.83 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 8.0 c.f.s.

VOLUME: 18,900 c.f. 0.43ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 5
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
LANDSCAPING (GOOD)	A	0.30	39	7 %
IMPERVIOUS AREA		4.30	95	93 %
	COMPOSITE:	4.60	91	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 750 ft.
DROP: 26 ft.
SLOPE: 3.4 %
TIME OF CONCENTRATION: 0.00 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 1.46 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 10.3 c.f.s.

VOLUME: 24,300 c.f. 0.56ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.80 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 5.7 c.f.s.

VOLUME: 13,300 c.f. 0.31ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 6
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
IMPERVIOUS AREA		7.00	95	100 %
	COMPOSITE:	7.00	95	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 750 ft.
DROP: 11 ft.
SLOPE: 1.4 %
TIME OF CONCENTRATION: 0.11 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 1.77 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 19.2 c.f.s.

VOLUME: 44,900 c.f. 1.03ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 1.06 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 11.4 c.f.s.

VOLUME: 26,900 c.f. 0.62ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 7
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
S.W. LANDSCAPING	A	1.40	54	12 %
IMPERVIOUS AREA		10.10	95	88 %
	COMPOSITE:	11.50	90	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 950 ft.
DROP: 22 ft.
SLOPE: 2.3 %
TIME OF CONCENTRATION: 0.11 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 1.35 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 24.1 c.f.s.

VOLUME: 56,500 c.f. 1.30ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.72 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 12.8 c.f.s.

VOLUME: 30,100 c.f. 0.69ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 8A
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
LANDSCAPING (GOOD)	A	1.60	39	73 %
IMPERVIOUS AREA		0.60	95	27 %
	COMPOSITE:	2.20	54	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 450 ft.
DROP: 4 ft.
SLOPE: 0.8 %
TIME OF CONCENTRATION: 0.09 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 0.04 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 0.1 c.f.s.

VOLUME: 300 c.f. 0.01ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.00 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 0.0 c.f.s.

VOLUME: 0 c.f. 0.00ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 8
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
NATIVE GRASS (FAIR)	A	18.60	54	76 %
LANDSCAPING (GOOD)	A	2.20	39	9 %
IMPERVIOUS AREA		3.70	95	15 %
COMPOSITE:		24.50	59	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 1500 ft.
DROP: 76 ft.
SLOPE: 5.0 %
TIME OF CONCENTRATION: 0.11 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 0.10 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 3.9 c.f.s.

VOLUME: 9,100 c.f. 0.21ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.00 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 0.1 c.f.s.

VOLUME: 200 c.f. 0.00ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 9
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
LANDSCAPING (GOOD)	A	0.90	39	35 %
IMPERVIOUS AREA		1.70	95	65 %
	COMPOSITE:	2.60	76	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 900 ft.
DROP: 12 ft.
SLOPE: 1.3 %
TIME OF CONCENTRATION: 0.13 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 0.56 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 2.2 c.f.s.

VOLUME: 5,200 c.f. 0.12ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.20 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 0.7 c.f.s.

VOLUME: 1,800 c.f. 0.04ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 10
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
LANDSCAPING (GOOD)	A	1.50	39	94 %
IMPERVIOUS AREA		0.10	95	6 %
	COMPOSITE:	1.60	43	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 450 ft.
DROP: 14 ft.
SLOPE: 3.1 %
TIME OF CONCENTRATION: 0.05 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 0.01 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 0.0 c.f.s.

VOLUME: 0 c.f. 0.00ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.11 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 0.2 c.f.s.

VOLUME: 600 c.f. 0.01ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 11A
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
S.W.LANDSCAPING	A	0.70	54	8 %
IMPERVIOUS AREA		7.70	95	92 %
	COMPOSITE:	8.40	92	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 850 ft.
DROP: 20 ft.
SLOPE: 2.3 %
TIME OF CONCENTRATION: 0.10 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 1.48 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 19.2 c.f.s.

VOLUME: 44,900 c.f. 1.03ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.82 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 10.6 c.f.s.

VOLUME: 24,900 c.f. 0.57ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 11B
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
S.W. LANDSCAPING	A	1.20	54	27 %
LANDSCAPING (GOOD)	A	1.70	39	39 %
IMPERVIOUS AREA		1.50	95	34 %
COMPOSITE:		4.40	62	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 850 ft.
DROP: 36 ft.
SLOPE: 4.2 %
TIME OF CONCENTRATION: 0.08 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 0.16 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 1.1 c.f.s.

VOLUME: 2,600 c.f. 0.06ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.02 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 0.1 c.f.s.

VOLUME: 200 c.f. 0.00ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 11
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
S.W. LANDSCAPING	A	2.60	54	15 %
LANDSCAPING (GOOD)	A	1.90	39	11 %
IMPERVIOUS AREA		12.60	95	74 %
COMPOSITE:		17.10	83	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 2550 ft.
DROP: 104 ft.
SLOPE: 4.0 %
TIME OF CONCENTRATION: 0.19 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 0.88 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.47 cfs/ac/in
PEAK DISCHARGE (Q): 22.1 c.f.s.

VOLUME: 54,700 c.f. 1.26ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.39 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.47 cfs/ac/in
PEAK DISCHARGE (Q): 9.8 c.f.s.

VOLUME: 24,300 c.f. 0.56ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 12
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
IMPERVIOUS AREA		2.00	95	100 %
	COMPOSITE:	2.00	95	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 350 ft.
DROP: 8 ft.
SLOPE: 2.2 %
TIME OF CONCENTRATION: 0.05 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 1.77 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 5.4 c.f.s.

VOLUME: 12,800 c.f. 0.29ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 1.06 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 3.2 c.f.s.

VOLUME: 7,600 c.f. 0.17ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 13
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
S.W. LANSCAPING/NATIVE GRASS	A	3.20	54	57 %
IMPERVIOUS AREA		2.40	95	43 %
	COMPOSITE:	5.60	72	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 1700 ft.
DROP: 35 ft.
SLOPE: 2.0 %
TIME OF CONCENTRATION: 0.18 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 0.41 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.48 cfs/ac/in
PEAK DISCHARGE (Q): 3.4 c.f.s.

VOLUME: 8,400 c.f. 0.19ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.12 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.48 cfs/ac/in
PEAK DISCHARGE (Q): 1.0 c.f.s.

VOLUME: 2,400 c.f. 0.06ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 14A
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
NATIVE GRASS (FAIR)	B	1.70	70	85 %
IMPERVIOUS AREA		0.30	95	15 %
	COMPOSITE:	2.00	74	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 700 ft.
DROP: 7 ft.
SLOPE: 1.0 %
TIME OF CONCENTRATION: 0.12 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 0.49 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 1.5 c.f.s.

VOLUME: 3,500 c.f. 0.08ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.16 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 0.4 c.f.s.

VOLUME: 1,100 c.f. 0.03ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 14
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
NATIVE GRASS (FAIR)	B	14.10	70	25 %
NATIVE GRASS (FAIR)	A	11.70	54	21 %
LANDSCAPING (GOOD)	A	2.00	39	4 %
IMPERVIOUS AREA		28.70	95	51 %
COMPOSITE:		56.50	78	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 3400 ft.
DROP: 106 ft.
SLOPE: 3.1 %
TIME OF CONCENTRATION: 0.26 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 0.67 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.38 cfs/ac/in
PEAK DISCHARGE (Q): 52.5 c.f.s.

VOLUME: 138,200 c.f. 3.17ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.26 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.38 cfs/ac/in
PEAK DISCHARGE (Q): 20.4 c.f.s.

VOLUME: 53,900 c.f. 1.24ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN RUNOFF SUMMARY
CONDITION: EXISTING

DESCRIPTION	AREA (ac)	Q-100 (cfs)	V-100 (cf.)	Q-10 (cfs)	V-10 (cf.)
1A & 3	12.00	29.8	74,100	17.5	43,600
1A; 3; PART 4; & PART 5	14.50	34.0	87,500	19.8	51,000
PART OF 5	1.00	2.7	6,400	1.6	3,800
1A; 3; PART 4; & PART 5	15.50	36.5	93,900	21.2	54,800
1A; 3; PART 4; PART 5; & 6	22.50	52.0	138,900	30.6	81,600
1; 2 & 4	23.90	25.4	62,700	10.2	25,300
1; 2; 3; 4; PART 5; & 6	41.90	63.4	163,400	30.7	79,100
1; 2; 3; 4; 5; 6; 7 & 13	62.60	90.0	240,400	43.3	115,600
12	2.00	5.4	12,800	3.2	7,600
1; 2; 3; 4; 5; 6; 7; 12 & 13	64.60	95.9	252,300	46.4	122,200
1; 2; 3; 4; 5; 6; 7; 8; 12 & 13	89.10	82.6	220,500	32.4	86,500

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 1A & 3
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
GRAVEL/DIRT	B	0.80	84	7 %
IMPERVIOUS AREA		11.20	95	93 %
	COMPOSITE:	12.00	94	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 1400 ft.
DROP: 15 ft.
SLOPE: 1.0 %
TIME OF CONCENTRATION: 0.20 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 1.70 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.46 cfs/ac/in
PEAK DISCHARGE (Q): 29.8 c.f.s.

VOLUME: 74,100 c.f. 1.70ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 1.00 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.46 cfs/ac/in
PEAK DISCHARGE (Q): 17.5 c.f.s.

VOLUME: 43,600 c.f. 1.00ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 1A; 3; PART 4; & PART 5
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
S.W. LANDSCAPING	A	0.20	54	1 %
GRAVEL/DIRT	B	0.80	84	6 %
IMPERVIOUS AREA		13.50	95	93 %
COMPOSITE:		14.50	94	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 1630 ft.
DROP: 16 ft.
SLOPE: 0.9 %
TIME OF CONCENTRATION: 0.24 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 1.66 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.41 cfs/ac/in
PEAK DISCHARGE (Q): 34.0 c.f.s.

VOLUME: 87,500 c.f. 2.01ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.97 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.41 cfs/ac/in
PEAK DISCHARGE (Q): 19.8 c.f.s.

VOLUME: 51,000 c.f. 1.17ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 1A; 3; PART 4; & PART 5
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
S.W. LANDSCAPING	A	0.20	54	1 %
GRAVEL/DIRT	B	0.80	84	5 %
IMPERVIOUS AREA		14.50	95	94 %
COMPOSITE:		15.50	94	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 1630 ft.
DROP: 16 ft.
SLOPE: 0.9 %
TIME OF CONCENTRATION: 0.24 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 1.67 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.41 cfs/ac/in
PEAK DISCHARGE (Q): 36.5 c.f.s.

VOLUME: 93,900 c.f. 2.16ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.97 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.41 cfs/ac/in
PEAK DISCHARGE (Q): 21.2 c.f.s.

VOLUME: 54,800 c.f. 1.26ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 1A; 3; PART 4; PART 5; & 6
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
S.W. LANDSCAPING	A	0.20	54	1 %
GRAVEL/DIRT	B	0.80	84	4 %
IMPERVIOUS AREA		21.50	95	96 %
COMPOSITE:		22.50	94	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 2700 ft.
DROP: 48 ft.
SLOPE: 1.7 %
TIME OF CONCENTRATION: 0.27 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 1.70 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.36 cfs/ac/in
PEAK DISCHARGE (Q): 52.0 c.f.s.

VOLUME: 138,900 c.f. 3.19ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 1.00 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.36 cfs/ac/in
PEAK DISCHARGE (Q): 30.6 c.f.s.

VOLUME: 81,600 c.f. 1.87ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 1; 2 & 4
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
NATIVE GRASS (FAIR)	B	3.30	70	14 %
NATIVE GRASS/S.W. LANDSCAPING	A	7.10	54	30 %
IMPERVIOUS AREA		13.50	95	56 %
COMPOSITE:		23.90	79	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 1650 ft.
DROP: 27 ft.
SLOPE: 1.6 %
TIME OF CONCENTRATION: 0.19 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 0.72 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.47 cfs/ac/in
PEAK DISCHARGE (Q): 25.4 c.f.s.

VOLUME: 62,700 c.f. 1.44ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.29 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.47 cfs/ac/in
PEAK DISCHARGE (Q): 10.2 c.f.s.

VOLUME: 25,300 c.f. 0.58ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 1; 2; 3; 4; PART 5; & 6
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
NATIVE GRASS (FAIR)	B	3.30	70	8 %
NATIVE GRASS/S.W. LANDSCAPING	A	7.10	54	17 %
GRAVEL/DIRT	B	0.80	84	2 %
IMPERVIOUS AREA		30.70	95	73 %
COMPOSITE:		41.90	86	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 2150 ft.
DROP: 35 ft.
SLOPE: 1.6 %
TIME OF CONCENTRATION: 0.24 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 1.07 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.41 cfs/ac/in
PEAK DISCHARGE (Q): 63.4 c.f.s.

VOLUME: 163,400 c.f. 3.75ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.52 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.41 cfs/ac/in
PEAK DISCHARGE (Q): 30.7 c.f.s.

VOLUME: 79,100 c.f. 1.82ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 1; 2; 3; 4; 5; 6; 7 & 13
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
NATIVE GRASS (FAIR)	B	3.30	70	5 %
NATIVE GRASS/S.W. LANDSCAPING	A	11.70	54	19 %
LANDSCAPING (GOOD)	A	0.30	39	%
GRAVEL/DIRT	B	0.80	84	1 %
IMPERVIOUS AREA		46.50	95	74 %
COMPOSITE:		62.60	86	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 2700 ft.
DROP: 48 ft.
SLOPE: 1.7 %
TIME OF CONCENTRATION: 0.27 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 1.06 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.36 cfs/ac/in
PEAK DISCHARGE (Q): 90.0 c.f.s.

VOLUME: 240,400 c.f. 5.52ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.51 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.36 cfs/ac/in
PEAK DISCHARGE (Q): 43.3 c.f.s.

VOLUME: 115,600 c.f. 2.65ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 12
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
IMPERVIOUS AREA		2.00	95	100 %
	COMPOSITE:	2.00	95	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 350 ft.
DROP: 8 ft.
SLOPE: 2.2 %
TIME OF CONCENTRATION: 0.05 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 1.77 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 5.4 c.f.s.

VOLUME: 12,800 c.f. 0.29ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 1.06 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 3.2 c.f.s.

VOLUME: 7,600 c.f. 0.17ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 1; 2; 3; 4; 5; 6; 7; 12 & 13
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
NATIVE GRASS (FAIR)	B	3.30	70	5 %
NATIVE GRASS/S.W. LANDSCAPING	A	11.70	54	18 %
LANDSCAPING (GOOD)	A	0.30	39	%
GRAVEL/DIRT	B	0.80	84	1 %
IMPERVIOUS AREA		48.50	95	75 %
COMPOSITE:		64.60	86	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 3150 ft.
DROP: 86 ft.
SLOPE: 2.7 %
TIME OF CONCENTRATION: 0.26 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 1.08 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.38 cfs/ac/in
PEAK DISCHARGE (Q): 95.9 c.f.s.

VOLUME: 252,300 c.f. 5.79ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.52 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.38 cfs/ac/in
PEAK DISCHARGE (Q): 46.4 c.f.s.

VOLUME: 122,200 c.f. 2.81ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 1; 2; 3; 4; 5; 6; 7; 8; 12 & 13
CONDITION: EXISTING

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
NATIVE GRASS (FAIR)	B	3.30	70	4 %
NATIVE GRASS/S.W. LANDSCAPING	A	30.30	54	34 %
LANDSCAPING (GOOD)	A	2.50	39	3 %
GRAVEL/DIRT	B	0.80	84	1 %
IMPERVIOUS AREA		52.20	95	59 %
COMPOSITE:		89.10	78	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 3400 ft.
DROP: 92 ft.
SLOPE: 2.7 %
TIME OF CONCENTRATION: 0.27 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 0.68 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.36 cfs/ac/in
PEAK DISCHARGE (Q): 82.6 c.f.s.

VOLUME: 220,500 c.f. 5.06ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.27 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.36 cfs/ac/in
PEAK DISCHARGE (Q): 32.4 c.f.s.

VOLUME: 86,500 c.f. 1.99ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN RUNOFF SUMMARY
CONDITION: DEVELOPED

DESCRIPTION	AREA (ac)	Q-100 (cfs)	V-100 (cf.)	Q-10 (cfs)	V-10 (cf.)
1	7.60	9.8	24,200	4.3	10,700
2	25.00	61.5	160,600	36.7	96,100
4	6.00	15.7	38,500	9.4	23,000
5	5.80	7.8	18,400	3.4	8,100
6	7.00	19.2	44,900	11.4	26,900
7	10.30	20.9	49,000	11.0	25,800
8A	2.20	0.1	300	0.0	0
8	12.80	18.6	43,700	8.5	20,000
9	2.60	2.2	5,200	0.7	1,800
10	1.60	0.0	0	0.2	600
11A	8.40	19.2	44,900	10.6	24,900
11B	4.40	1.1	2,600	0.1	200
11	17.10	22.1	54,700	9.8	24,300
12	2.00	5.4	12,800	3.2	7,600
13	7.90	19.5	48,000	11.4	28,000
14	54.50	125.4	314,000	71.6	179,200
15	10.00	27.4	64,200	16.4	38,400
1; 2; 4; 5; 6; 7; 12 & 13	71.60	147.0	386,700	81.6	214,800
1; 2; 4; 5; 6; 7; 8; 12; 13 & 15	94.40	183.3	489,400	100.1	267,300

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 1
CONDITION: DEVELOPED

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
S.W. LANDSCAPING	B	3.80	70	50 %
IMPERVIOUS AREA		3.80	95	50 %
	COMPOSITE:	7.60	83	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 1550 ft.
DROP: 22 ft.
SLOPE: 1.4 %
TIME OF CONCENTRATION: 0.19 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 0.88 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.47 cfs/ac/in
PEAK DISCHARGE (Q): 9.8 c.f.s.

VOLUME: 24,200 c.f. 0.56ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.39 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.47 cfs/ac/in
PEAK DISCHARGE (Q): 4.3 c.f.s.

VOLUME: 10,700 c.f. 0.25ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 2
CONDITION: DEVELOPED

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
IMPERVIOUS AREA		25.00	95	100 %
	COMPOSITE:	25.00	95	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 850 ft.
DROP: 2 ft.
SLOPE: 0.2 %
TIME OF CONCENTRATION: 0.26 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.: 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 1.77 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.39 cfs/ac/in
PEAK DISCHARGE (Q): 61.5 c.f.s.

VOLUME: 160,600 c.f. 3.69ac.ft.

FREQUENCY: 10 yr.: 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 1.06 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.39 cfs/ac/in
PEAK DISCHARGE (Q): 36.7 c.f.s.

VOLUME: 96,100 c.f. 2.21ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 4
CONDITION: DEVELOPED

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
IMPERVIOUS AREA		6.00	95	100 %
	COMPOSITE:	6.00	95	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 1250 ft.
DROP: 13 ft.
SLOPE: 1.0 %
TIME OF CONCENTRATION: 0.19 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 1.77 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.48 cfs/ac/in
PEAK DISCHARGE (Q): 15.7 c.f.s.

VOLUME: 38,500 c.f. 0.88ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 1.06 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.48 cfs/ac/in
PEAK DISCHARGE (Q): 9.4 c.f.s.

VOLUME: 23,000 c.f. 0.53ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 5
CONDITION: DEVELOPED

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
LANDSCAPING (GOOD)	A	1.30	39	22 %
IMPERVIOUS AREA		4.50	95	78 %
COMPOSITE:		5.80	82	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 750 ft.
DROP: 26 ft.
SLOPE: 3.4 %
TIME OF CONCENTRATION: 0.08 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 0.88 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 7.8 c.f.s.

VOLUME: 18,400 c.f. 0.42ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.39 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 3.4 c.f.s.

VOLUME: 8,100 c.f. 0.19ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 6
CONDITION: DEVELOPED

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
IMPERVIOUS AREA		7.00	95	100 %
	COMPOSITE:	7.00	95	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 750 ft.
DROP: 11 ft.
SLOPE: 1.4 %
TIME OF CONCENTRATION: 0.11 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 1.77 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 19.2 c.f.s.

VOLUME: 44,900 c.f. 1.03ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 1.06 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 11.4 c.f.s.

VOLUME: 26,900 c.f. 0.62ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 7
CONDITION: DEVELOPED

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
S.W. LANDSCAPING	A	1.40	54	14 %
IMPERVIOUS AREA		8.90	95	86 %
	COMPOSITE:	10.30	89	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 950 ft.
DROP: 22 ft.
SLOPE: 2.3 %
TIME OF CONCENTRATION: 0.11 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 1.31 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 20.9 c.f.s.

VOLUME: 49,000 c.f. 1.12ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.69 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 11.0 c.f.s.

VOLUME: 25,800 c.f. 0.59ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 8A
CONDITION: DEVELOPED

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
LANDSCAPING (GOOD)	A	1.60	39	73 %
IMPERVIOUS AREA		0.60	95	27 %
	COMPOSITE:	2.20	54	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 450 ft.
DROP: 4 ft.
SLOPE: 0.8 %
TIME OF CONCENTRATION: 0.09 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 0.04 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 0.1 c.f.s.

VOLUME: 300 c.f. 0.01ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.00 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 0.0 c.f.s.

VOLUME: 0 c.f. 0.00ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: B
CONDITION: DEVELOPED

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
LANDSCAPING (GOOD)	A	2.60	39	20 %
IMPERVIOUS AREA		10.20	95	80 %
COMPOSITE:		12.80	84	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 1100 ft.
DROP: 37 ft.
SLOPE: 3.3 %
TIME OF CONCENTRATION: 0.11 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 0.94 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 18.6 c.f.s.

VOLUME: 43,700 c.f. 1.00ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.43 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 8.5 c.f.s.

VOLUME: 20,000 c.f. 0.46ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 9
CONDITION: DEVELOPED

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
LANDSCAPING (GOOD)	A	0.90	39	35 %
IMPERVIOUS AREA		1.70	95	65 %
	COMPOSITE:	2.60	76	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 900 ft.
DROP: 12 ft.
SLOPE: 1.3 %
TIME OF CONCENTRATION: 0.13 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 0.56 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 2.2 c.f.s.

VOLUME: 5,200 c.f. 0.12ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.20 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 0.7 c.f.s.

VOLUME: 1,800 c.f. 0.04ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 10
CONDITION: DEVELOPED

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
LANDSCAPING (GOOD)	A	1.50	39	94 %
IMPERVIOUS AREA		0.10	95	6 %
	COMPOSITE:	1.60	43	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 450 ft.
DROP: 14 ft.
SLOPE: 3.1 %
TIME OF CONCENTRATION: 0.05 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 0.01 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 0.0 c.f.s.

VOLUME: 0 c.f. 0.00ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.11 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 0.2 c.f.s.

VOLUME: 600 c.f. 0.01ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 11A
CONDITION: DEVELOPED

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
S.W. LANDSCAPING	A	0.70	54	8 %
IMPERVIOUS AREA		7.70	95	92 %
	COMPOSITE:	8.40	92	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 850 ft.
DROP: 20 ft.
SLOPE: 2.3 %
TIME OF CONCENTRATION: 0.10 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 1.48 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 19.2 c.f.s.

VOLUME: 44,900 c.f. 1.03ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.82 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 10.6 c.f.s.

VOLUME: 24,900 c.f. 0.57ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 11B
CONDITION: DEVELOPED

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
S.W. LANDSCAPING	A	1.20	54	27 %
LANDSCAPING (GOOD)	A	1.70	39	39 %
IMPERVIOUS AREA		1.50	95	34 %
COMPOSITE:		4.40	62	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 850 ft.
DROP: 36 ft.
SLOPE: 4.2 %
TIME OF CONCENTRATION: 0.08 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 0.16 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 1.1 c.f.s.

VOLUME: 2,600 c.f. 0.06ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.02 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 0.1 c.f.s.

VOLUME: 200 c.f. 0.00ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 11
CONDITION: DEVELOPED

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
S.W. LANDSCAPING	A	2.60	54	15 %
LANDSCAPING (GOOD)	A	1.90	39	11 %
IMPERVIOUS AREA		12.60	95	74 %
COMPOSITE:		17.10	83	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 2550 ft.
DROP: 104 ft.
SLOPE: 4.0 %
TIME OF CONCENTRATION: 0.19 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 0.88 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.47 cfs/ac/in
PEAK DISCHARGE (Q): 22.1 c.f.s.

VOLUME: 54,700 c.f. 1.26ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.39 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.47 cfs/ac/in
PEAK DISCHARGE (Q): 9.8 c.f.s.

VOLUME: 24,300 c.f. 0.56ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 12
CONDITION: DEVELOPED

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
IMPERVIOUS AREA		2.00	95	100 %
	COMPOSITE:	2.00	95	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 350 ft.
DROP: 8 ft.
SLOPE: 2.2 %
TIME OF CONCENTRATION: 0.05 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 1.77 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 5.4 c.f.s.

VOLUME: 12,800 c.f. 0.29ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 1.06 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 3.2 c.f.s.

VOLUME: 7,600 c.f. 0.17ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 13
CONDITION: DEVELOPED

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
S.W. LANDSCAPING	A	0.20	54	3 %
IMPERVIOUS AREA		7.70	95	97 %
	COMPOSITE:	7.90	94	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 1700 ft.
DROP: 35 ft.
SLOPE: 2.0 %
TIME OF CONCENTRATION: 0.18 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 1.68 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.48 cfs/ac/in
PEAK DISCHARGE (Q): 19.5 c.f.s.

VOLUME: 48,000 c.f. 1.10ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.98 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.48 cfs/ac/in
PEAK DISCHARGE (Q): 11.4 c.f.s.

VOLUME: 28,000 c.f. 0.64ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 14
CONDITION: DEVELOPED

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
LANDSCAPING (GOOD)	A	2.00	39	4 %
IMPERVIOUS AREA		52.50	95	96 %
	COMPOSITE:	54.50	93	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 2700 ft.
DROP: 99 ft.
SLOPE: 3.6 %
TIME OF CONCENTRATION: 0.21 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 1.59 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.45 cfs/ac/in
PEAK DISCHARGE (Q): 125.4 c.f.s.

VOLUME: 314,000 c.f. 7.21ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 0.91 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.45 cfs/ac/in
PEAK DISCHARGE (Q): 71.6 c.f.s.

VOLUME: 179,200 c.f. 4.11ac.ft.

PROJECT: ALBUQUERQUE INTERNATIONAL AIRPORT
PROJECT NO.: 1-051

DRAINAGE BASIN: 15
CONDITION: DEVELOPED

LAND-USE DESCRIPTION:

DESCRIPTION	SOIL GROUP	AREA (ac)	Cn	% OF TOTAL
IMPERVIOUS AREA		10.00	95	100 %
	COMPOSITE:	10.00	95	100 %

WATERSHED CHARACTERISTICS:

FLOW TYPE: GULLIED
LENGTH: 700 ft.
DROP: 48 ft.
SLOPE: 6.8 %
TIME OF CONCENTRATION: 0.06 hr.

STORM CHARACTERISTICS:

FREQUENCY: 100 yr.; 6 hr.
RAINFALL: 2.3 in.
RUNOFF: 1.77 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 27.4 c.f.s.

VOLUME: 64,200 c.f. 1.47ac.ft.

FREQUENCY: 10 yr.; 6 hr.
RAINFALL: 1.6 in.
RUNOFF: 1.06 in.
DISTRIBUTION CURVE: 75
DISCHARGE: 1.55 cfs/ac/in
PEAK DISCHARGE (Q): 16.4 c.f.s.

VOLUME: 38,400 c.f. 0.88ac.ft.

APPENDIX B:

CATCH BASIN CAPACITY CALCULATIONS

CATCH BASIN CAPACITY CALCULATIONS:

1) Single "C" & Single "D" (Sag-point & Low-point):

$Q = vA = C_v\sqrt{2gh}\ A = 14.4$ cfs*

where $C_v = 0.67$

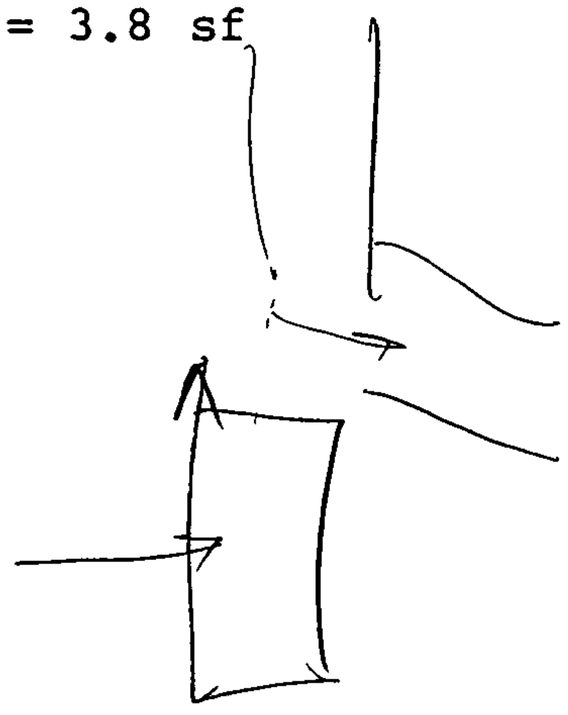
$g = 32.2$ ft/sec^2

$h = 0.5'$

$A = (15\ 1/2")(35\ 3/8") = 548$ in^2 $= 3.8$ sf

2) Single "C" & Single "D" (Curb-line):

$Q_{(\text{curb line})} \approx 1/3\ Q_{(\text{lo-point})} = 5$ cfs*



3) 3' x 0.5' Weir:

$Q = 3.33\ LH^{1.5} = 3.5$ cfs

where $L = 3'$ and $H = 0.5'$

4) 5.5' x 1.0' Weir:

$Q = 3.33\ LH^{1.5} = 18.3$ cfs

where $L = 5.5'$ and $H = 1.0'$

5) 7' x 0.5' Weir:

$Q = 3.33\ LH^{1.5} = 8.2$ cfs

where $L = 7'$ and $H = 0.5'$

However analyzing as an orifice,

$Q = C_v\sqrt{2gh}\ A = 16.4$ cfs

where $C_v = 0.90$

$g = 32.2$ ft/sec^2

$h = 0.67 - 0.25 = 0.42'$

$A = 0.5(7) = 3.5$ sf

6) 2' x 3' Grate:

$Q = C_v\sqrt{2gh}\ A = 5$ cfs

where $C_v = 0.67$

$g = 32.2$ ft/sec^2

$h = 0.1'$ (estimated)

$A' = 2(3) = 6$ sf

$A = 1/2\ A' = 3$ sf (effective area)

* Double value for double "C" or double "D".

7) 2' x 2' Grate:

$$Q = C_v \sqrt{2gh}\ A = 3.4 \text{ cfs}$$

where $C_v = 0.67$
$g = 32.2 \text{ ft/sec}^2$
$h = 0.1'$ (estimated)
$A = 1/2(2)(2) = 2 \text{ sf}$

8) 2'x 2' Grate (CB-18):

$$Q = C_v \sqrt{2gh}\ A = 7.6 \text{ cfs}$$

where $C_v = 0.67$
$g = 32.2 \text{ ft/sec}^2$
$h = 0.5'$ (field determined)
$A = 1/2\ (2)(2) = 2 \text{ sf}$

9) 3' x 3' Grate:

$$Q = C_v \sqrt{2gh}\ A = 12 \text{ cfs}$$

where $C_v = 0.67$
$g = 32.2 \text{ ft/sec}^2$
$h = 0.25'$ (field determined)
$A = 1/2\ (3)(3) = 4.5 \text{ sf}$

10) Cattle Guard:

$$Q = C_v \sqrt{2gh}\ A = 1.27 \text{ cfs/LF}$$

where $C_v = 0.50$
$g = 32.2 \text{ ft/sec}^2$
$h = 0.1'$ (estimated)
$A = 1/2\ (1)(2) = 1 \text{ sf}$

For 24' cattle guard,

$Q = 24(1.27) = 30.5 \text{ cfs} \simeq 30 \text{ cfs}$

For 36' cattle guard,

$Q = 36\ (1.27) = 45.7 \text{ cfs} \simeq 45 \text{ cfs}$

11) 2' x 3' Grate (Phase II):

$$Q = C_v \sqrt{2gh}\ A = 11.4 \text{ cfs}$$

where $C_v = 0.67$
$g = 32.2 \text{ ft/sec}^2$
$h = 0.5'$
$A = 1/2\ (2)(3) = 3 \text{ sf}$

ALBUQUERQUE INTERNATIONAL AIRPORT/ UNIVERSITY GOLF COURSE DRAINAGE STUDY

(A/E 82-14)

ALBUQUERQUE
NEW MEXICO
1706

Tom Mann Associates, Inc.
811 DALLAS, N E • ALBUQUERQUE • NEW MEXICO • 87110
ENGINEERS

Tom Mann Associates, Inc.
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20441
July 30, 1982

Mr. Robert Gill
Aviation Supervisor
Albuquerque International Airport
P. O. Box 9022
Albuquerque, New Mexico 87119

Re: Albuquerque International Airport/University
Golf Course Drainage Study (A/E 82-14)

Dear Bob:

Transmitted herewith are five (5) copies of the subject drainage report. Accompanying the five (5) copies is one copy of the compiled computer print-outs for Aviation Department use. Also transmitted at this time is the aerial photograph (circa 1972) which you provided to assist in this study.

Due to the multitude of data presented in this report, reviewers may request a conference to explain the contents and findings of this study effort. Such a meeting can be arranged upon request.

If you have any questions or comments concerning this submittal, do not hesitate to call.

Sincerely,

TOM MANN & ASSOCIATES, INC.

Jeffrey G. Mortensen
Project Engineer

Thomas T. Mann, Jr., P.E.
President

JGM:bb
Encs.

ALBUQUERQUE INTERNATIONAL AIRPORT/UNIVERSITY GOLF COURSE

DRAINAGE STUDY (A/E 82-14)

JULY 1982

PREPARED FOR: AVIATION DEPARTMENT
CITY OF ALBUQUERQUE
P. O. BOX 9022
ALBUQUERQUE, NEW MEXICO 87119

PREPARED BY: TOM MANN & ASSOCIATES, INC.
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SYMBOLS AND ABBREVIATIONS

A	Cross-sectional area, L^2
AMAFCA	Albuquerque Metropolitan Arroyo Flood Control Authority
BKD	SCS soil group; Bluepoint-Kokan Association
BuREC	Bureau of Reclamation
C	Cropping - Management factor
C_h	Hazen-Williams friction coefficient, dimensionless
C_w	Weir coefficient, constant
Cu	SCS soil group; Cut and Fill Land
C_v	Orifice discharge coefficient, constant
cfs	cubic feet per second
CMP	Corregated metal pipe
CN	Curve number
g	Acceleration of gravity, L/T^2
H	Head on weir, L
h	Head above orifice, L
K	Soil-erodibility factor
L	Weir length, L
LS	Slope length and gradient factor
n	Manning roughness coefficient, constant
NMSHD	New Mexico State Highway Department
NOAA	National Oceanic and Atmosphere Administration
P	Erosion control practice factor
P_w	Wetted perimeter, L
Q	Discharge rate, L^3/T
R	Hydraulic radius = $\frac{A}{P}$, L

RCP	Reinforced concrete pipe
s	Slope, L/L
SCS	Soil Conservation Service
U.N.M.	University of New Mexico
U.S.D.A.	United States Department of Agriculture
U.S.G.S.	United States Geologic Survey
V	Volume, L^3
v	Velocity, L/T
WaB	SCS soil group; Wink fine sandy loam

ACKNOWLEDGEMENTS

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Robert Gill, Airport Supervisor, Albuquerque International Airport

Alfred Kline, Superintendent, University of New Mexico South Golf Course

Dick McGuire, Coach, University of New Mexico South Golf Course

Jim Monaghan, Soil Conservation Service

Ralph Pike, National Weather Service

I. SUMMARY

I. SUMMARY

This report has been prepared for the City of Albuquerque Aviation Department to study the hydrology of the General Aviation/Golf Course drainage basin. This basin, which is situated in the Southeast portion of the City of Albuquerque, drains east to west and measures approximately 0.4 square miles. The runoff from the study area drains to a point along the east boundary of the University of New Mexico South Golf Course where it enters the Golf Course. This point of entry is the primary focus of hydrologic analysis.

State-of-the-art hydrologic computer techniques are used to model the watershed in both existing and developed conditions. Conventional hydraulic formulae are used to assist in the development of the hydrologic models. The models are discussed along with all relevant assumptions. Lastly, a comparison of existing and developed runoff quantities and frequency is made.

In addition to the hydrologic modeling, sediment yield is estimated. This is accomplished by using the modified Universal Soil Loss Equation and the Meyer-Peter, Muller formula to determine washload and bedload respectively. As in the case of the hydrologic analysis, existing and developed conditions are compared.

Following the review of the study area hydrology and sediment yield determinations, these conclusions are presented:

1. The damage to the Golf Course Fairways 2 and 8 is largely due to the failure to integrate the arroyo into the Golf Course layout. Fairways 2 and 8 were constructed across an existing arroyo.

2. The development of the General Aviation area has increased the peak discharge of runoff, the overall runoff volume and the frequency of runoff generated by the study area. These increases have, to some degree, accelerated the deterioration of Fairways 2 and 8.

3. Consideration must be given to the development of the remaining portions of the study area. This development will almost certainly increase the quantity and frequency of runoff which impacts the Golf Course.

4. In view of the preceding statement, any storm drainage improvements intended to relieve the current problems should take place within the Golf Course, or perhaps just east of the Golf Course. Any improvements implemented further upstream could very easily be bypassed by future development, thereby rendering them a futile expense of time, effort and money. In addition, upstream improvements would not correct the most serious problem, the failure to integrate the arroyo into the layout of the golf course.

II. INTRODUCTION

II. INTRODUCTION

BACKGROUND

The University of New Mexico South Golf Course opened to the public in December, 1966 (6). The newly constructed golf course interrupted the flow of an existing arroyo with Fairway 2 and Fairway 8. As a consequence, runoff entering the golf course from the east eventually became a concern to the Golf Course. In an attempt to keep potentially damaging runoff from crossing Fairway 2, an earthen dike was constructed in 1966 by the Golf Course between Fairway 2 and Fairway 8 when the golf course opened (7). Again, the purpose of this dike was to collect runoff and prevent it from crossing Fairway 2.

As time proceeded, a portion of Fairway 8 was lost to sedimentation (7) which was mobilized by upstream runoff. This caused the Golf Course to construct a second earthen dike between Fairways 2 and 8 (7). These dikes were constructed by the T.V.I. construction classes at no charge to the University.

During the Spring of 1980, the dikes failed during a period of fairly heavy runoff (5). The dikes were rebuilt with one CMP culvert per each dike to permit the dams to drain (4, 5). These rebuilt dikes failed in the Fall of 1980 (5). The dikes were rebuilt once more, this time with both culverts in the downstream dike which lies along the east edge of Fairway 2 (4, 5). These last repairs failed shortly after completion in the Fall of 1981 (5). No subsequent repairs were made to the breached dikes.

Since the last failure, runoff has been permitted to pass through the golf course. As a result, the golf course has been damaged by scouring and sedimentation. Approximately six feet of sand has been deposited on Fairway 8 since 1966 (4), the "rough" along the west edge of Fairway 2 is being progressively eroded, and, due to the passage of runoff across Fairway 2, the west bank of the arroyo bordering Fairway 17 is beginning to erode, thereby presenting a potential threat to Fairway 17.

To further complicate matters, a "new" General Aviation Facility was constructed for the Albuquerque International Airport in the upstream portion of the contributing watershed. The construction of this facility was completed in the fall of 1979 (8). The effect of this improvement within the watershed was not clearly known, nor evaluated at that time. It is because of this that this study has been undertaken to more clearly understand the hydrology of the study area.

PURPOSE AND SCOPE

This report has been prepared to study the hydrology of the General Aviation/Golf Course drainage basin. The overall study area is shown by Figure 1. The basin measures slightly less than 0.4 square mile and it drains west from Runways 12-30 and 3-21 to the east boundary of the University of New Mexico South Golf Course. That portion of the basin which lies between the above limits includes the "new" General Aviation area, Robertson's Aircraft Service, and a portion of Ross

Aviation. Figure 2 is a map which delineates property ownership in conjunction with the developed drainage basin boundary. Ownership information was obtained from the Bernalillo County Assessor's Office records.

The analysis of the hydrology of the study area includes the analysis of the "existing" and "developed" conditions. The existing condition refers to that period in time which predates the construction of the new General Aviation facilities. In following, the "developed condition" acknowledges the presence of the newly constructed General Aviation facilities.

The hydrologic parameters determined and discussed in this report are the peak discharge rates, runoff volumes, and sediment loads associated with the 2-, 5-, 10-, 25-, 50- and 100-year, 6-hour storms falling within the study area under both the existing and developed conditions. The methods, technology, assumptions and data base, all of which were incorporated into the study of the General Aviation/Golf Course drainage basin, are discussed in detail in the following section. Lastly, recommendations are presented by which the runoff which poses a maintenance problem to the Golf Course can be handled.

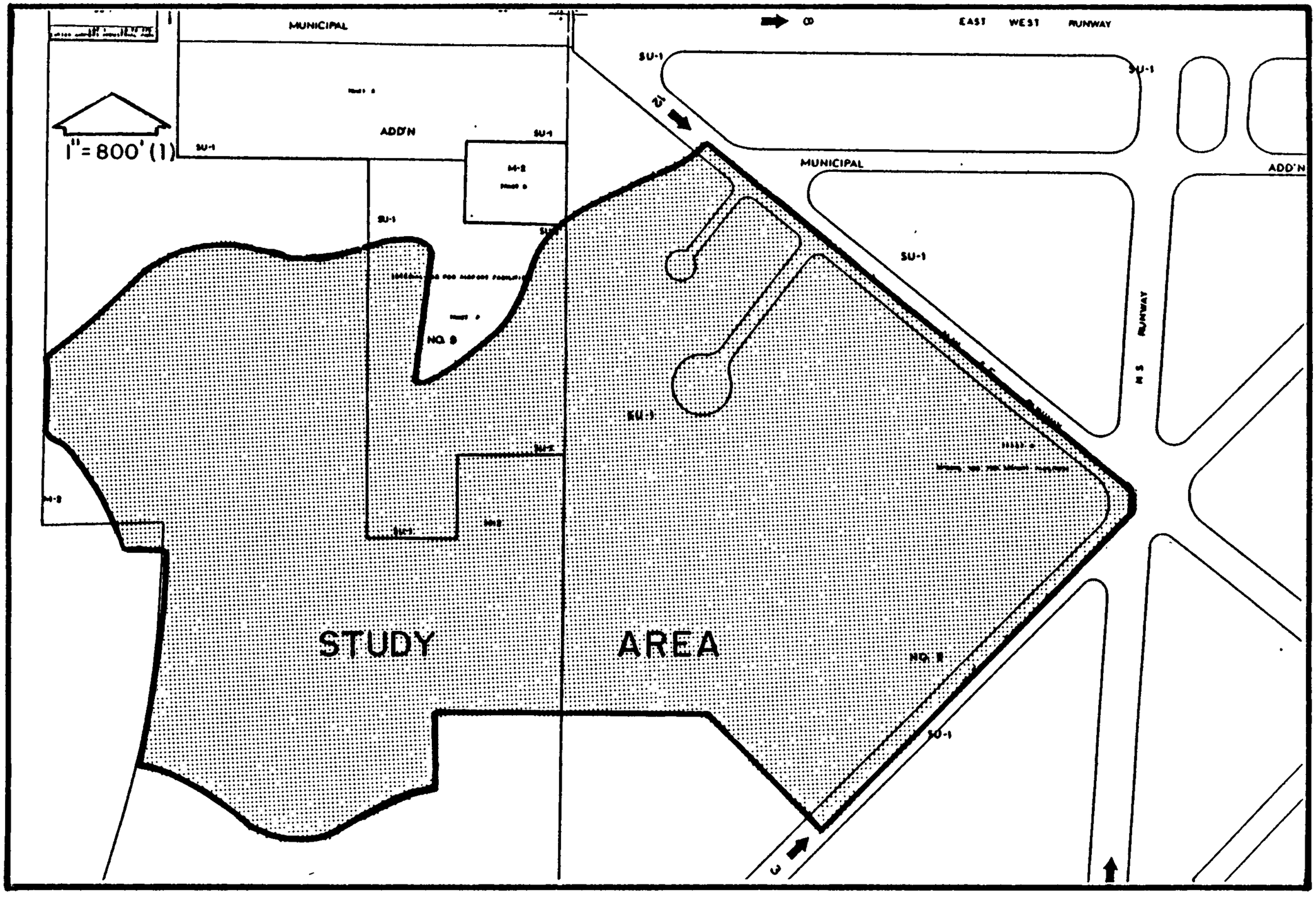
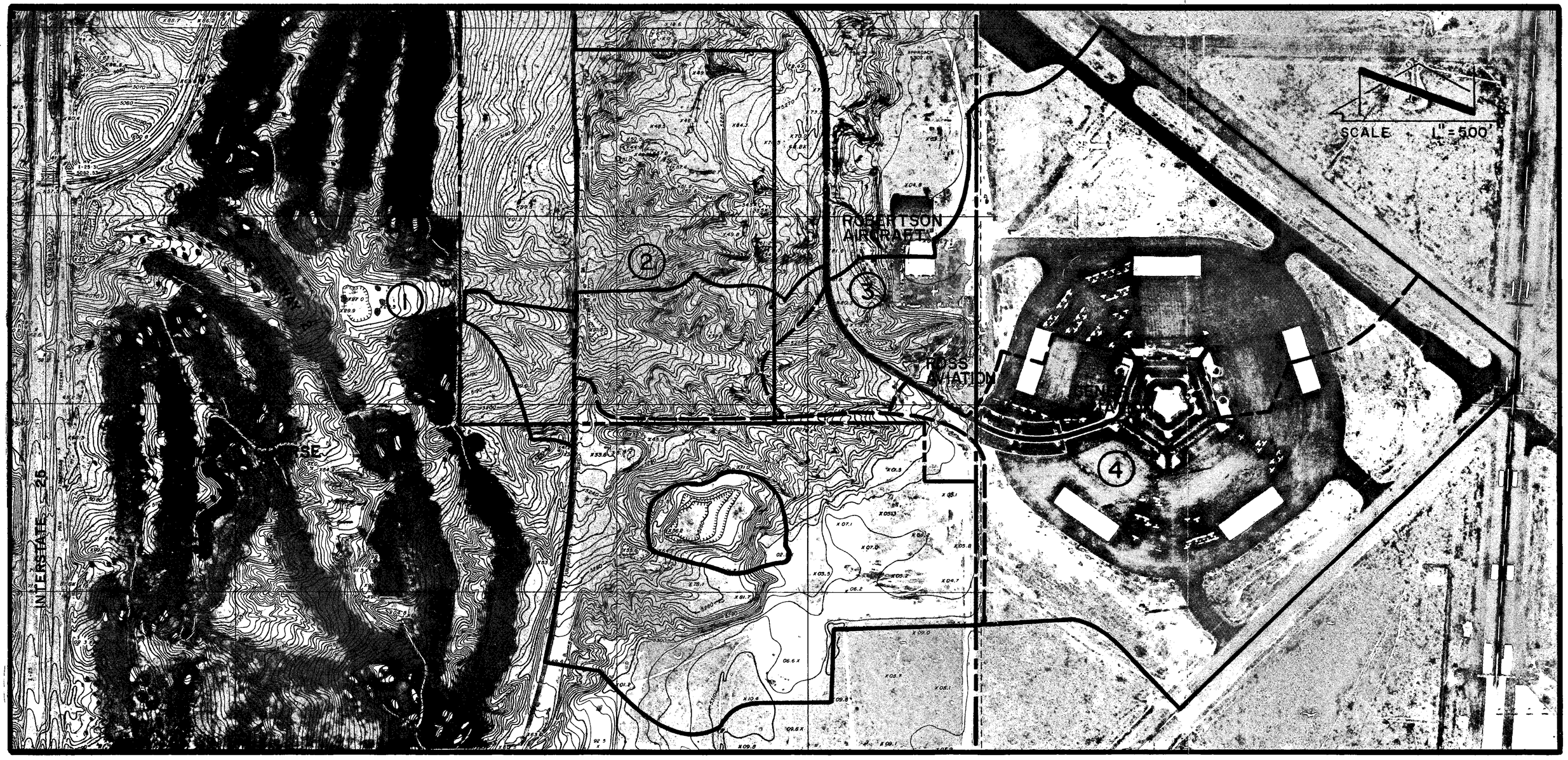


FIGURE 1: VICINITY MAP



LEGEND

- DRAINAGE BASIN BOUNDARY
- DRAINAGE SUB AREA BOUNDARY
- PROPERTY LINE
- (0) LAND OWNERSHIP IDENTIFICATION

LAND OWNERSHIP

1. UNIVERSITY OF NEW MEXICO
2. CUTTER ENTERPRISES
3. CITY OF ALBUQUERQUE
4. UNITED STATES GOVERNMENT

FIGURE 2 : MAP OF LAND OWNERSHIP SUPERIMPOSED WITH DEVELOPED DRAINAGE BASIN.

III. HYDROLOGIC AND HYDRAULIC ANALYSES

III. HYDROLOGIC AND HYDRAULIC ANALYSES

The hydrologic and hydraulic analyses performed for this study incorporated digital computer modeling as well as the use of conventional hydraulic formulae. The results from these analyses were used to evaluate both existing and developed drainage conditions and assisted in the development of several drainage improvement alternates by which the current drainage problems can be controlled. Specific results are discussed in more detail in the appropriate following sections.

The computer model used for this study is the same that was used in the Albuquerque Master Drainage Study (1). The program, HYMO, was developed by Williams and Hann (21) and is explained in detail in the Albuquerque Master Drainage Study as it applies to the Albuquerque area. The basic assumptions in the analysis are as follows:

1. Rainfall is distributed uniformly over the entire watershed. The n-year, 6-hour rainfall is used for analysis. Rainfall estimates are tabulated below.

n	Rainfall, inches (9)
100	2.3
50	2.15
25	1.8
10	1.5
5	1.3
2	1.0

2. The total 5-day antecedent rainfall is greater than 1.4 inches and less than 2.1 inches (1).
3. The basic unit hydrograph of runoff is triangularly shaped such that one-third of runoff volume occurs

before the hydrograph peak and the remaining two-thirds of the runoff volume occurs after the peak. This is a basic assumption employed by the Soil Conservation Service.

4. The hydrology of the study area was not altered during the study. This assumes that land use and/or development did not change since the date of the mapping used for this study. The date of the mapping is October 8, 1980. Field inspection of the area revealed substantial agreement between present day conditions and the October 8, 1980 photography. Earlier records have been used, when available, to represent "pre-development" conditions.

5. The natural depressions which serve as ponds are "dry" at the time of the design storm. Several natural ponding areas exist throughout the study area. Runoff from contributing areas is routed through these ponds. For the convenience of analysis, the ponds are assumed to be empty and "dry" at the onset of the design storm.

Total sediment yield delivery was computed by two separate methods. The methods used were the SEDIMENT YIELD HYMO routine for washload and the Meyer-Peter, Muller formula for bedload. In each case, sediment yield is expressed in tons. Total sediment yield is the sum of the two results.

The SEDIMENT YIELD routine in HYMO is based upon the modified Universal Soil Loss Equation. The modified equation is

$$S = 95(Q \times V)^{0.56} \times K \times C \times P \times LS,$$

where S = sediment yield in tons,
Q_p = peak flow rate in cubic feet per second,
V = volume of runoff in acre-feet,
K = the soil-erodibility factor,
C = the cropping-management factor,
P = the erosion control practice factor,
LS = the slope length and gradient factor.

Detailed instructions for determining K, C, P, and LS are given by the Soil Conservation Service (15). Assistance in the determination of these factors was offered by Mr. Jim Monaghan, SCS (10). Additional assistance in the determination of "C" and "K" was offered by Mr. Jimmy R. Williams, U.S.D.A. (19, 20). The determination of these factors appears in Appendix C.

The computation of bedload transport, by the Meyer-Peter, Muller method is based upon the following formula:

$$g_s = 1.606[3.306(\frac{Q_s}{Q})(\frac{D_{90}^{1/6}}{n_s})^{3/2} dS-0.627\ D_m]^{3/2} \text{ tons/day/ft. width.}$$

This equation is explained in more detail by Sheppard (13). A sample calculation sheet is shown by Figure 3. The nomographs to which Figure 3 refers are found in the publication by Shepppard (13). The computation of bedload transport in both the existing and developed conditions appears in Appendix D.

The annual sediment yield was determined based upon the work of Simons, et al (14). This method, by which annual sediment yield is calculated, multiplies the median storm sediment yield by the ratio of the annual runoff to the median storm

COMPUTATION OF BEDLOAD TRANSPORT BY MEYER-PETER, MULLER FORMULA FOR ONE DISCHARGE

River_____ Computed by_____

Date_____ Checked by_____

Time_____ Date_____

Section_____ Sheet_____ of_____

FORMULA

$$g_s = 1.606 \left[3.306 \left(\frac{Q_s}{Q}\right) \left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} dS - 0.627\ D_m\right]^{3/2} \text{ tons/day/ft. width}$$

HYDRAULIC DATA

Q - Discharge = _____ cfs

d - depth = _____ ft

B - width = _____ ft

S - slope = _____ ft/ft.

n_m - total stream "n" = _____

n_w - channel sides "n" = _____

SIZE ANALYSIS - BED MATERIAL

Size Fraction mm	Geometric Mean D mm	ΔP Percent	DΔP
< .0625			
.0625 - .125	0884		
125 - 25	.177		
.25 - .5	.354		
.5 - 1.0	.707		
1 0 - 2 0	1 41		
2 0 - 4 0	2 83		
4 0 - 8 0	5 66		
8 0 - 16 0	11. 32		
16 0 - 32 0	22.63		
Total			

BEDLOAD TRANSPORT COMPUTATIONS

FIGURE 1

$\frac{n_w}{n_m}$ = ____ = _____

$\frac{d}{B}$ = ____ = _____

n_s (from Nomograph) = _____

$D_m = \frac{\Sigma D \Delta P}{100}$ = _____

D_{90} = _____ mm

$D_{90}^{1/6}$ = _____

$\left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} = (\quad)^{3/2} = \quad^{3/2} =$ _____ = _____

$g_s = 1.606\ [3.306(\quad)(\quad)\quad\quad -0.627\quad\quad]^{3/2}$

$g_s = 1.606\ [\quad\quad -\quad\quad]^{3/2}$

$g_s = 1.606\ [\quad\quad]^{3/2}$

$g_s = 1.606$ _____

g_s = _____ tons / day / ft. width

G_s = _____ (B) = _____ tons / day

FIGURE 2

$\frac{n_w}{n_s}$ = ____ = _____

$\frac{Q_s}{Q}$ (from Nomograph) = _____

FIGURE 3 : Sample Computation Sheet - Meyer-Peter, Muller Formula (13)

runoff as demonstrated by the following equations:

$$V_a = P_a \times \%R\text{-}0 \times A_{ws}$$

where V_a = annual volume of runoff, ac-ft
P_a = annual rainfall, feet
%R-O = annual percent of runoff, decimal
A_{ws} = area of watershed, acres

and,

$$SY_a = SY_m \times \left(\frac{V_a}{V_m}\right)$$

where SY_a = annual sediment yield, tons
SY_m = median sediment yield, tons
V_m = median volume of runoff, ac-ft

Annual sediment yield is calculated for both the existing and developed conditions. These calculations appear in Appendix E.

Samples of the arroyo bed were taken so that grain size distribution curves could be developed. Two samples were taken in all. Each was obtained in conformance with the following distribution:

0-6" depth	40%
6-12" depth	30%
12-18" depth	20%
18-24" depth	10%

The grain size distribution curves appear in Appendix D.

The determination of the capacity of channels and arroyos was accomplished by using the Manning equation:

$$Q = \frac{1.49}{n} A R^{2/3} s^{1/2}$$

where Q = flow in cfs

n = Manning roughness coefficient, dimensionless.

A = Flow cross-sectional area in square feet.

R = Hydraulic radius, in feet, which is the ratio of the flow cross-sectional area to the wetted perimeter of the flow cross section.

s = Energy grade line slope (which is assumed equal to the slope of the channel) expressed as a decimal.

The discharge from natural ponding areas was calculated using the weir equation:

$$Q = C_w L H^{1.5}$$

where Q = flow in cfs.

C_w = weir coefficient (assumed to be 3 for this study), constant.

L = length of weir, feet.

H = height of weir, feet.

The capacity of and travel time through the storm drain network in and around the "new" General Aviation area were determined by the Hazen-Williams equation for uniform culvert flow. The Hazen-Williams equation for uniform flow is as follows:

$$Q = 1.318\ AC_h R^{0.63}\ s^{0.54}$$

where Q = flow in cfs,

A = flow cross-sectional area in square feet.

C = Hazen-Williams friction coefficient, dimensionless.

R = hydraulic radius, in feet, which is the ratio of the flow cross-sectional area to the wetted perimeter of the flow cross section.

s = energy grade line slope (which is assumed equal to the slope of the pipe) expressed as a decimal.

Travel time was approximated with the pipe flowing at two-thirds capacity. Values of 'C_h' used in this analysis are 150 for reinforced concrete pipe (RCP) and 120 for corregated metal pipe (CMP). Supporting calculations appear in Appendix A.

The capacity of road crossing culverts was analyzed by using the Bureau of Public Roads charts for inlet control (3). Figure 4 shows these charts as they apply to RCP and CMP culverts.

The following hydraulic assumptions were also incorporated into this study.

1. Channels, catch basins and culverts are free of debris and in good repair.
2. Catch basins and culverts have been properly sized and located.
3. Information extracted from the 1 inch = 500 feet topographic maps are reasonably accurate representations of actual field conditions. This refers to the determination of channel crossing sections and drainage basin areas. Cross sections were used in calculating hydraulic capacities, determining flood plain limits, and performing flood routing routines in HYMO.

4. The use of Manning's roughness coefficient 'n' is summarized as follows:

Material	n
Corregated Metal Pipe	0.025
Reinforced Concrete Pipe	0.015
Paved Streets	0.020
Concrete Channels	0.015
Earth Channels	0.025

The hydrologic and hydraulic analyses were performed using the best available data. This data, which is described below, was supplemented with several field visits to verify questionable field conditions. The data base is summarized as follows:

1. Topographic Mapping. The majority of the Topographic mapping compiled in this study effort was 1 inch = 500 feet photo based topographic maps prepared by Bohannan-Huston, Inc. on October 8, 1980. The uppermost portion of the study area, however, is not mapped topographically. Because of this, it was necessary to rely upon other sources of mapping. The other sources were record construction drawings (8), City of Albuquerque Topographic Orthophoto Map No. N-15 prepared May 31, 1979, by Bohannan-Huston, Inc. and field excursions. No field surveys were performed for this study.
2. Rainfall Data. Rainfall data was taken from the National Oceanic and Atmospheric Administration (NOAA) publication "NOAA Atlas 2: Precipitation-Frequency Atlas of the Western United States, Volume IV - New Mexico" (9).

The n-year, 6-hour rainfall depths for the study area are presented earlier in this section.

3. Soils. Soils information was taken exclusively from U.S.D.A. Soil Conservation Service publications (16, 17, 18). The soils found within the study area are shown by Figure 5. Table 1 lists those soils as well as corresponding hydrologic soil complex classifications and curve numbers.

4. Land Use and Vegetation. The study area land uses and vegetal cover were determined through the combination of map review and field reconnaissance.

5. Storm Drainage Facilities. The locations and dimensions of existing storm drainage facilities were obtained from the review of record construction drawings (8) and supplemented by field investigation. Information was then plotted on the 1:500 mapping of the study area.

6. Sediment Yield. Information relating to the determination of sediment yield was obtained from SCS publications and personnel (10, 15, 16, 19, 20).

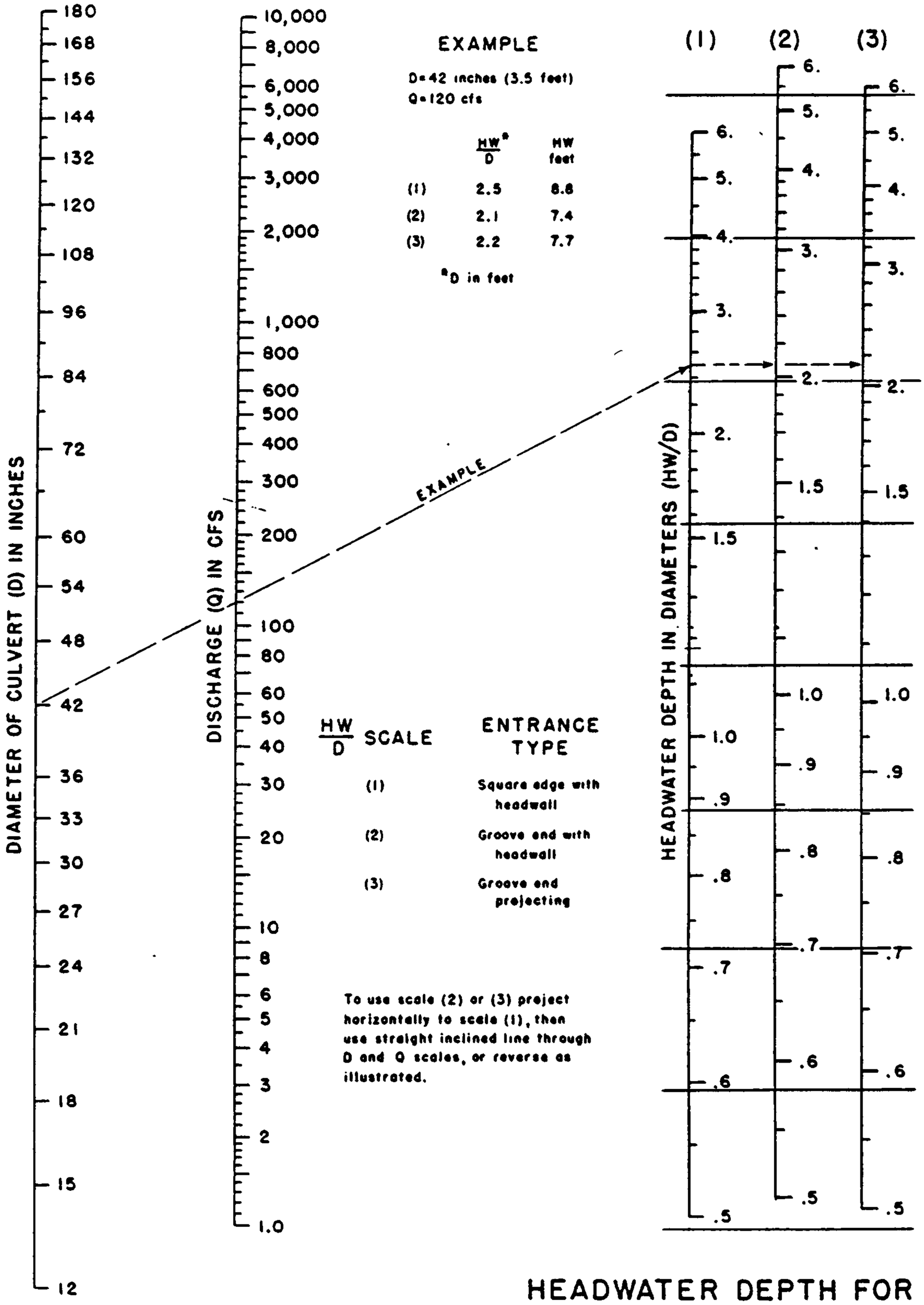


FIGURE 4a: RCP Culvert Capacity Chart for Inlet Control (3)

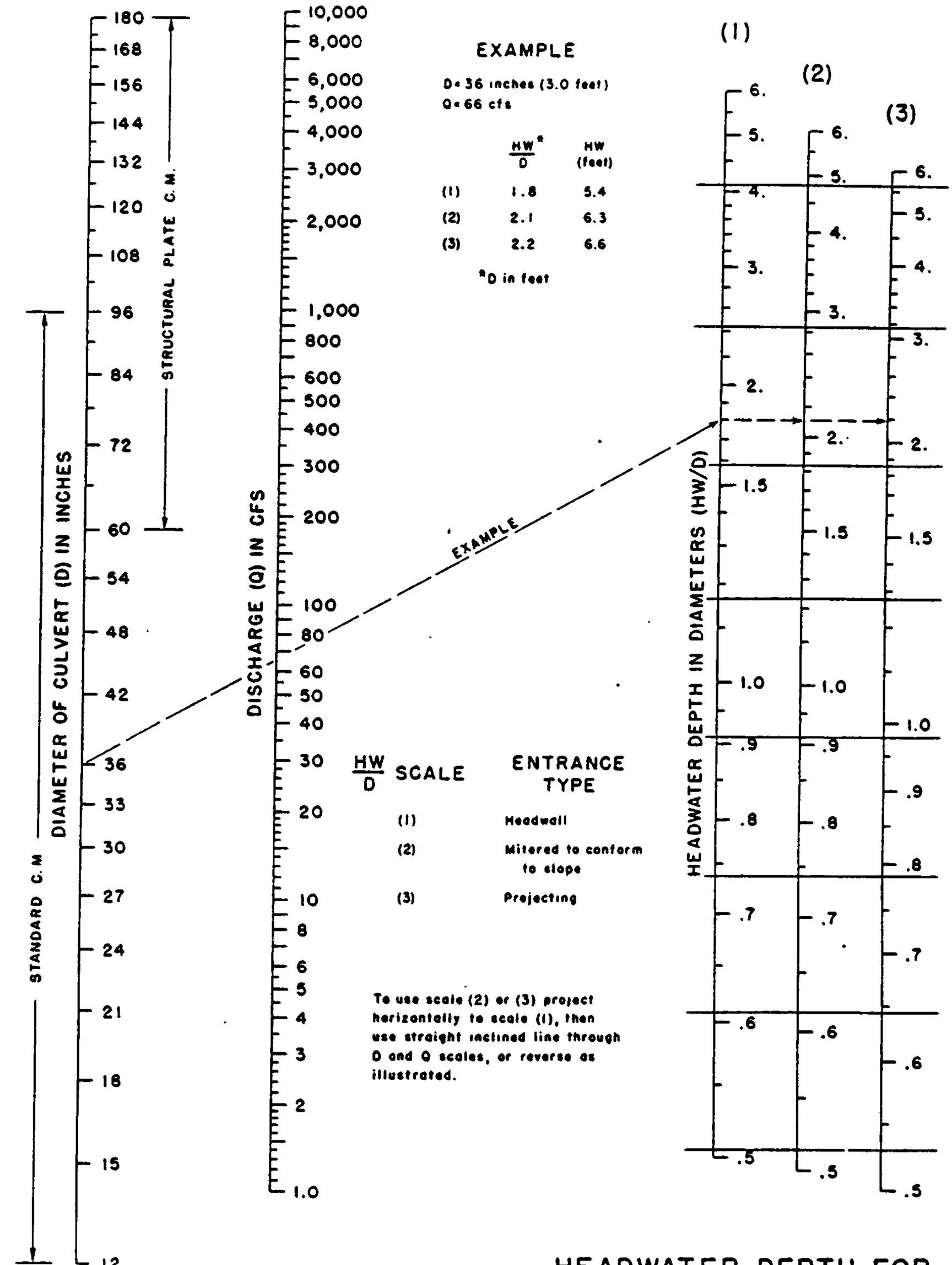
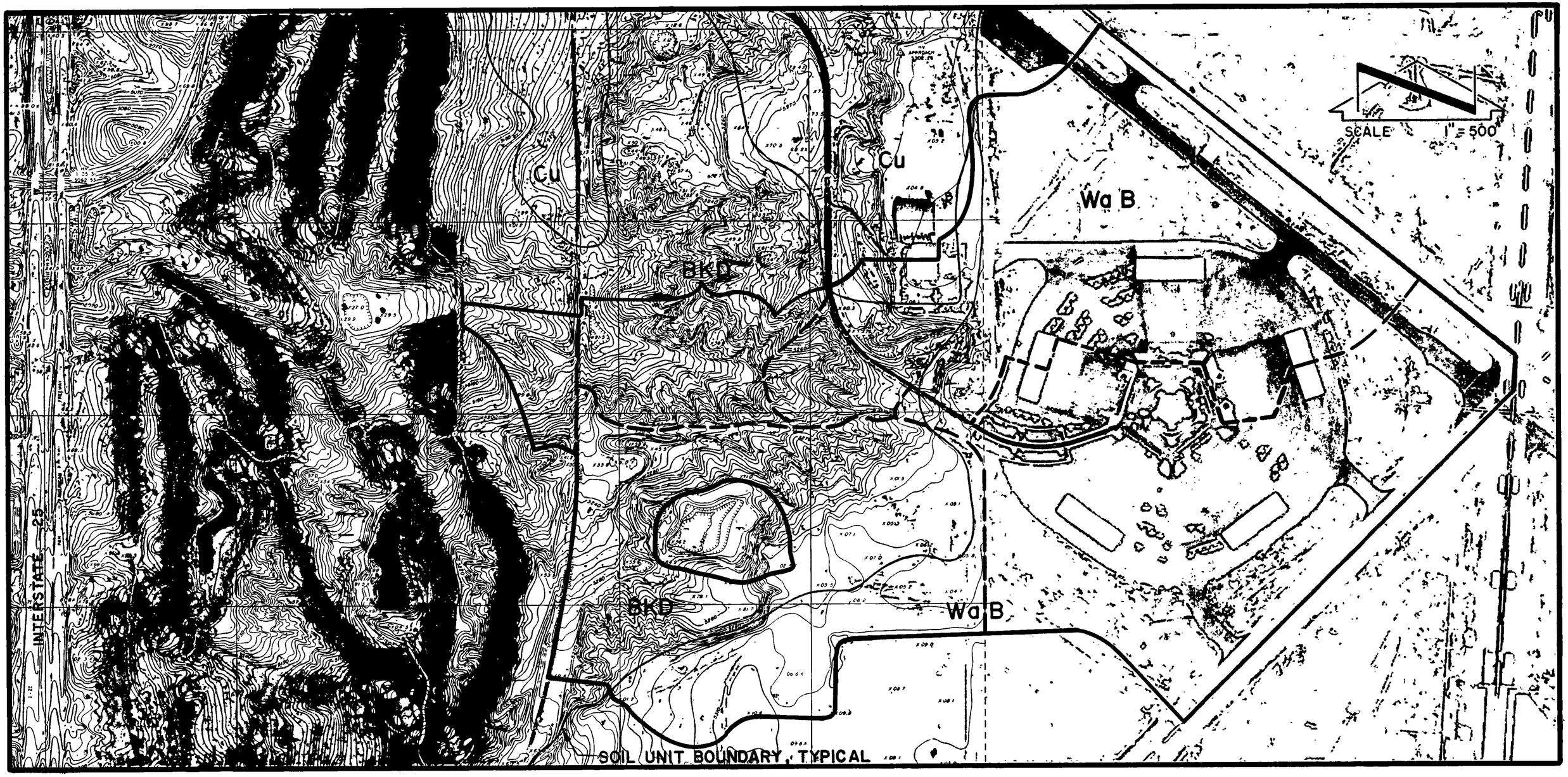


FIGURE 4b: CMP Culvert Capacity Chart for Inlet Control (3)



LEGEND

BKD	BLUEPOINT-KOKAN ASSOCIATION
Cu	CUT AND FILL LAND
WaB	WINK FINE SANDY LOAM

SOURCE : "SOIL SURVEY OF BERNALILLO COUNTY AND PARTS OF SANDOVAL AND VALENCIA COUNTIES", PLATE 41 (16).

SOILS INFORMATION

SOIL	COMPLEX	CN
BKD	A	68
Cu	A	68
WaB	B	79

FIGURE 5 : MAP OF STUDY AREA SOILS SUPERIMPOSED WITH DEVELOPED DRAINAGE BASIN.

Soil Unit	Map Symbol	Hydrologic Soil Complex	Curve Number (CN)
Bluepoint-Kokan Association	BKD	A	68
Cut and Fill Land	Cu	A	68
Wink Fine Sandy Loam	WaB	B	79

References: "Soil Survey of Bernalillo County and Parts of Sandoval and Valencia Counties, New Mexico" (16) and "Peak Rates of Discharge for Small Watersheds, Chapter 2" (18).

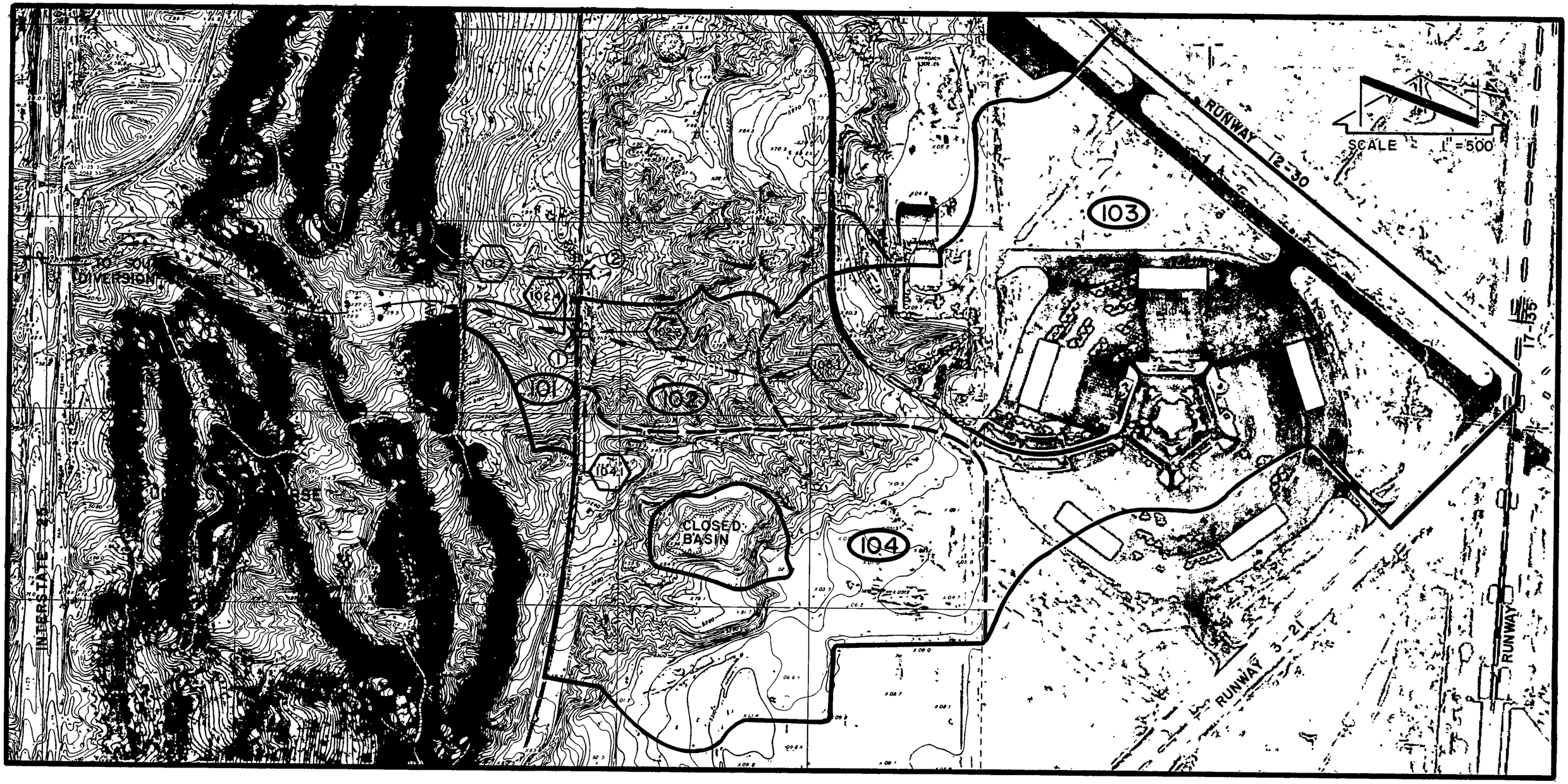
TABLE 1: Study Area Soils

IV. EXISTING DRAINAGE CONDITION

IV. EXISTING DRAINAGE CONDITION

The "existing" drainage condition, as previously discussed, refers to those conditions which prevailed just prior to the construction of the "new" General Aviation facility. The existing drainage basin is shown by Figure 6. The boundary of the basin is superimposed over the photo base for the present-day condition. Much of the study area has not changed significantly, therefore, the majority of the drainage basin boundary was easily delineated. The uppermost portion of the basin, however, has experienced construction. The construction drawings (8) for these improvements recorded the preconstruction conditions and therefore enabled the reconstruction of the remainder of the existing drainage basin.

The existing drainage basin measures 0.342 square miles in size and drains east to west. The majority of the study area drains via sheet and gully flow to an arroyo trending east/west within sub-area 102. The flows from this arroyo are collected by an existing 60-inch CMP culvert under University Boulevard S.E. and discharged into sub-area 101. From this point, runoff is conveyed by the downstream portion of the arroyo to the east boundary of the Golf Course. Within the Golf Course, the arroyo widens and drains across Fairway 8 into the two diked areas described in the preceding section. The diked areas measure approximately 2/3 acre with a maximum depth of about 6 feet. Runoff leaving the breached dikes



LEGEND

- FLOW PATH
- DRAINAGE BASIN BOUNDARY
- DRAINAGE SUB-AREA BOUNDARY
- 000 DRAINAGE SUB-AREA IDENTIFICATION
- 000.0 ANALYSIS POINT & ASSOCIATED HYDROGRAPH
- ROAD CROSSING CULVERT

KEYED NOTES

① 60" CMP CULVERT

② 36" CMP CULVERT

FIGURE 6 : EXISTING DRAINAGE CONDITION

continues to flow in a westerly direction across Fairway 2 to a continuation of the aforementioned arroyo. The continuation of the arroyo drains to two existing concrete box culverts which flow under Interstate 25. From this point, runoff is discharged into an arroyo which ultimately discharges into the AMAFCA South Diversion Channel.

The drainage boundaries and patterns shown on Figure 6 were determined from the available mapping and supplemented by field observation whenever uncertainties arose. Soil types and ground cover conditions were evaluated using Figure 5, Figure 6 and the SCS Engineering Field Manual (18). Curve numbers for the study area are listed in Table 1. For the purpose of analyzing the existing condition, a weighted curve number was assigned to each area. The computation of the weighted curve numbers is presented in Appendix B.

The hydrologic modeling of the study area in the existing condition was accomplished using HYMO. HYMO, as discussed earlier, is a digital computer model developed by the U.S.D.A. Agricultural Research Service and adopted by the City of Albuquerque. The entire capabilities of HYMO are numerous; most of the program routines were used in this study. The program and its capabilities are discussed in further detail by Williams and Hann (21).

The HYMO routines employed in this facet of the study permitted the computation of hydrographs; the routing of hydrographs through downstream basins, culverts or ponds; the addition of hydrographs; and the determination of the washload

sediment yield. All of this was performed on a per area basis thus enabling the study of the watershed at strategic intermediate analysis points. These analysis points and sub-areas are delineated on Figure 6. The analysis points are identified by a corresponding hydrograph number enclosed in a "hexagon". The head of the arrow points to the point of discharge from the area. The 2, 5, 10, 25, 50 and 100-year runoff quantities (peak rate and volume) associated with the analysis points are presented in Table 2. Each sub-area is identified by a number enclosed in an ellipse. The "100" series numbers refer to the existing condition.

Hydrographs calculated for the study area and the sub-areas of the watershed are compiled in computer print-out form. The compiled HYMO output is available for inspection at the Albuquerque International Airport, Aviation Department. A portion of the output, however, is summarized in this report in Table 2. Hydrographs have also been plotted for the 2-, 5-, 10-, 25-, 50- and 100-year, 6-hour events. These hydrographs appear in Figures 12 through 17 in Appendix F.

Whereas peak rates of discharge for runoff and runoff volumes are computed for each analysis point, sediment yield is computed for a single point - the delivery point. This point corresponds to analysis point 101.2. It is for this point that the sediment yield is determined on a per event basis. As previously mentioned, the modified Universal Soil Loss Equation and the Meyer-Peter, Muller formula are used to

collectively predict the sediment yield leaving the study area and entering the U.N.M. Golf Course. Table 3 presents the results of the sediment yield analysis. Appendix D contains the Meyer-Peter, Muller computation sheets and the grain size distribution curves for the two samples of the arroyo bed material. The bedload transport for each event is calculated as the average value of the bedload transport for each sample.

HYDROGRAPH	2-YR EVENT		5-YR EVENT		10-YR EVENT		25-YR EVENT		50-YR EVENT		100-YR EVENT	
	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V
104.1	1.0	0.064	2.1	0.334	5.9	0.612	16.1	1.152	34.6	1.938	44.4	2.319
103.1	16.1	1.450	42.0	2.973	64.6	4.176	104.8	6.200	159.6	8.818	185.4	10.009
102.3	14.9	1.461	40.6	3.264	65.6	4.866	113.7	7.527	180.6	11.079	211.3	12.723
102.4	12.4	1.435	32.9	3.256	48.5	4.861	76.6	7.520	162.3	11.067	203.3	12.709
101.2	12.3	1.443	32.7	3.268	48.5	4.883	76.6	7.572	156.4	11.170	197.8	12.838

Q in units of cfs
V in units of acre-feet

TABLE 2: RUNOFF SUMMARY FOR EXISTING CONDITION

EVENT* (yr)	RUNOFF VOLUME** (ac-ft)	RUNOFF PEAK** (cfs)	WASHLOAD** (Tons)	BEDLOAD*** (Tons)	TOTAL SEDIMENT YIELD (Tons)
2	1.443	12.3	1,160.6	1,535.1	2,695.7
5	3.268	32.7	3,171.6	1,937.8	5,109.4
10	4.883	48.5	4,951.6	2,233.6	7,185.2
25	7.572	76.6	8,178.4	2,838.4	11,016.8
50	11.170	156.4	15,164.5	3,456.9	18,621.4
100	12.838	197.8	18,695.8	3,561.0	22,256.8

* 6-Hour Duration
** HYMO
*** Meyer-Peter, Muller

TABLE 3 : SEDIMENT YIELD SUMMARY FOR EXISTING CONDITION

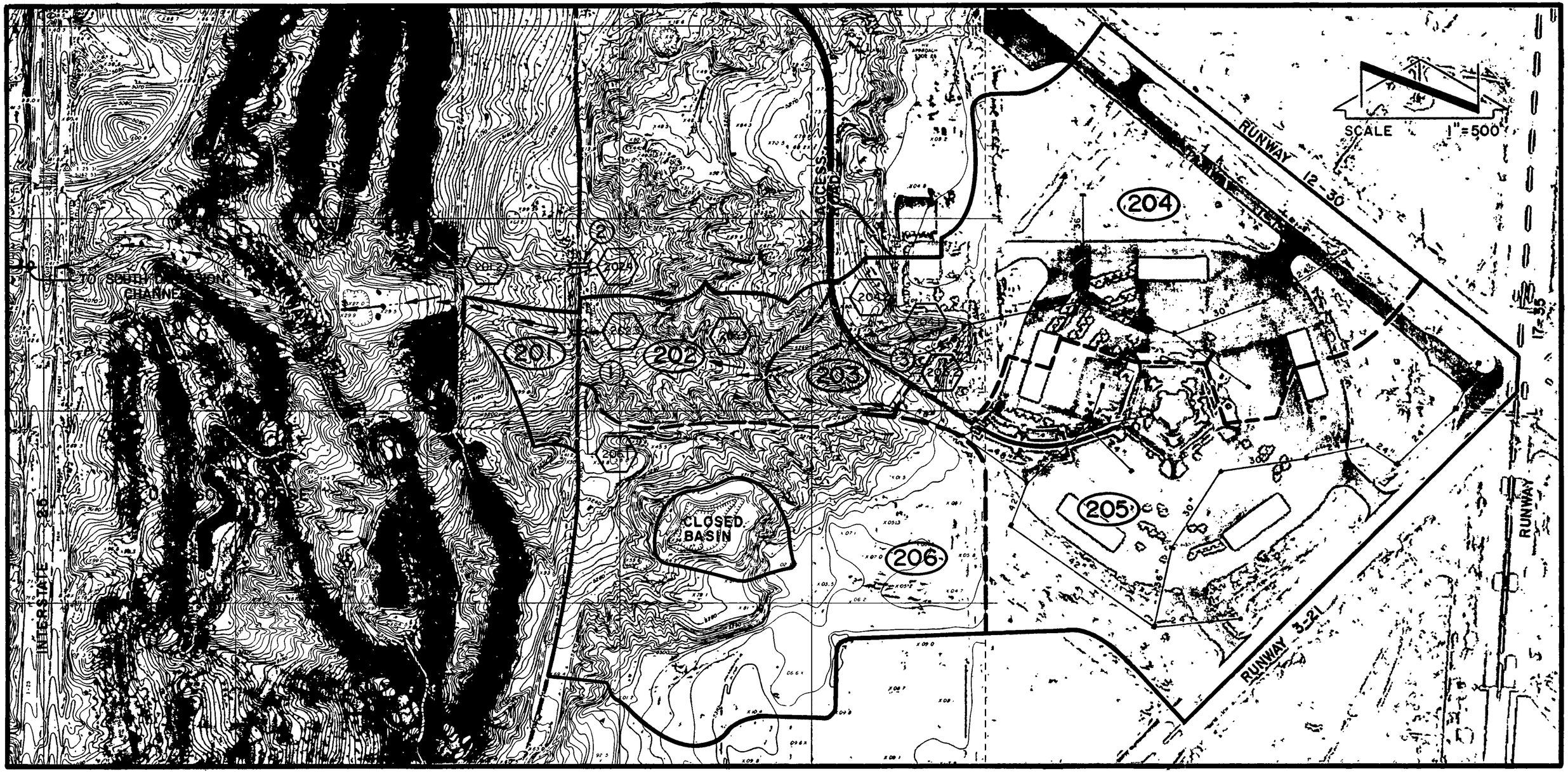
V. DEVELOPED DRAINAGE CONDITION

V. DEVELOPED DRAINAGE CONDITION

The "developed" drainage condition, as described earlier, refers to those conditions which are now present within the study area. This, of course, includes the construction of the "new" General Aviation facility and associated improvements. The developed drainage basin is shown by Figure 7. As in the case with Figure 6, the boundary of the basin is superimposed over the photo-base for the present-day condition. The aerial photograph of the uppermost portion of the basin lacks topographic information. Sufficient topographic information from which the eastern boundary of the watershed and the boundaries of sub-areas 204 and 205 could be delineated was obtained from the record construction drawings of the General Aviation facility (8). These drawings also provided information pertaining to the storm drainage facilities in the General Aviation area.

The developed drainage basin measures 0.398 square miles in size and drains east to west. The majority of the uppermost portion of the study area, sub-areas 204 and 205, are paved and have storm drain collection and conveyance systems. These are delineated by Figure 7 and evaluated for capacity in Appendix A.

The two storm drain systems described above discharge into arroyos which converge at analysis point 203.3. From this point, the arroyo continues to flow west collecting runoff from sub-areas 202 and 206. These flows collect at



LEGEND

- FLOW PATH
- DRAINAGE BASIN BOUNDARY
- DRAINAGE SUB-AREA BOUNDARY
- 000 DRAINAGE SUB-AREA IDENTIFICATION
- 000.0 ANALYSIS POINT & ASSOCIATED HYDROGRAPH
- ROAD CROSSING CULVERT
- 00" STORM DRAIN

KEYED NOTES

1. 60" CMP CULVERT
2. 36" CMP CULVERT
3. 48" RCP CULVERT

FIGURE 7 : DEVELOPED DRAINAGE CONDITION

the existing 60-inch CMP culvert under University Boulevard SE, and are then discharged into sub-area 201. Upon entering sub-area 201, the runoff within the arroyo continues to flow westerly to the east boundary of the U.N.M. Golf Course. Within the Golf Course, the arroyo widens as it crosses Fairway 8. Runoff carried to this point flows through the two breached dikes discussed earlier, and across Fairway 2 to the continuation of the aforementioned arroyo. As stated earlier, the arroyo continues to drain west to ultimately enter the AMAFCA South Diversion Channel.

The drainage boundaries and patterns shown on Figure 7 were determined from the available mapping and supplemented by field observation whenever uncertainties arose. Soil types and ground cover conditions were evaluated using Figure 5, Figure 7 and the SCS Engineering Field Manual (18). Curve numbers for the study area are listed in Table 1. For the purpose of analyzing the developed condition, a weighted curve number was assigned to each area. The computation of the weighted curve numbers is presented in Appendix B.

The hydrologic modeling of the study area in the developed condition was also accomplished using HYMO. HYMO, as discussed earlier, is a digital computer model developed by the U.S.D.A. Agricultural Research Service and adopted by the City of Albuquerque. The HYMO routines employed in this facet of the study permitted the computation of hydrographs; the routing of hydrographs through downstream basins, culverts or ponds; the addition of

hydrographs; and the determination of the washload sediment yield. All of this was performed on a per area basis to enable the study of the watershed at strategic intermediate analysis points. These analysis points and sub-areas are delineated on Figure 7. The analysis points are identified by a corresponding hydrograph number enclosed in a "hexagon". The head of the arrow points to the point of discharge from the area. The 2, 5, 10, 25, 50 and 100-year runoff quantities (peak rate and volume) associated with the analysis points are presented in Table 4. Each sub-area is identified by a number enclosed in an ellipse. The "200" series numbers refer to the developed condition.

Hydrographs calculated for the study area and the sub-areas of the watershed are compiled in computer print-out form. The compiled HYMO output is available for inspection at the Albuquerque International Airport, Aviation Department. A portion of the output, however, is summarized in this report in Table 4. Hydrographs have also been plotted for the 2-, 5-10-, 25-, 50- and 100-year, 6-hour events. These hydrographs appear in Figures 12 through 17 in Appendix F.

Whereas peak rates of discharge for runoff and runoff volumes are computed for each analysis point, sediment yield is computed for a single point - the delivery point. This point corresponds to analysis point 201.2. It is for this point that the sediment yield is determined on a per event basis. As previously mentioned, the modified Universal Soil

Loss Equation and the Meyer-Peter, Muller formula are used to collectively predict the sediment yield leaving the study area and entering the U.N.M. Golf Course. Table 5 presents the results of the sediment yield analysis for each event. Appendix D contains the Meyer-Peter, Muller computation sheets and the grain size distribution curves for the two samples of the arroyo bed material. The bedload transport for each event is calculated as the average value of the bedload transport for each sample.

HYDROGRAPH	2-YR EVENT		5-YR EVENT		10-YR EVENT		25-YR EVENT		50-YR EVENT		100-YR EVENT	
	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V
206.1	1.0	0.064	2.1	0.334	5.9	0.612	16.1	1.152	34.6	1.938	44.4	2.319
205.2	40.5	1.873	77.5	3.282	104.3	4.277	134.8	5.868	134.8	7.831	134.9	8.701
204.1	14.0	1.134	32.1	2.168	47.0	2.962	72.7	4.275	107.0	5.947	122.9	6.699
204.2	11.6	0.948	26.6	2.109	38.8	2.908	55.2	4.224	75.4	5.897	84.9	6.649
203.3	39.3	2.837	81.3	5.419	116.0	7.229	177.1	10.168	210.8	13.863	220.4	15.513
202.3	36.1	2.849	77.3	5.704	116.7	7.909	184.2	11.485	225.2	16.114	237.8	18.220
202.4	29.9	2.836	56.6	5.699	78.3	7.900	159.8	11.469	225.1	16.089	237.7	18.192
201.2	29.8	2.844	56.5	5.703	78.3	7.916	152.0	11.516	225.5	16.187	238.3	18.316

Q in units of cfs
V in units of acre-feet

TABLE 4: RUNOFF SUMMARY FOR DEVELOPED CONDITION

EVENT* (yr)	RUNOFF VOLUME ** (ac-ft)	RUNOFF PEAK** (cfs)	WASHLOAD** (Tons)	BED*** LOAD (Tons)	TOTAL SEDIMENT YIELD (Tons)
2	2.844	29.8	1,314.1	1,684.8	2,998.9
5	5.703	56.5	2,776.7	2,408.5	5,185.2
10	7.916	78.3	4,003.3	2,947.6	6,950.9
25	11.516	152.0	7,161.5	3,246.5	10,408.0
50	16.187	225.5	10,804.9	4,982.5	15,787.4
100	18.316	238.3	11,943.6	5,753.2	17,696.8

* 6-Hour Duration
** HYMO
*** Meyer-Peter, Muller

TABLE 5: SEDIMENT YIELD SUMMARY FOR DEVELOPED CONDITION

VI. CONCLUSIONS

VI. CONCLUSIONS

Several conclusions can be drawn from this study of the General Aviation/Golf Course drainage basin. These conclusions consider the following topics:

1. Problem recognition/identification
2. Runoff quantity
3. Runoff frequency
4. Sediment yield
5. Watershed drainage pattern and arroyo geometry
6. Future development

A more detailed discussion of these topics appears below.

1. Problem Recognition/Identification. The main problem associated with the study area, as identified by the U.N.M. Golf Course (4, 5), is the repeated deterioration of Fairway 2 following periods of heavy runoff. Runoff passing over Fairway 2 has begun to erode the rough along the west side of the fairway and continually deposits sediment (i.e. sand) on the fairway itself. This, as can be realized, presents a maintenance problem to the Golf Course.

Unfortunately, this problem has been largely created by the Golf Course in that both Fairways 2 and 8 were constructed across an existing arroyo. However, the Golf Course was not entirely unaware of this situation because the first dike was built in 1966 when the course opened (7). According to McGuire (7), this first dike was constructed to keep runoff from crossing Fairway 2. In doing this, the Golf Course not

only recognized the presence of a drainage problem, but created a more severe situation for Fairway 8. All runoff periodically retained by the dike, was then caused to drop its sediment load on Fairway 8. In essence, the Golf Course sacrificed Fairway 8 to hopefully protect Fairway 2.

The series of dikes constructed between Fairways 2 and 8 were not engineered, but merely constructed in hopes that they would be adequate. On three separate occasions, the dikes proved to be inadequate for rainfall events less than or equal to the 25-year, 6-hour storm in the developed condition. This statement is based upon the review of National Weather Service data (11, 12) and the rainfall data tabulated in Appendix G.

In the cases where CMP culvert outlets were incorporated into the construction of the dikes, failure occurred at those installations. Seepage along the outside of the pipe mobilized the "piping" of the embankment material thereby leading to the displacement of the pipe from the earthen structure. This, of course, lead to the breach of the dike and the uncontrolled discharge of runoff through the golf course.

In addition to the absence of anti-seep collars and/or other anchoring devices for the outlet culverts, the dikes were apparently not provided with spillway sections to adequately convey any overflow runoff in a controlled fashion. This further illustrates the lack of engineering design in the construction of the dikes.

Review of Figure 7, coupled with field observation and interviews (4), reveals that the dikes may have had sufficient storage to contain the anticipated runoff from a 100-year, 6-hour storm in the developed condition. This finding, however, does not explain the topping of the lower dike during the last failure, as was recalled by Kline (4). The storm which caused the overtopping was possibly storm No. 67, shown by Table 10, which occurred October 2, 1981. Storm No. 67 was really two storms which collectively approximate a 2-year, 6-hour rainfall. In view of this, the dikes should have been able to contain the flood waters. According to Kline (4), however, the lower dike could not hold the runoff. This indicates that the volume of storage of the dikes was significantly reduced since the initial construction in 1966. This, in turn, reveals the lack of a properly administered maintenance program for the "ponding" area. Sediment should have been removed from behind the dikes on a periodic basis in order to maintain full storage capacity. Conversations with Kline (4, 5) did not reveal that any such program existed, only one of rebuilding in the event of a failure.

Lastly, it should be stressed that had the arroyo been integrated into the layout and design of the golf course, the current problem would not exist. Unfortunately, this was not the case, and therefore, the presence of the arroyo must be acknowledged and its flows handled by sound engineering practices.

2. Runoff Quantity. Runoff "quantity" refers to both the peak rate of discharge and the total runoff volume. These values, as described earlier, have been determined for the analysis points shown on Figures 6 and 7. Table 6 compares existing and developed condition results at identical analysis points within the watersheds. Of particular interest in this study are the changes observed at the outfall from the study area. This point lies along the east boundary of the U.N.M. Golf Course and is identified by hydrographs 101.2 and 201.2.

The improvements in the General Aviation area have increased the amount of impervious area, as well as the total area, in the watershed, thereby increasing both the peak discharge and the total volume of runoff. The increase in the peak discharge is most dramatic for the 25-year, 6-hour event where it doubles. Runoff volume is dramatically increased for all events. Increases in runoff volume range from 97 percent for the 2-year, 6-hour storm to 43 percent for the 100-year, 6-hour storm. The results are based upon the modeling of the existing and developed conditions using HYMO.

It was discovered during the analysis of the developed watershed that the storm drain system in sub-area 205 is inadequate to convey the peak discharge from the 25-year, 6-hour storm, as well as less frequent storms. Because of this, runoff mementarily ponds in the aircraft tie-down areas. These areas were designed and constructed as "dishes" in the pavement with catch basins strategically located in the low-points. The effect of this occurrence is to attenuate the

EVENT		HYDROGRAPH									
		103.1	203.3	104.1	206.1	102.3	202.3	102.4	202.4	101.2	201.2
TWO YEAR	Q	16.1	39.3	1.0	1.0	14.9	36.1	12.4	29.9	12.3	29.8
	V	1.450	2.837	0.064	0.064	1.461	2.849	1.435	2.836	1.443	2.844
	T	0.34	0.30	0.42	0.42	0.42	0.38	0.56	0.52	0.60	0.54
FIVE YEAR	Q	42.0	81.3	2.1	2.1	40.6	77.3	32.9	56.6	32.7	56.5
	V	2.973	5.419	0.334	0.334	3.264	5.704	3.256	5.699	3.268	5.703
	T	0.32	0.28	0.34	0.34	0.38	0.34	0.52	0.54	0.54	0.54
TEN YEAR	Q	64.6	116.0	5.9	5.9	65.6	116.7	48.5	78.3	48.5	78.3
	V	4.176	7.229	0.612	0.612	4.886	7.909	4.861	7.900	4.883	7.916
	T	0.32	0.28	0.26	0.26	0.36	0.32	0.56	0.52	0.54	0.54
TWENTY FIVE YEAR	Q	104.8	177.1	16.1	16.1	113.7	184.2	76.6	159.8	76.6	152.0
	V	6.200	10.168	1.152	1.152	7.527	11.485	7.520	11.469	7.572	11.516
	T	0.32	0.32	0.22	0.22	0.34	0.34	0.50	0.40	0.52	0.42
FIFTY YEAR	Q	159.6	210.8	34.6	34.6	180.6	225.2	162.3	225.1	156.4	225.5
	V	8.818	13.863	1.938	1.938	11.079	16.114	11.067	16.089	11.170	16.187
	T	0.30	0.48	0.20	0.20	0.32	0.46	0.40	0.48	0.42	0.48
ONE HUNDRED YEAR	Q	185.4	220.4	44.4	44.4	211.3	237.8	203.3	237.7	197.8	238.3
	V	10.009	15.513	2.319	2.319	12.723	18.220	12.709	18.192	12.838	18.316
	T	0.30	0.48	0.20	0.20	0.32	0.48	0.36	0.48	0.38	0.50

TABLE 6: COMPARISON OF IDENTICAL ANALYSIS POINTS FOR EXISTING AND DEVELOPED CONDITIONS

peak discharge from sub-area 205 while lengthening the duration of the runoff. Consequently, sub-area 205 has been modeled to reflect the restrictiveness of the storm drain system. It should be noted that through modeling it was determined that runoff will pond to a maximum depth of about 6 inches during the 100-year, 6-hour storm. This should not pose a threat to aircraft parked in the lower portions of the paved areas.

The storm drain system in sub-area 204 is slightly restrictive for only the 100-year, 6-hour event. The marginal nature of this restriction was deemed insignificant and therefore, sub-area 204 was not modeled with a restrictive outlet.

The full effect of the increases in runoff due to the development of the General Aviation area is not known. It is speculated, however, that the increase in runoff peak discharge for all events along with the lengthening of the base of the hydrograph for the 50-year and 100-year events has increased the bedload transport observed at the Golf Course. This topic is discussed later in this section.

3. Runoff Frequency. The development of the General Aviation area not only increased the quantity of runoff within the study area, but also the frequency of runoff. The increase in runoff frequency is linked directly with the increase in impervious area within the study area. This occurrence has been analyzed based upon the extrapolation of National Weather Service rainfall data by Easterling (22). The determination of composite

curve numbers and corresponding thresholds of runoff appear in Appendices B and E respectively. The determination of these values, and knowledge of the work by Easterling (22), permitted the development of Figure 8. This figure illustrates, along with Table 7, that the increase in impervious area has at least doubled the frequency of runoff leaving sub-areas 204 and 205. This increase is not expected to be nearly so dramatic for the overall watershed.

An increase in the frequency of runoff for the study area means that more runoff events per year can be expected to reach the Golf Course property. This, of course, is dependent upon the antecedent moisture conditions and the distribution of rainfall for a particular event. Due to the coarseness of the arroyo sand and the very low percentage of fines present in Samples A and B, it is anticipated that the arroyo has a rather high infiltration capacity. The observation of rainfall/runoff events during this study effort revealed that the arroyo bed readily absorbed the majority of the runoff it received from the General Aviation area. The rainfall during the first observation, July 19, 1982, only measured 0.09 inches (23). Needless to say, no runoff was generated by the arroyo bed itself or the surrounding range land. Minor amounts of runoff were generated by the unpaved roadways which surround the General Aviation area. Runoff was observed being discharged from both the north and south storm drain systems. Runoff from

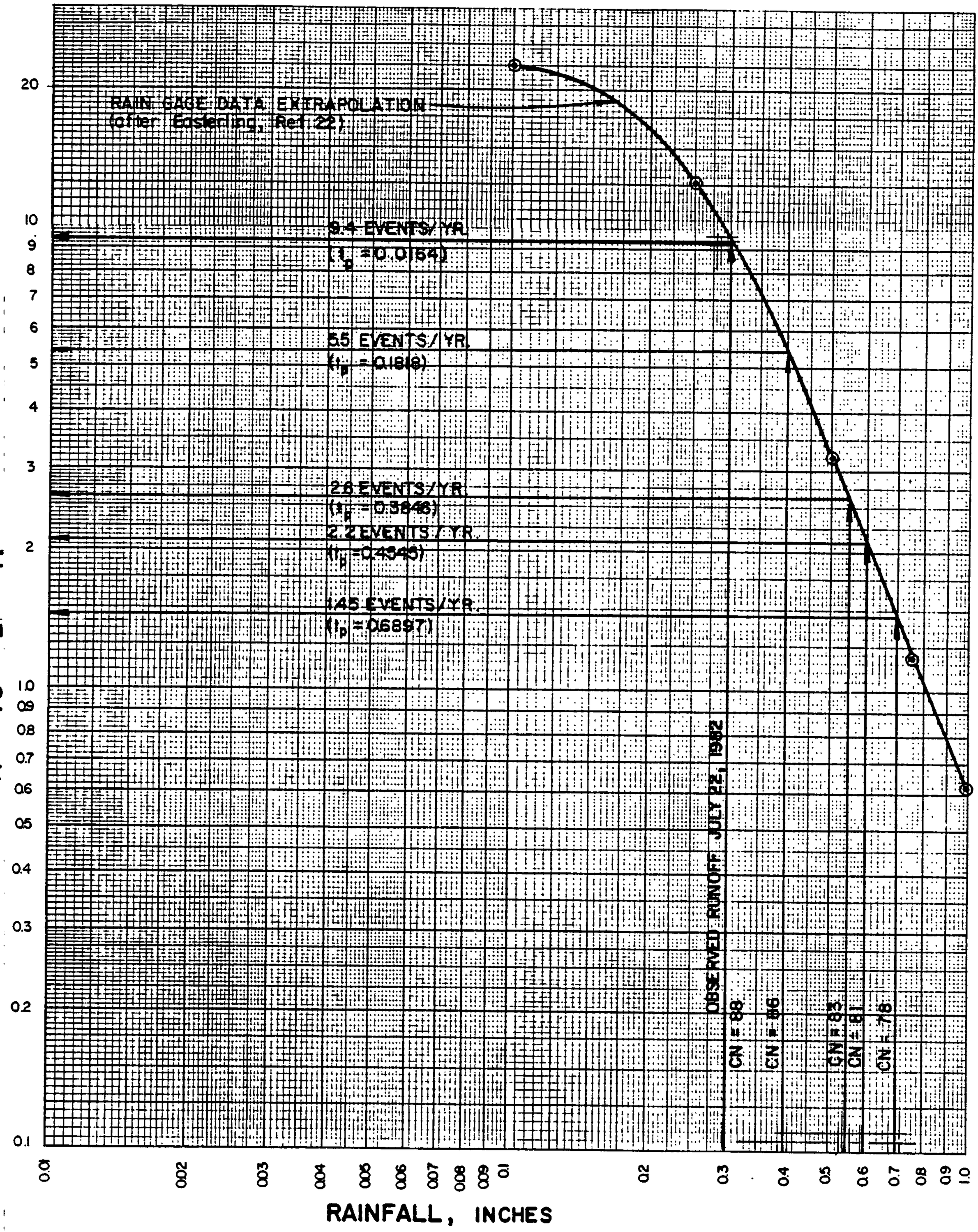


FIGURE 8. FREQUENCY OF RAINFALL AND RUNOFF EVENTS

AREA	CONDITION	COMPOSITE CN	RUNOFF FREQUENCY, EVENTS/YEAR	RECURRENCE INTERVAL
203, 204 & 205 = 103	EXISTING	83	2.6	0.3846
	DEVELOPED	86	5.5	0.1818
OVERALL WATERSHED	EXISTING	78	1.45	0.6897
	DEVELOPED	81	2.2	0.4545
	OBSERVED	88	9.4	0.1064

TABLE 7: COMPARISON OF ESTIMATED RUNOFF FREQUENCY

the north culvert barely reached the 48-inch RCP under the Access Road. Runoff from the south culvert trickled to analysis point 203.3 before vanishing into the arroyo bed.

The second storm observed during this study effort measured 0.30 inches (24). This event produced enough runoff in sub-area 205 to ultimately reach Fairway 8. Upon reaching Fairway 8, the runoff rapidly infiltrated into the sand deposits. The runoff reaching Fairway 8 was between 1 and 2 inches in depth and was entirely contributed by sub-area 205. The north culvert, however, did not appear to contribute any substantial surface flow to the arroyo; runoff from sub-area 204 did not reach analysis point 203.3.

The observation of the July 22 storm revealed that runoff generated by a 0.30-inch rainfall does reach the golf course. Selecting the 0.30-inch rainfall as the runoff threshold raised the frequency of runoff due to development in the study area to approximately three times the existing frequency.

In view of the fact that the runoff which reached the golf course was generated by sub-area 205 is not surprising. Assigning a threshold to runoff equal to 0.30 inches for sub-area 205 produces an approximate curve number of 88. The composite curve number for sub-area 205 alone is 89. Because of this, the July 22, 1982, field observation does not present a contradiction to the relationship established by Figure 8. It must be noted, however, that the runoff observed on that occasion was not damaging in nature. Furthermore, it must be

realized that although the frequency of runoff has been tripled by the development of one-third of the total study area, it can not be necessarily deduced that the frequency of potentially damaging runoff has also been tripled.

4. Sediment Yield. The analysis of the sediment yield in both existing and developed conditions reveals no significant changes for the more frequent events, nor for the estimated annual yield. Table 8 compares calculated values of sediment yield for both the existing and developed conditions. This table also separates sediment yield by washload and bedload.

Figure 9 illustrates the relationships between washload, bedload and total sediment yield for each event for both existing and developed conditions. This figure demonstrates that for the more frequent events, total sediment yield for the watershed has not been significantly influenced by development. Whereas bedload has been increased by increased peak discharges, washload has been decreased by decreasing the overall area of watershed capable of contributing sediment. These changes appear to balance for the more frequent events. Total sediment yield for larger storms, however, is expected to be less for the developed condition than before development largely due to a significant decrease in the estimated washload.

Annual sediment yield calculations reveal no significant change due to development. These calculations, which appear in Appendix E, estimate the average annual sediment yield at 20,400 tons/year for both the existing and developed conditions.

EVENT	WASHLOAD*		BEDLOAD*		SEDIMENT YIELD*	
	EXIST.	DEV.	EXIST.	DEV.	EXIST.	DEV.
2-YEAR	1,160.6	1,314.1	1,535.1	1,684.8	2,695.7	2,998.9
5-YEAR	3,171.6	2,776.7	1,937.8	2,408.5	5,109.4	5,185.2
10-YEAR	4,951.6	4,003.3	2,233.6	2,947.6	7,185.2	6,950.9
25-YEAR	8,178.4	7,161.5	2,838.4	3,246.5	11,016.8	10,408.0
50-YEAR	15,164.5	10,804.9	3,456.9	4,982.5	18,621.4	15,787.4
100-YEAR	18,695.8	11,943.6	3,561.0	5,753.2	22,256.8	17,696.8

*TONS

TABLE 8: COMPARATIVE SUMMARY OF SEDIMENT YIELD CALCULATIONS

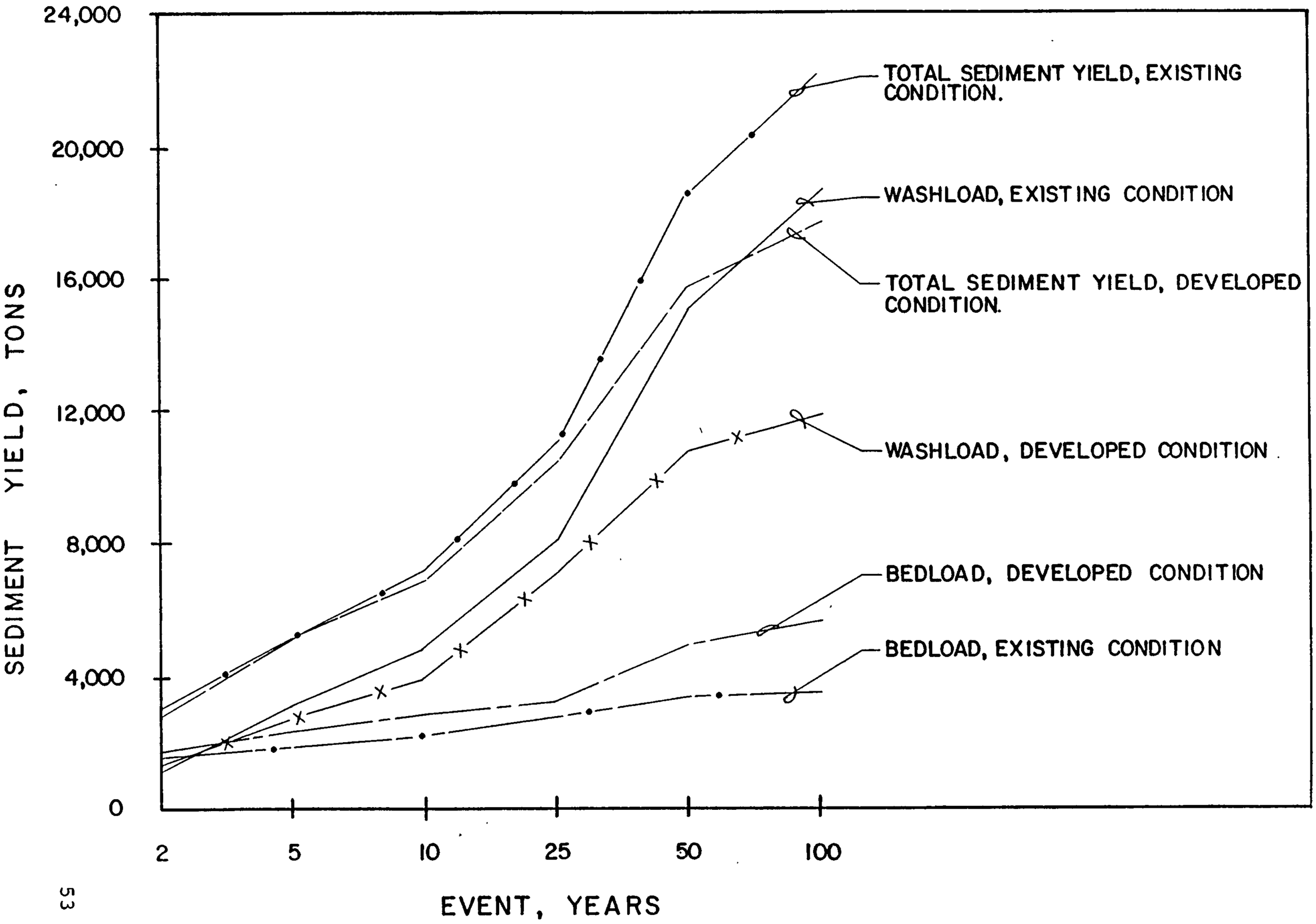


FIGURE 9: SEDIMENT YIELD COMPARISON FOR EXISTING AND DEVELOPED CONDITIONS

Assuming a specific gravity of 2.65 for the arroyo sand allows the conversion of the 20,400 tons/year to approximately 9,150 CY/year. This relates closely to the average 10,230 CY/year observed over the last two years at the golf course. It must be realized, however, that the calculation of the observed sediment yield is based upon an eyewitness account (4) coupled with the review of available aerial photography.

5. Watershed Drainage Pattern and Arroyo Geometry. Development has not altered the basic drainage pattern of the study area. The development of the General Aviation area did, however, increase the size of the watershed by approximately 0.05 square miles. Despite this minor change, runoff still flows to the arroyo which runs east to west through the center of the study area. Once in the arroyo, runoff flows west to the east property line of the U.N.M. Golf Course. This pattern was described earlier in more detail in Section IV and V.

Figure 10 is an aerial photograph of the General Aviation/ Golf Course drainage basin. This photograph, which was taken about 1972, illustrates that the main arroyo which is present today was also present in 1972 prior to the construction of the General Aviation facilities. In fact, the arroyo width scales approximately the same in the 1972 photograph as it does in Figure 7, a 1980 photograph. This indicates that the upstream development in the study area has not significantly altered the arroyo geometry. Minor alteration has occurred,

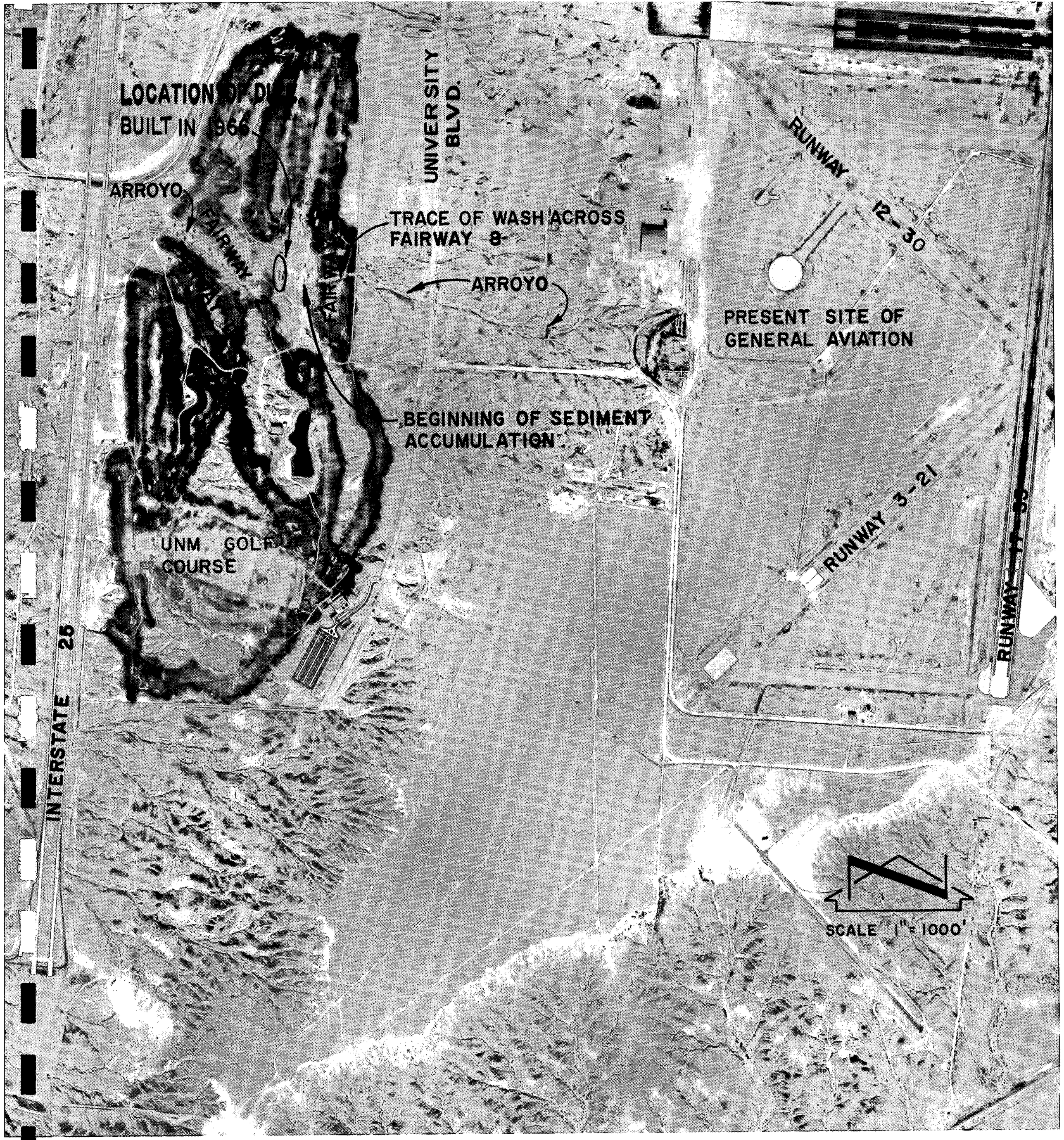


FIGURE 10: Aerial photograph of study area (circa 1972 ; photograph courtesy of Aviation Department, Albuquerque International Airport.).

however, at the outlet of the storm drain system in sub-area 205. According to Gill (25), initial scouring occurred at this point of discharge following the completion of construction. This scouring appears to have been the result of a significant drop at the pipe outlet coupled with insufficient energy dissipation, lack of an adequate erosion control apron, and an increase in runoff frequency from the General Aviation area. This problem appears to have been remedied by the Albuquerque International Airport by the construction of a massive erosion control/energy dissipation structure composed of both dumped and grouted rip-rap at the outlet of the 48-inch RCP. As a result of this construction, it appears that the scouring along this portion of the arroyo has been stabilized.

6. Future Development. The treatment of storm water generated by future development should be a far more important concern than the discussion of the present-day conditions. The development of the land which lies between the General Aviation area and the U.N.M. Golf Course will almost certainly increase the quantity and frequency of runoff observed at the golf course. Assuming that the contributing area remains approximately 0.4 square miles and that the area develops 90 percent impervious, it is not unreasonable to anticipate a three-fold increase in runoff peak discharge (100-year, 6-hour), a two-fold increase in runoff volume (100-year, 6-hour), and a three-fold increase in runoff frequency. These comparisons are made with respect to present-day conditions. Increases in the order

of those which are projected above will have a severe impact on the U.N.M. Golf Course. Whereas it is difficult to assess the full impact of the General Aviation area improvements on the golf course, there will be a marked difference observed under a fully developed watershed.

VII. RECOMMENDATIONS

VII. RECOMMENDATIONS

Several alternates have been formulated as a result of this study of the General Aviation/Golf Course drainage basin. The alternates presented herein address varying schemes by which storm water generated within the study area can be handled. These alternates are summarized as follows:

1. "Do Nothing"
2. Repair, Stabilize and Maintain Fairway 2
3. Construct Concrete Channel Across Fairway 2
4. Construct a Detention Dam on Fairway 8

Each alternate is a separate proposal intended for either selection or rejection. In addition, each alternate is discussed in more detail below. Recommendations are made to assist reviewers in the selection of an alternate.

1. "Do Nothing". The "Do Nothing" solution is one in which the runoff generated within the study area is permitted to flow uninhibited through the golf course. This will probably result in the loss of a portion of Fairway 2 as the arroyo attempts to re-establish its course. Consequently, the selection of this alternate will ultimately sacrifice another portion of golf course fairway.

This alternate is not strongly recommended because it will almost certainly create some hardship on those who use the golf course. Permitting the arroyo to re-establish itself will not only make Hole #2 more difficult to play, but also more difficult to reach via either golf cart or on foot.

2. Repair, Stabilize and Maintain Fairway 2. This alternate requires the replacement of the "rough" along the west side of Fairway 2 where it has begun to erode, and then placing rip-rap from the west edge of the rough to the arroyo bed. The placement of the rip-rap will prevent the erosion of the rough caused by runoff flowing across Fairway 2 and entering the arroyo. Additional rip-rap should be placed along the south bank of the arroyo where it borders Fairway 17.

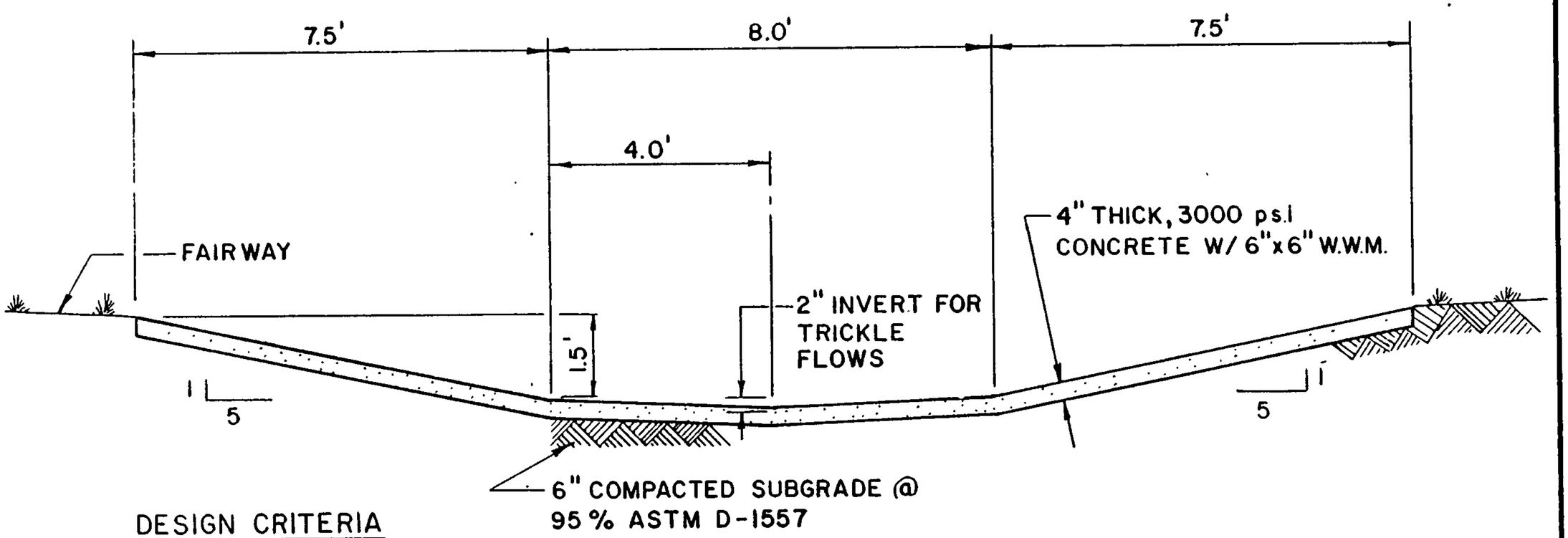
In addition to the repairs and stabilization mentioned above, periodic maintenance will be required following storms which transport sediment. This maintenance will involve the clean-up of sand from Fairway 2. In lieu of this, the Golf Course could construct a sedimentation pond on the west edge of Fairway 8. This pond would intercept and trap sediment before it reaches Fairway 2. The pond should have the capacity to contain at least the estimated annual sediment yield approximately 10,000 cubic yards. It must be kept in mind, however, that this pond must be re-excavated on a periodic basis to maintain its capacity.

This alternate is strongly recommended as an interim solution to the present problem of golf course degradation. The The alternate can be expeditiously put into effect at a rather low monetary cost. Also, this alternate provides for the continuance of play on Fairway 2.

3. Construct Concrete Channel Across Fairway 2. This alternate proposes the construction of a channel capable of conveying the 100-year, 6-hour peak discharge for the developed condition. This channel should begin in Fairway 8, and flow across Fairway 2 to discharge into the arroyo which borders Fairway 17. Of course, rip-rap will be required at both the inlet and outlet of this channel.

As shown by Figure 11, the side slopes of the proposed channel are limited to 5:1 to accommodate golf carts and golfers on foot. Also included on Figure 11 are the design criteria upon which the suggested channel section is based. Concrete is recommended for the channel lining to facilitate and minimize maintenance. An integrally mixed colored concrete could be used to soften the appearance of the channel, if so desired.

The sizing of the channel, as shown by Figure 11, does not account for any future development within the study area. Because of this, it may be desirable to re-evaluate the hydrology of the contributing area once the future of the watershed is established. At that time, the full contribution of runoff from the watershed can be assessed and a complete channel design prepared. This channel should extend from Fairway 2 to the existing box culverts under Interstate 25. From this point, as in the case for the developed condition, runoff will flow across presently undeveloped land and into the A.M.A.F.C.A. South Diversion Channel at a point due west of Interstate 25.



DESIGN CRITERIA

DESIGN Q = 240 cfs
n = 0.015
s = 0.02
$Q_{MAX.}$ = 325 cfs

FIGURE 11 : TYPICAL CHANNEL SECTION ACROSS FAIRWAY 2

SCALE 1" = 3'

At such time as the study area begins to fully develop, the latter of the channels should be constructed in order to safely convey storm water to a suitable outfall, the South Diversion Channel. It appears likely that the construction of the channel through the golf course could be accomplished as a joint participation project with all who benefit sharing in the construction cost. With this in mind, the Golf Course may wish to "repair, stabilize and maintain Fairway 2" as specified in the preceding alternate, and wait until future development necessitates channel construction to provide a safe, suitable outfall. This will relieve the Golf Course of immediate financial burden while avoiding the construction of a facility which will ultimately become obsolete.

4. Construct a Detention Dam on Fairway 8. The construction of a detention dam on Fairway 8 would require extensive earthwork in order to provide a reservoir capable of holding the 100-year, 6-hour storm runoff volume, 18.316 acre-feet. The dam would have to be built with an outlet capable of discharging the runoff within 96 hours, and have a spillway capable of passing the "Probable Maximum Flood", both in accordance with the New Mexico State Engineer's Office. Such a structure would be a rather large expenditure, would be a constant maintenance problem with respect to sedimentation, would be an inconvenience to golfers, would be a potential safety hazard if not properly maintained, and would most likely be obsolete in a relatively short time when the study area begins to

further develop. For the reasons stated above, this alternate is not recommended.

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VIII. REFERENCES

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APPENDIX A: TIME OF CONCENTRATION CALCULATIONS

TIME OF CONCENTRATION CALCULATIONS

EXISTING CONDITION

Area 101:

L = 1100', Δ elev = 50'
By Kirpich Equation,

T_c = 0.094 hr

T_p = 2/3 T_c = 0.063 hr

K = 1/2 T_p = 0.031 hr

Area 102:

L = 1250', Δ elev = 58'

T_c = 0.103 hr

T_p = 0.069 hr

K = 0.034 hr

Area 103:

L = 4400', Δ elev = 66'

T_c = 0.418 hr

T_p = 0.278 hr

K = 0.139 hr

Area 104:

L = 3000', Δ elev = 82'

T_c = 0.247 hr

T_p = 0.165 hr

K = 0.082 hr

DEVELOPED CONDITION

Area 201:

SAME AS AREA 101

Area 202:

SAME AS AREA 102

Area 203:

L = 800', Δ elev = 48'

T_c = 0.066 hr

T_p = 0.044 hr

K = 0.022 hr

Area 204:

1. Overland Flow - Upstream

L = 1400', Δ elev = 5.2'

T_c = 0.297 hr

2. Pipe Flow

24" RCP

L_{24} = 512'

Δ elev = 3.76-1.58 = 2.18'

s = 0.004

r = 1'

C = 150

Max. flow occurs @ D = 1.8811 r

=> Q_{cap} = 21.8 cfs

2/3 Q_{cap} = 14.5 cfs

=> D = 1.25'

=> A = 2.07 sf

$$v = \frac{Q}{A} = \frac{14.5}{2.07} = 7 \text{ fps}$$

$$T_t = \frac{L}{v} = \frac{512}{7} = 73 \text{ sec}$$

30" RCP

L_{30} = 482+505 = 987'

Δ elev = 101.58-99.73 = 1.85'

s = 0.002

r = 1.25'

C = 150

Max. flow occurs @ D = 1.8811 r

=> Q_{cap} = 26.9 cfs

2/3 Q_{cap} = 18.0 cfs

=> D = 1.57'

=> A = 3.23 sf

$$v = \frac{Q}{A} = \frac{18}{3.23} = 5.6 \text{ fps}$$

$$T_t = \frac{L}{v} = \frac{987}{5.6} = 177 \text{ sec}$$

42" RCP

L_{42} = 415+200 = 615'

Δ elev = 99.73-96.37 = 3.36'

s = 0.0055

r = 1.75'

C = 1.50

Max. flow occurs @ D = 1.8811 r

=> Q_{cap} = 112.7 cfs

2/3 Q_{cap} = 75.1 cfs

=> D = 2.19'

=> A = 6.34 sf

$$v = \frac{Q}{A} = \frac{75.1}{6.34} = 11.8 \text{ fps}$$

$$T_t = \frac{L}{v} = \frac{615}{11.8} = 52 \text{ sec}$$

$$\Sigma T_t = 73+177+52 = 302 \text{ sec}$$
$$= \frac{302}{3600} = 0.084 \text{ hr}$$

Let $T_t = T_{c,\ pipe}$

3. Overland Flow - Downstream

L = 600', Δ elev = 19.7'

T_c = 0.067 hr

4. Total

$$\Sigma T_c = 0.297+0.084+0.067$$
$$= 0.448 \text{ hr}$$

T_p = 0.298 hr

K = 0.149 hr

Area 205:

1. Overland Flow - Upstream

L = 700', Δ elev = 3.5'

T_c = 0.155 hr

2. Pipe Flow

24" RCP

L_{24} = 170+318+460+168 = 1116'

Δ elev = 4.5'

s = 0.004

r = 1'

C = 150

Max. flow occurs @ D = 1.8811r

=> Q_{cap} = 21.8 cfs

$2/3\ Q_{cap} = 14.5$ cfs

$\Rightarrow D = 1.25'$

$\Rightarrow A = 2.07$ sf

$v = \frac{Q}{A} = 7$ fps

$T_t = \frac{L}{v} = \frac{1116}{7} = 160$ sec

30" RCP

$L_{30} = 439+470 = 909'$

$s = 0.004$

$r = 1.25'$

$C = 150$

$D = 1.8811(1.25) = 2.35'$

$\Rightarrow Q_{cap} = 39.2$ cfs

$2/3\ Q_{cap} = 26.1$ cfs

$\Rightarrow D = 1.57'$

$\Rightarrow A = 3.23$ sf

$v = \frac{Q}{A} = \frac{26.1}{3.23} = 8.1$ fps

$T_t = \frac{L}{v} = \frac{909}{8.1} = 113$ sec

36" RCP

$L_{36} = 416'$

Δ elev $= 1.66'$

$s = 0.004$

$r = 1.5$

$C = 150$

$D = 1.8811(1.50) = 2.82'$

$\Rightarrow Q_{cap} = 63.2$ cfs

$2/3\ Q_{cap} = 42.2$ cfs

$\Rightarrow D = 1.88'$

$\Rightarrow A = 4.66$ sf

$v = \frac{Q}{A} = \frac{42.2}{4.66} = 9.1$ fps

$T_t = \frac{L}{V} = \frac{416}{9.1} = 46$ sec

42" RCP

$L_{42} = 880+232 = 1112'$

$s = 0.004$

$r = 1.75'$

$C = 150$

$D = 1.8811(1.75) = 3.29'$

$\Rightarrow Q_{cap} = 94.9$ cfs

2/3 Q_{cap} = 63.2 cfs

=> D = 2.19'

=> A = 6.34 sf

$$v = \frac{Q}{A} = \frac{63.2}{6.34} = 10 \text{ fps}$$

$$T_t = \frac{L}{v} = \frac{1112}{10} = 112 \text{ sec}$$

48" RCP

L_{48} = 498+288 = 786'

s = 0.004

r = 2'

C = 150

D = 1.8811(2) = 3.76'

=> Q_{cap} = 134.8 cfs

2/3 Q_{cap} = 89.9 cfs

=> D = 2.50'

=> A = 8.28 sf

$$v = \frac{Q}{A} = \frac{89.9}{8.28} = 10.8 \text{ fps}$$

$$T_t = \frac{L}{v} = \frac{786}{10.8} = 72 \text{ sec}$$

ΣT_t= 160+113+46+112+72 = 503 sec = $\frac{503}{3600}$ = 0.140 hr = T_c

3. Total

ΣT_c = 0.155+0.140 = 0.295 hr

T_p = 0.196 hr

K = 0.098 hr

Area 206:

SAME AS AREA 104

APPENDIX B: CURVE NUMBER CALCULATIONS

SUB-AREAS

EXISTING CONDITION

Area 101:

All BKD and Cu soils, both of which are "A" soils ∴ CN = 68

Area 102:

Impervious area = 0.001 SM
BKD & Cu soils (A) = 0.022 SM
Total = 0.023 SM

$$\text{Weighted Curve Number} = \frac{0.001}{0.023}(95)+\frac{0.022}{0.023}(68) = 69$$

Area 103:

Impervious area = 0.071 SM
BKD & Cu soils (A) = 0.032 SM
WaB soil (B) = 0.105 SM
Total = 0.208 SM

$$\text{Weighted Curve Number} = \frac{0.071}{0.208}(95)+\frac{0.105}{0.208}(79)+\frac{0.032}{0.208}(68) = 83$$

Area 104:

Impervious area = 0.001 SM
BKD soil (A soil) = 0.069 SM
WaB soil (B soil) = 0.032 SM
Total = 0.102 SM

$$\text{Weighted Curve Number} = \frac{0.001}{0.102}(95)+\frac{0.032}{0.102}(79)+\frac{0.069}{0.102}(68) = 72$$

DEVELOPED CONDITION

Area 201:

All BKD and Cu soils, both of which are "A" soils ∴ CN = 68

Area 202:

All BKD and Cu soils, both of which are "A" soils ∴ CN = 68

Area 203:

All BKD soil (A soil) ∴ CN = 68

Area 204:

Impervious area	= 0.054 SM
BKD & Cu (A soil)	= 0.013 SM
WaB (B soil)	= 0.057 SM
Total	= 0.124 SM

$$\text{Weighted Curve Number} = \frac{0.054}{0.124}(95)+\frac{0.057}{0.124}(79)+\frac{0.013}{0.124}(68) = 85$$

Area 205:

Impervious area	= 0.078 SM
WaB (B soil)	= 0.050 SM
BKD (A soil)	=< 0.001 SM
Total	= 0.128 SM

$$\text{Weighted Curve Number} = \frac{0.078}{0.128}(95)+\frac{0.050}{0.128}(79) = 89$$

Area 206:

Impervious area	= 0.001 SM
BKD soil (A soil)	= 0.069 SM
WaB soil (B soil)	= 0.032 SM
Total	0.102 SM

$$\text{Weighted Curve Number} = \frac{0.001}{0.102}(95)+\frac{0.032}{0.102}(79)+\frac{0.069}{0.102}(68) = 72$$

COMPOSITE AREAS

EXISTING CONDITION

Area 103: CN = 83

Equivalent to Areas 203, 204 and 205

Watershed:

$$\frac{0.009}{0.342}(68)+\frac{0.023}{0.342}(69)+\frac{0.208}{0.342}(83)+\frac{0.102}{0.342}(72) = 78$$

DEVELOPED CONDITION

Area 203, 204 and 205:

$$\frac{0.010}{0.262}(68)+\frac{0.124}{0.262}(85)+\frac{0.128}{0.262}(89) = 86$$

Watershed:

$$\frac{0.009+0.025+0.010}{0.398}(68)+\frac{0.124}{0.398}(85)+\frac{0.128}{0.398}(89)+\frac{0.102}{0.398}(72) = 81$$

APPENDIX C: MODIFIED U.S.L.E. INPUT

M.U.S.L.E. INPUT

EXISTING CONDITION

K = 0.20 Loamy Fine Sand
(Table 1, p. 4, Ref. 17)

C = 0.33, No appreciable canopy, 0-20% ground cover
(Table 4, p. 10, Ref. 17)

P = 1, No erosion control

LS = 1/2[1/2(1.4+1.8)+1/2(1.8+2.2)] = 1.8
(Table 2, p. 6, Ref. 17)

L = 50' (Field determined)
S = 0.15 (Determined from map)

DEVELOPED CONDITION

K = 0.13 (Weighted)

$$\frac{0.264}{0.398}(0.20)+\frac{0.134}{0.398}(0) = 0.13$$

where total area = 0.398 SM
range land = 0.264 SM (K = 0.20)
impervious area = 0.134 SM (K = 0)

C = 0.22 (Weighted)

$$\frac{0.264}{0.398}(0.33)+\frac{0.134}{0.398}(0) = 0.22$$

P = 1

LS = 1.8

APPENDIX D: MEYER-PETER, MULLER BEDLOAD TRANSPORT COMPUTATIONS

JUN 2 3 1982

FOX Consulting Engineers and Geologists

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(505) 884-0900 – FMFOX

June 22, 1982

Tom Mann Associates, Inc.
811 Dallas N.E.
Albuquerque, N.M. 87110

Job No. 424480

Subject: Sieve Analyses on soils for Airport - Golf Course Drainage Study

Gentlemen:

Transmitted herein is the detailed test data for the subject project.

F.M. FOX & ASSOCIATES OF THE SOUTHWEST, INC.

Reviewed by:

CHARLES M. MILLER NEW MEXICO 6018 REGISTERED PROFESSIONAL ENGINEER

Larry L. Renfro
Laboratory Supervisor

Charles M. Miller, P.E.
Principal Geotechnical Engineer

Copies: Addressee (2)

Attached: Data Sheets (2)

kn

Job No. 424480

SAMPLE A

Project: Airport - Golf Course Drainage Study

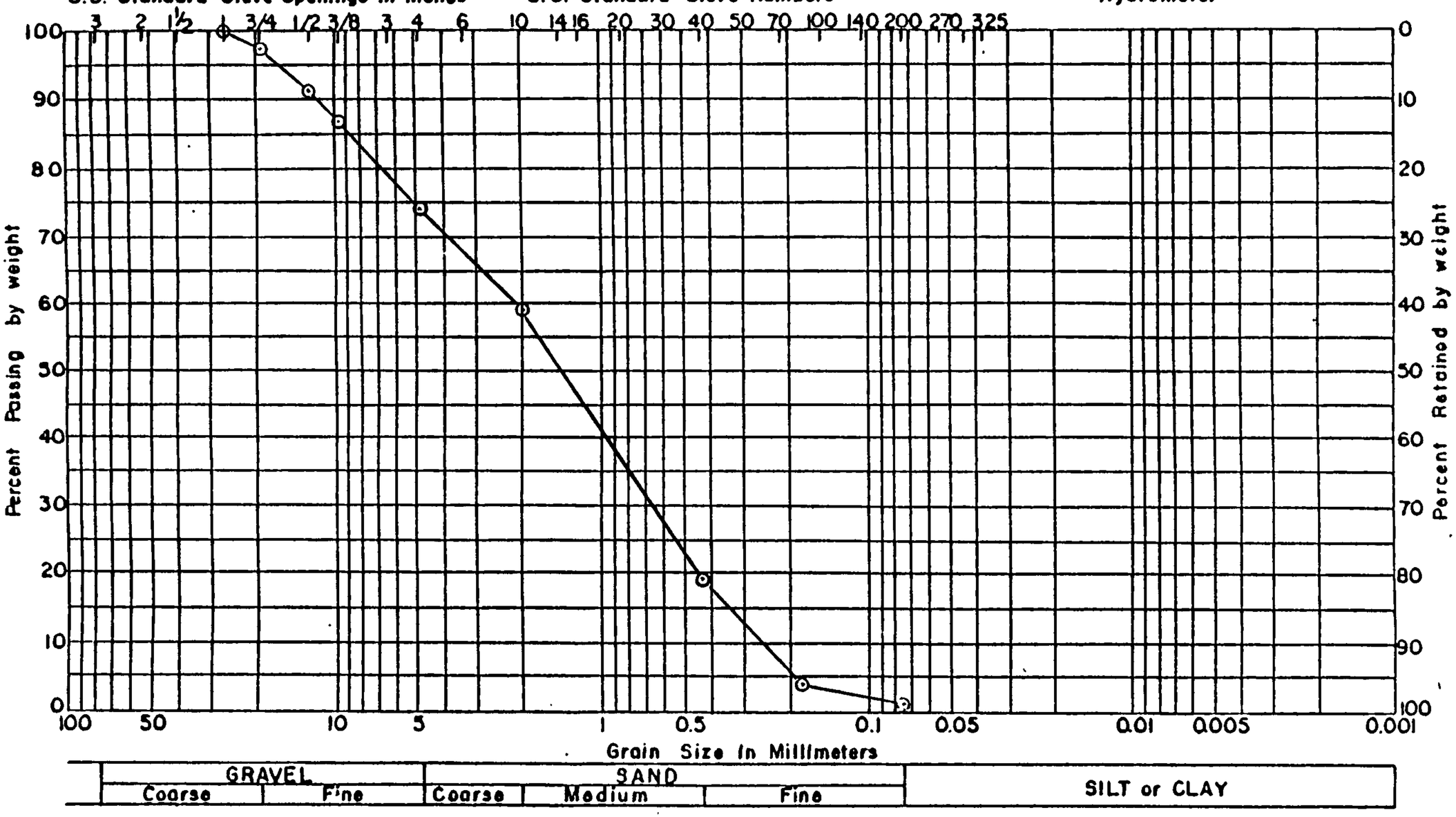
Sieve Analysis on soil sampled from 50' West of downstream end of culvert under University Boulevard S.E., sampled by Tom Mann Associates, Inc., on June 21, 1982.

SIEVE ANALYSIS

Sieve Size	% Passing
1"	100
3/4"	98
1/2"	91
3/8"	87
No. 4	74
No. 10	59
No. 40	19
No. 80	4
No. 200	1.1

Form 1-116

MECHANICAL ANALYSIS CHART



SAMPLE A

Job No. 424480

SAMPLE B

Project: Airport - Golf Course Drainage Study

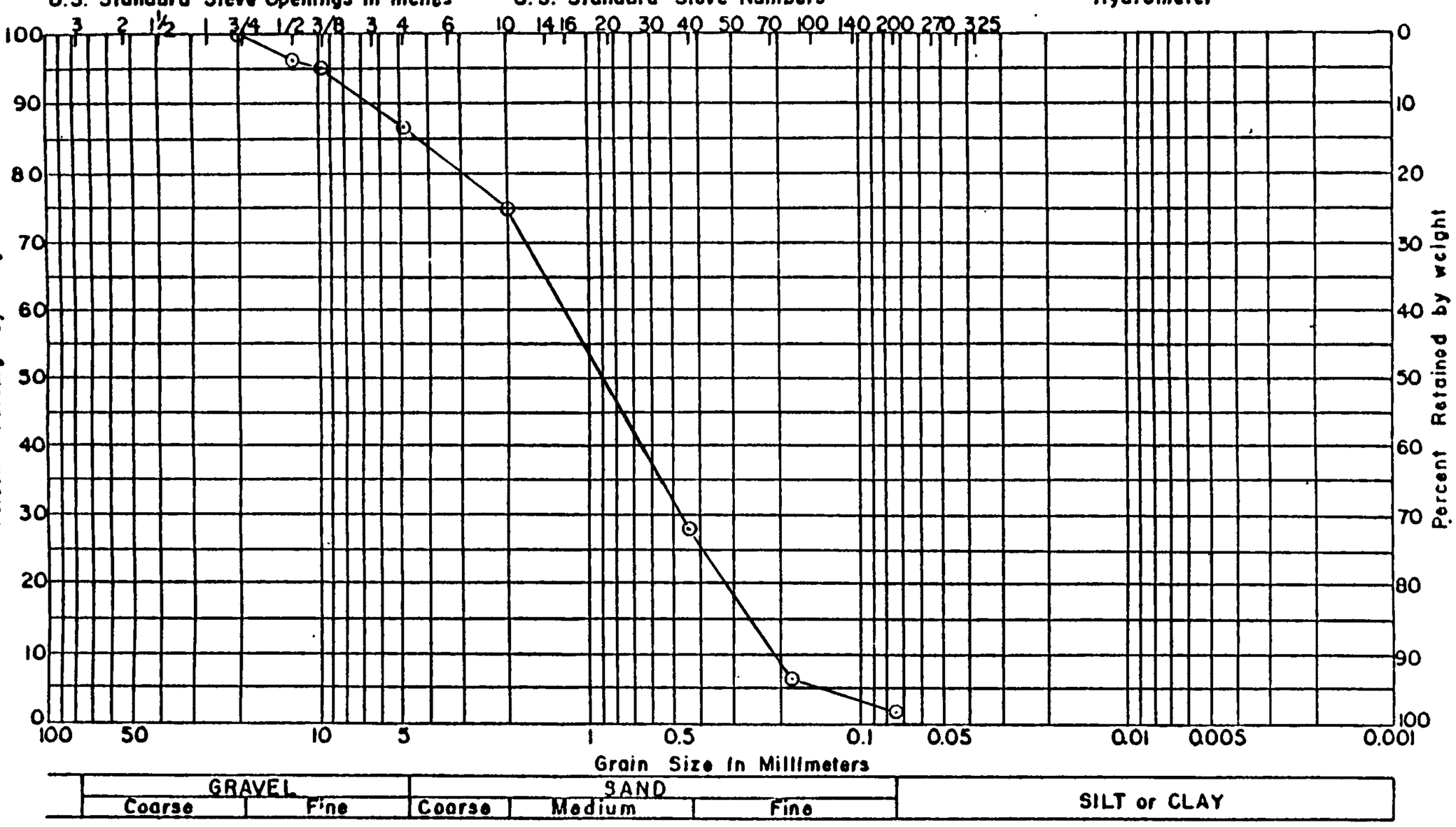
Sieve Analysis on soil sampled from 75' East of upstream end of culvert on University Boulevard S.E., sampled by Tom Mann Associates, Inc., on June 21, 1982.

SIEVE ANALYSIS

Sieve Size	% Passing
3/4"	100
1/2"	96
3/8"	95
No. 4	87
No. 10	75
No. 40	28
No. 80	7
No. 200	2.1

Form 1-116

MECHANICAL ANALYSIS CHART



SAMPLE B

D-6

COMPUTATION OF BEDLOAD TRANSPORT BY MEYER-PETER, MULLER FORMULA FOR ONE DISCHARGE

~~River~~ AIRPORT / GOLF COURSE
Date _____
Time _____
Section SAMPLE A

Computed by JGM
Checked by JGM
Date 7/8/82
Sheet 1 of 12

FORMULA

$$g_s = 1.606 \left[3.306 \left(\frac{Q_s}{Q}\right) \left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} dS - 0.627\ D_m\right]^{3/2} \text{ tons/day/ft. width}$$

CONDITION: EXISTING
EVENT: 2-YR, 6-HR

HYDRAULIC DATA

Q - Discharge = 12.3 cfs
d - depth = 0.4 ft
B - width = 70 ft.
S - slope = 0.06 ft./ft.
n_m - total stream "n" = 0.025
n_w - channel sides "n" = 0.025

SIZE ANALYSIS - BED MATERIAL

Size Fraction mm	Geometric Mean D mm	ΔP Percent	DΔP
< .0625		1	—
.0625 - .125	0884	2	0.1768
125 - .25	.177	7	1.2390
.25 - .5	.354	13	4.6020
.5 - 1.0	.707	18	12.7260
1.0 - 2.0	1 41	18	25.3800
2.0 - 4 0	2.83	11	31.1300
4.0 - 8.0	5.66	13	73.5800
8 0 - 16 0	11.32	10	113.2000
16.0 - 32.0	22.63	7	158.4100
Total		100	420.4438

BEDLOAD TRANSPORT COMPUTATIONS

FIGURE 1

$\frac{n_w}{n_m} = \frac{\quad}{\quad} = 1$

$\frac{d}{B} = \frac{0.4}{70} = 0.0057$

n_s (from Nomograph) = 0.025

$D_m = \frac{\Sigma D \Delta P}{100} = 4.2044$

D_{90} = 11 mm

$D_{90}^{1/6} = 1.4913$

$\left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} = \left(\frac{1.4913}{0.025}\right)^{3/2} = 59.692 = 460.72$

FIGURE 2

$\frac{n_w}{n_s} = \frac{\quad}{\quad} = 1$

$\frac{Q_s}{Q}$ (from Nomograph) = 1

$g_s = 1.606 \left[3.306 (1)(460.72)(0.4)(0.06) - 0.627(4.2044)\right]^{3/2}$

$g_s = 1.606 \left[36.5554 - 2.63762\right]^{3/2}$

$g_s = 1.606 \left[33.9192\right]^{3/2}$

$g_s = 1.606 (197.55)$

$g_s = 317.3$ tons / day / ft. width

$G_s = 317.3 (B) = 22,208.2$ tons / day

COMPUTATION OF BEDLOAD TRANSPORT BY MEYER-PETER, MULLER FORMULA FOR ONE DISCHARGE

~~River~~ AIRPORT/GOLF COURSE
Date
Time
Section SAMPLE B

Computed by JGM
Checked by JGM
Date 7/8/82
Sheet 2 of 12

FORMULA

$$g_s = 1.606 \left[3.306 \left(\frac{Q_s}{Q}\right) \left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} dS - 0.627\ D_m\right]^{3/2} \text{ tons/day/ft. width}$$

CONDITION: EXISTING
EVENT: 2 YR, 6 HR

HYDRAULIC DATA

Q - Discharge = 12.3 cfs
d - depth = 0.4 ft
B - width = 70 ft.
S - slope = 0.06 ft/ft.
n_m - total stream "n" = 0.025
n_w - channel sides "n" = 0.025

SIZE ANALYSIS - BED MATERIAL

Size Fraction mm	Geometric Mean D mm	ΔP Percent	DΔP
<.0625		1	—
.0625-.125	.0884	4	0.3536
125-.25	.177	9	1.5930
.25 - .5	.354	18	6.3720
.5 - 1.0	.707	22	15.5540
1.0 - 2.0	1 41	21	29.6100
2.0 - 4 0	2 83	9	25.4700
4 0 - 8 0	5 66	9	50.9400
8.0 - 16 0	11. 32	4	45.2800
16.0 - 32.0	22.63	3	67.8900
Total		100	243.0626

BEDLOAD TRANSPORT COMPUTATIONS

FIGURE 1

$\frac{n_w}{n_m}$ = ——— = 1

$\frac{d}{B} = \frac{0.4}{70} = 0.0057$

n_s (from Nomograph) = 0.025

FIGURE 2

$\frac{n_w}{n_s}$ = ——— = 1

$\frac{Q_s}{Q}$ (from Nomograph) = 1

$D_m = \frac{\Sigma D \Delta P}{100} = 2.4306$

D_{90} = 6.3 mm

$D_{90}^{1/6}$ = 1.3590

$\left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} = \left(\frac{1.3590}{0.025}\right)^{3/2} = 54.36^{3/2} = 400.79$

$g_s = 1.606\ [3.306(1)(400.79)(0.4)(0.06) - 0.627(2.4306)]^{3/2}$

$g_s = 1.606\ [31.8003 - 1.5240]^{3/2}$

$g_s = 1.606\ [30.2763]^{3/2}$

$g_s = 1.606\ (166.59)$

g_s = 267.5 tons / day / ft. width

$G_s = 267.5\ (B) = 18{,}728.3$ tons / day

COMPUTATION OF BEDLOAD TRANSPORT BY MEYER-PETER, MULLER FORMULA FOR ONE DISCHARGE

~~River~~ AIRPORT / GOLF COURSE

Date _____

Time _____

Section SAMPLE A

Computed by JGM

Checked by JGM

Date 7/8/82

Sheet 3 of 12

FORMULA

$$g_s = 1.606 \left[3.306 \left(\frac{Q_s}{Q}\right) \left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} dS - 0.627\ D_m\right]^{3/2} \text{ tons/day/ft. width}$$

CONDITION: EXISTING

EVENT: 5 YR, 6 HR

HYDRAULIC DATA

Q - Discharge = 32.7 cfs

d - depth = 0.5 ft

B - width = 70 ft.

S - slope = 0.06 ft./ft.

n_m - total stream "n" = 0.025

n_w - channel sides "n" = 0.025

SIZE ANALYSIS - BED MATERIAL

Size Fraction mm	Geometric Mean D mm	ΔP Percent	DΔP
< .0625		1	—
.0625 - .125	.0884	2	0.1768
125 - .25	.177	7	1.2390
.25 - .5	.354	13	4.6020
.5 - 1.0	.707	18	12.7260
1.0 - 2 0	1 41	18	25.3800
2.0 - 4 0	2 83	11	31.1300
4.0 - 8 0	5.66	13	73.5800
8.0 - 16 0	11. 32	10	113.2000
16.0 - 32.0	22.63	7	158.4100
Total		100	420.4438

BEDLOAD TRANSPORT COMPUTATIONS

FIGURE 1

$\frac{n_w}{n_m} = \frac{\quad}{\quad} = 1$

$\frac{d}{B} = \frac{0.5}{70} = 0.0071$

n_s (from Nomograph) = 0.025

FIGURE 2

$\frac{n_w}{n_s} = \frac{\quad}{\quad} = 1$

$\frac{Q_s}{Q}$ (from Nomograph) = 0.99

$D_m = \frac{\Sigma D \Delta P}{100} = 4.2044$

D_{90} = 11 mm

$D_{90}^{1/6} = 1.4913$

$\left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} = \left(\frac{1.4913}{0.025}\right)^{3/2} = 59.652^{3/2} = 460.72$

$g_s = 1.606 \left[3.306(0.99)(460.72)(0.5)(0.06) - 0.627(4.2044)\right]^{3/2}$

$g_s = 1.606 \left[45.2373 - 2.6362\right]^{3/2}$

$g_s = 1.606 \left[42.6011\right]^{3/2}$

$g_s = 1.606\ (278.0550)$

$g_s = 446.6$ tons / day / ft. width

$G_s = 446.6\ (B) = 31,258.9$ tons / day

COMPUTATION OF BEDLOAD TRANSPORT BY MEYER-PETER, MULLER FORMULA FOR ONE DISCHARGE

~~River~~ AIRPORT/GOLF COURSE
Date
Time
Section SAMPLE B

Computed by JGM
Checked by JGM
Date 7/8/82
Sheet 4 of 12

FORMULA

$$g_s = 1.606 \left[3.306 \left(\frac{Q_s}{Q}\right) \left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} dS - 0.627\ D_m\right]^{3/2} \text{ tons/day/ft. width}$$

CONDITION: EXISTING
EVENT: 5YR. 6 HR

HYDRAULIC DATA

Q - Discharge = 32.7 cfs
d - depth = 0.5 ft
B - width = 70 ft.
S - slope = 0.06 ft./ft.
n_m - total stream "n" = 0.025
n_w - channel sides "n" = 0.025

SIZE ANALYSIS - BED MATERIAL

Size Fraction mm	Geometric Mean D mm	ΔP Percent	DΔP
< .0625		1	—
.0625-.125	0884	4	0.3536
125-.25	.177	9	1.5930
.25 - .5	.354	18	6.3720
.5 - 1.0	.707	22	15.5540
1.0 - 2.0	1.41	21	29.6100
2.0 - 4.0	2.83	9	25.4700
4.0 - 8.0	5.66	9	50.9400
8.0 - 16.0	11.32	4	45.2800
16.0 - 32.0	22.63	3	67.8900
Total		100	243.0626

BEDLOAD TRANSPORT COMPUTATIONS

FIGURE 1

$\frac{n_w}{n_m} = \frac{\quad}{\quad} = 1$

$\frac{d}{B} = \frac{0.5}{70} = 0.0071$

n_s (from Nomograph) = 0.025

FIGURE 2

$\frac{n_w}{n_s} = \frac{\quad}{\quad} = 1$

$\frac{Q_s}{Q}$ (from Nomograph) = 0.99

$D_m = \frac{\Sigma D\Delta P}{100} = 2.4306$

D_{90} = 6.3 mm

$D_{90}^{1/6} = 1.3590$

$\left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} = \left(\frac{1.3590}{0.025}\right)^{3/2} = 54.36^{3/2} = 400.79$

$g_s = 1.606 \left[3.306(0.99)(400.79)(0.5)(0.06) - 0.627(2.4306)\right]^{3/2}$

$g_s = 1.606 \left[39.3528 - 1.5240\right]^{3/2}$

$g_s = 1.606 \left[37.8288\right]^{3/2}$

$g_s = 1.606\ (232.6669)$

$g_s = 373.7$ tons / day / ft. width

$G_s = 373.7\ (B) = 26,156.4$ tons / day

COMPUTATION OF BEDLOAD TRANSPORT BY MEYER-PETER, MULLER FORMULA FOR ONE DISCHARGE

~~River~~ AIRPORT / GOLF COURSE
Date _____
Time _____
Section SAMPLE A

Computed by JGM
Checked by JGM
Date 7/8/82
Sheet 5 of 12

FORMULA

$$g_s = 1.606 \left[3.306 \left(\frac{Q_s}{Q}\right) \left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} dS - 0.627\ D_m\right]^{3/2} \text{ tons/day/ft. width}$$

CONDITION: EXISTING
EVENT: 10YR, 6 HR

HYDRAULIC DATA

Q - Discharge = 48.5 cfs
d - depth = 0.55 ft
B - width = 70 ft.
S - slope = 0.06 ft./ft.
n_m - total stream "n" = 0.025
n_w - channel sides "n" = 0.025

SIZE ANALYSIS - BED MATERIAL

Size Fraction mm	Geometric Mean D mm	ΔP Percent	DΔP
< .0625		1	—
.0625 - .125	.0884	2	0.1768
125 - .25	.177	7	1.2390
.25 - .5	.354	13	4.6020
.5 - 1.0	.707	18	12.7260
1.0 - 2.0	1.41	18	25.3800
2.0 - 4.0	2.83	11	31.1300
4.0 - 8.0	5.66	13	73.5800
8.0 - 16.0	11.32	10	113.2000
16.0 - 32.0	22.63	7	158.4100
Total		100	420.4438

BEDLOAD TRANSPORT COMPUTATIONS

FIGURE 1

$\frac{n_w}{n_m} = \frac{\quad}{\quad} = 1$

$\frac{d}{B} = \frac{0.55}{70} = 0.0079$

n_s (from Nomograph) = 0.025

FIGURE 2

$\frac{n_w}{n_s} = \frac{\quad}{\quad} = 1$

$\frac{Q_s}{Q}$ (from Nomograph) = 0.985

$D_m = \frac{\Sigma D \Delta P}{100} = 4.2044$

D_{90} = 11 mm

$D_{90}^{1/6} = 1.4913$

$\left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} = \left(\frac{1.4913}{0.025}\right)^{3/2} = 59.652 = 460.72$

$g_s = 1.606 \left[3.306(0.985)(460.72)(0.55)(0.06) - 0.627(4.2044)\right]^{3/2}$

$g_s = 1.606 \left[49.5097 - 2.6362\right]^{3/2}$

$g_s = 1.606 \left[46.8735\right]^{3/2}$

$g_s = 1.606\ (320.9155)$

$g_s = 515.4$ tons / day / ft. width

$G_s = 515.4\ (B) = 36,077.3$ tons / day

COMPUTATION OF BEDLOAD TRANSPORT BY MEYER-PETER, MULLER FORMULA FOR ONE DISCHARGE

~~River~~ AIRPORT/GOLF COURSE
Date
Time
Section SAMPLE B

Computed by JGM
Checked by JGM
Date 2/8/82
Sheet 6 of 12

FORMULA

$$g_s = 1.606 \left[3.306 \left(\frac{Q_s}{Q}\right) \left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} dS - 0.627\ D_m\right]^{3/2} \text{ tons/day/ft. width}$$

CONDITION: EXISTING
EVENT: 10 YR, 6 HR

HYDRAULIC DATA

Q - Discharge = 48.5 cfs
d - depth = 0.55 ft
B - width = 70 ft.
S - slope = 0.06 ft/ft.
n_m - total stream "n" = 0.025
n_w - channel sides "n" = 0.025

SIZE ANALYSIS - BED MATERIAL

Size Fraction mm	Geometric Mean D mm	ΔP Percent	DΔP
< .0625		1	—
.0625-.125	.0884	4	0.3536
125-.25	.177	9	1.5930
.25 - .5	.354	18	6.3720
.5 - 1.0	.707	22	15.5540
1.0 - 2.0	1 41	21	29.6100
2 0 - 4 0	2.83	9	25.4700
4 0 - 8.0	5.66	9	50.9400
8.0 - 16 0	11.32	4	45.2800
16 0 - 32.0	22.63	3	67.8900
Total		100	243.0626

BEDLOAD TRANSPORT COMPUTATIONS

FIGURE 1

$\frac{n_w}{n_m}$ = ——— = 1

$\frac{d}{B}$ = ——— = 0.0079

n_s (from Nomograph) = 0.025

$D_m = \frac{\Sigma D \Delta P}{100}$ = 2.43306

D_{90} = 6.3 mm

$D_{90}^{1/6}$ = 1.3590

$\left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} = \left(\frac{1.3590}{0.025}\right)^{3/2} = 54.36^{3/2} = 400.79$

FIGURE 2

$\frac{n_w}{n_s}$ = ——— = 1

$\frac{Q_s}{Q}$ (from Nomograph) = 0.985

$g_s = 1.606 \left[3.306(0.985)(400.79)(0.55)(0.06) - 0.627(2.4306)\right]^{3/2}$

$g_s = 1.606 \left[43.0695 - 1.5240\right]^{3/2}$

$g_s = 1.606 \left[41.5455\right]^{3/2}$

$g_s = 1.606\ (267.7849)$

g_s = 430.1 tons / day / ft. width

G_s = 430.1 (B) = 30,104.4 tons / day

COMPUTATION OF BEDLOAD TRANSPORT BY MEYER-PETER, MULLER FORMULA FOR ONE DISCHARGE

~~River~~ AIRPORT / GOLF COURSE
Date
Time
Section SAMPLE A

Computed by JGM
Checked by JGM
Date 7/8/82
Sheet 7 of 12

FORMULA

$$g_s = 1.606 \left[3.306 \left(\frac{Q_s}{Q}\right) \left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} dS - 0.627\ D_m\right]^{3/2} \text{ tons/day/ft. width}$$

CONDITION: EXISTING
EVENT: 25 YR, 6 HR

HYDRAULIC DATA

Q - Discharge = 76.4 cfs
d - depth = 0.66 ft
B - width = 70 ft.
S - slope = 0.06 ft./ft.
n_m - total stream "n" = 0.025
n_w - channel sides "n" = 0.025

SIZE ANALYSIS - BED MATERIAL

Size Fraction mm	Geometric Mean D mm	ΔP Percent	DΔP
< .0625		1	—
.0625 - .125	.0884	2	0.1768
125 - .25	.177	7	1.2390
.25 - .5	.354	13	4.6020
.5 - 1.0	.707	18	12.7260
1.0 - 2.0	1 41	18	25.3800
2 0 - 4 0	2.83	11	31.1300
4 0 - 8 0	5 66	13	73.5800
8 0 - 16 0	11.32	10	113.2000
16 0 - 32.0	22.63	7	158.4100
Total		100	420.4438

BEDLOAD TRANSPORT COMPUTATIONS

FIGURE 1

$\frac{n_w}{n_m} = \frac{\quad}{\quad} = 1$

$\frac{d}{B} = \frac{0.66}{70} = 0.0094$

n_s (from Nomograph) = 0.025

$D_m = \frac{\Sigma D \Delta P}{100} = 4.2044$

D_{90} = 11 mm

$\left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} = \left(\frac{1.4913}{0.025}\right)^{3/2} = 59.652^{3/2} = 460.72$

FIGURE 2

$\frac{n_w}{n_s} = \frac{\quad}{\quad} = 1$

$\frac{Q_s}{Q}$ (from Nomograph) = 0.98

$D_{90}^{1/6} = 1.4913$

$g_s = 1.606 \left[3.306(0.98)(460.72)(0.66)(0.06) - 0.627(4.2044)\right]^{3/2}$

$g_s = 1.606 \left[59.1100 - 2.63762\right]^{3/2}$

$g_s = 1.606 \left[56.4738\right]^{3/2}$

$g_s = 1.606\ (424.3956)$

$g_s = 681.6$ tons / day / ft. width

$G_s = 681.6\ (B) = 47,710.6$ tons / day

COMPUTATION OF BEDLOAD TRANSPORT BY MEYER-PETER, MULLER FORMULA FOR ONE DISCHARGE

~~River~~ AIRPORT/GOLF COURSE
Date
Time
Section SAMPLE B

Computed by JGM
Checked by JGM
Date 7/8/82
Sheet 8 of 12

FORMULA

$$g_s = 1.606 \left[3.306 \left(\frac{Q_s}{Q}\right) \left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} dS - 0.627\ D_m\right]^{3/2} \text{ tons/day/ft. width}$$

CONDITION: EXISTING
EVENT: 25 YR, 6 HR

HYDRAULIC DATA

Q - Discharge = 76.6 cfs
d - depth = 0.66 ft
B - width = 70 ft.
S - slope = 0.06 ft./ft.
n_m - total stream "n" = 0.025
n_w - channel sides "n" = 0.025

SIZE ANALYSIS - BED MATERIAL

Size Fraction mm	Geometric Mean D mm	ΔP Percent	DΔP
< .0625		1	—
.0625-.125	.0884	4	0.3536
125-.25	.177	9	1.5930
.25 - .5	.354	18	6.3720
.5 - 1.0	.707	22	15.5540
1.0 - 2.0	1 41	21	29.6100
2.0 - 4 0	2.83	9	25.4700
4 0 - 8.0	5 66	9	50.9400
8.0 - 16 0	11. 32	4	45.2800
16.0 - 32.0	22.63	3	67.8900
Total		100	243.0626

BEDLOAD TRANSPORT COMPUTATIONS

FIGURE 1

$\frac{n_w}{n_m}$ = ——— = 1

$\frac{d}{B}$ = ——— = 0.0094

n_s (from Nomograph) = 0.025

FIGURE 2

$\frac{n_w}{n_s}$ = ——— = 1

$\frac{Q_s}{Q}$ (from Nomograph) = 0.98

$D_m = \frac{\Sigma D\Delta P}{100} = 2.4306$

D_{90} = 6.3 mm

$D_{90}^{1/6} = 1.3590$

$\left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} = \left(\frac{1.3590}{0.025}\right)^{3/2} = 54.36^{3/2} = 400.79$

$g_s = 1.606 \left[3.306(0.98)(400.79)(0.66)(0.06) - 0.627(2.4306)\right]^{3/2}$

$g_s = 1.606 \left[51.4211 - 1.5240\right]^{3/2}$

$g_s = 1.606 \left[49.8971\right]^{3/2}$

$g_s = 1.606\ (352.4621)$

g_s = 566.1 tons / day / ft. width

$G_s = 566.1\ (B) = 39{,}623.8$ tons / day

COMPUTATION OF BEDLOAD TRANSPORT BY MEYER-PETER, MULLER FORMULA FOR ONE DISCHARGE

~~River~~ AIRPORT / GOLF COURSE
Date _____
Time _____
Section SAMPLE A

Computed by JGM
Checked by JGM
Date 7/8/82
Sheet 9 of 12

FORMULA

$$g_s = 1.606 \left[3.306 \left(\frac{Q_s}{Q}\right) \left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} dS - 0.627\ D_m\right]^{3/2} \text{ tons/day/ft. width}$$

CONDITION: EXISTING
EVENT: 50 YR, 6-HR

HYDRAULIC DATA

Q - Discharge = 154.4 cfs
d - depth = 0.86 ft
B - width = 70 ft.
S - slope = 0.06 ft./ft.
n_m - total stream "n" = 0.025
n_w - channel sides "n" = 0.025

SIZE ANALYSIS - BED MATERIAL

Size Fraction mm	Geometric Mean D mm	ΔP Percent	DΔP
< .0625		1	—
.0625 - .125	.0884	2	0.1768
125 - .25	.177	7	1.2390
.25 - .5	.354	13	4.6020
.5 - 1.0	.707	18	12.7260
1.0 - 2 0	1 41	18	25.3800
2.0 - 4 0	2.83	11	31.1300
4 0 - 8.0	5 66	13	73.5800
8 0 - 16 0	11.32	10	113.2000
16 0 - 32.0	22.63	7	158.4100
Total		100	420.4438

BEDLOAD TRANSPORT COMPUTATIONS

FIGURE 1

$\frac{n_w}{n_m} = \frac{\quad}{\quad} = 1$

$\frac{d}{B} = \frac{0.86}{70} = 0.0123$

n_s (from Nomograph) = 0.025

FIGURE 2

$\frac{n_w}{n_s} = \frac{\quad}{\quad} = 1$

$\frac{Q_s}{Q}$ (from Nomograph) = 0.98

$D_m = \frac{\Sigma D \Delta P}{100} = 4.2044$

D_{90} = 11 mm

$D_{90}^{1/6} = 1.4913$

$\left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} = \left(\frac{1.4913}{0.025}\right)^{3/2} = 59.692^{3/2} = 460.72$

$g_s = 1.606 \left[3.306(0.98)(460.72)(0.86)(0.06) - 0.627(4.2044)\right]^{3/2}$

$g_s = 1.606 \left[77.0222 - 2.6362\right]^{3/2}$

$g_s = 1.606 \left[74.3860\right]^{3/2}$

$g_s = 1.606\ (641.5588)$

$g_s = 1030.3$ tons / day / ft. width

$G_s = 1030.3\ (B) = 72,124.0$ tons / day

COMPUTATION OF BEDLOAD TRANSPORT BY MEYER-PETER, MULLER FORMULA FOR ONE DISCHARGE

~~River~~ AIRPORT/GOLF COURSE
Date
Time
Section SAMPLE B

Computed by JGM
Checked by JGM
Date 7/8/82
Sheet 10 of 12

FORMULA

$$g_s = 1.606 \left[3.306 \left(\frac{Q_s}{Q}\right) \left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} dS - 0.627\ D_m\right]^{3/2} \text{ tons/day/ft. width}$$

CONDITION: EXISTING
EVENT: 50YR, 6-HR

HYDRAULIC DATA

Q - Discharge = 156.4 cfs
d - depth = 0.86 ft
B - width = 70 ft.
S - slope = 0.06 ft/ft.
n_m - total stream "n" = 0.025
n_w - channel sides "n" = 0.025

SIZE ANALYSIS - BED MATERIAL

Size Fraction mm	Geometric Mean D mm	ΔP Percent	DΔP
<.0625		1	—
.0625-.125	.0884	4	0.3536
125-.25	.177	9	1.5930
.25 - .5	.354	18	6.3720
.5 - 1.0	.707	22	15.5540
1.0 - 2.0	1 41	21	29.6100
2 0 - 4 0	2.83	9	25.4700
4 0 - 8.0	5.66	9	50.9400
8 0 - 16 0	11. 32	4	45.2800
16 0 - 32.0	22.63	3	67.8900
Total		100	243.0626

BEDLOAD TRANSPORT COMPUTATIONS

FIGURE 1

$\frac{n_w}{n_m}$ = ——— = 1

$\frac{d}{B}$ = ——— = 0.0123

n_s (from Nomograph) = 0.025

FIGURE 2

$\frac{n_w}{n_s}$ = ——— = 1

$\frac{Q_s}{Q}$ (from Nomograph) = 0.98

$D_m = \frac{\Sigma D \Delta P}{100}$ = 2.43306

D_{90} = 6.3 mm

$D_{90}^{1/6}$ = 1.3590

$\left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} = \left(\frac{1.3590}{0.025}\right)^{3/2} = 54.36^{3/2} = 400.79$

$g_s = 1606\ [3.306(0.98)(400.79)(0.86)(0.06) - 0.627(2.4306)]^{3/2}$

$g_s = 1.606\ [67.0032 - 1.5240]^{3/2}$

$g_s = 1.606\ [65.4792]^{3/2}$

$g_s = 1.606\ (529.8527)$

g_s = 850.9 tons / day / ft. width

G_s = 850.9 (B) = 59,566.0 tons / day

COMPUTATION OF BEDLOAD TRANSPORT BY MEYER-PETER, MULLER FORMULA FOR ONE DISCHARGE

~~River~~ AIRPORT / GOLF COURSE

Date _____

Time _____

Section SAMPLE A

Computed by JGM

Checked by JGM

Date 7/8/82

Sheet 11 of 12

FORMULA

$$g_s = 1.606 \left[3.306 \left(\frac{Q_s}{Q}\right) \left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} dS - 0.627\ D_m\right]^{3/2} \text{ tons/day/ft. width}$$

CONDITION: EXISTING

EVENT: 100 YR, 6-HR

HYDRAULIC DATA

Q - Discharge = 197.8 cfs

d - drpth = 0.95 ft

B - width = 70 ft.

S - slope = 0.06 ft./ft.

n_m - total stream "n" = 0.025

n_w - channel sides "n" = 0.025

SIZE ANALYSIS - BED MATERIAL

Size Fraction mm	Geometric Meon D mm	ΔP Percent	DΔP
< .0625		1	—
.0625 - .125	.0884	2	0.1768
125 - .25	.177	7	1.2390
.25 - .5	.354	13	4.6020
.5 - 1.0	.707	18	12.7260
1 0 - 2.0	1 41	18	25.3800
2.0 - 4 0	2 83	11	31.1300
4 0 - 8 0	5 66	13	73.5800
8 0 - 16 0	11. 32	10	113.2000
16 0 - 32.0	22.63	7	158.4100
Total		100	420.4438

BEDLOAD TRANSPORT COMPUTATIONS

FIGURE 1

$\frac{n_w}{n_m} = \frac{\quad}{\quad} = 1$

$\frac{d}{B} = \frac{0.95}{70} = 0.0136$

n_s (from Nomograph) = 0.025

FIGURE 2

$\frac{n_w}{n_s} = \frac{\quad}{\quad} = 1$

$\frac{Q_s}{Q}$ (from Nomograph) = 0.965

$D_m = \frac{\Sigma D \Delta P}{100} = 4.2044$

D_{90} = 11 mm

$D_{90}^{1/6} = 1.4913$

$\left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} = \left(\frac{1.4913}{0.025}\right)^{3/2} = 59.672^{3/2} = 460.72$

$g_s = 1.606 \left[3.306(0.965)(460.72)(0.95)(0.06) - 0.627(4.2044)\right]^{3/2}$

$g_s = 1.606 \left[83.7803 - 2.6362\right]^{3/2}$

$g_s = 1.606 \left[81.1441\right]^{3/2}$

$g_s = 1.606\ (730.9467)$

$g_s = 1173.9$ tons / day / ft. width

$G_s = 1173.9\ (8) = 82,173.0$ tons / day

COMPUTATION OF BEDLOAD TRANSPORT BY MEYER-PETER, MULLER FORMULA FOR ONE DISCHARGE

~~River~~ AIRPORT/GOLF COURSE
Date
Time
Section SAMPLE B

Computed by JGM
Checked by JGM
Date 7/8/82
Sheet 12 of 12

FORMULA

$$g_s = 1.606 \left[3.306 \left(\frac{Q_s}{Q}\right) \left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} dS - 0.627\ D_m\right]^{3/2} \text{ tons/day/ft. width}$$

CONDITION: EXISTING
EVENT: 100YR, 6 HR

HYDRAULIC DATA

Q - Discharge = 197.8 cfs
d - depth = 0.95 ft
B - width = 70 ft.
S - slope = 0.06 ft/ft.
n_m - total stream "n" = 0.025
n_w - channel sides "n" = 0.025

SIZE ANALYSIS - BED MATERIAL

Size Fraction mm	Geometric Mean D mm	ΔP Percent	DΔP
< .0625		1	—
.0625 - .125	.0884	4	0.3536
.125 - .25	.177	9	1.5930
.25 - .5	.354	18	6.3720
.5 - 1.0	.707	22	15.5540
1.0 - 2.0	1.41	21	29.6100
2.0 - 4.0	2.83	9	25.4700
4.0 - 8.0	5.66	9	50.9400
8.0 - 16.0	11.32	4	45.2800
16.0 - 32.0	22.63	3	67.8900
Total		100	243.0626

BEDLOAD TRANSPORT COMPUTATIONS

FIGURE 1

$\frac{n_w}{n_m}$ = ——— = 1

$\frac{d}{B}$ = ——— = 0.0136

n_s (from Nomograph) = 0.025

FIGURE 2

$\frac{n_w}{n_s}$ = ——— = 1

$\frac{Q_s}{Q}$ (from Nomograph) = 0.965

$D_m = \frac{\Sigma D \Delta P}{100} = 2.4306$

D_{90} = 6.3 mm

$D_{90}^{1/6} = 1.3590$

$\left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} = \left(\frac{1.3590}{0.025}\right)^{3/2} = 54.36^{3/2} = 400.79$

$g_s = 1.606 \left[3.306(0.965)(400.79)(0.95)(0.06) - 0.627(2.4306)\right]^{3/2}$

$g_s = 1.606 \left[72.8823 - 1.5240\right]^{3/2}$

$g_s = 1.606 \left[71.3583\right]^{3/2}$

$g_s = 1.606\ (602.7906)$

g_s = 968.1 tons / day / ft. width

$G_s = 968.1\ (B) = 67,765.7$ tons / day

EXISTING CONDITION : AVG. SEDIMENT YIELD PER EVENT

2-YR :

A $22,208.2 \text{ TONS/DAY} (3)(0.60 \text{ HRS})(\frac{1}{24 \text{ HRS}}) = 1665.6 \text{ TONS}$

B $18,728.3 (3)(0.60)(\frac{1}{24}) = 1404.6 \text{ TONS}$

AVG. BED LOAD = 1535.1 TONS

5-YR :

A $31,258.9 \text{ TONS/DAY} (3)(0.54)(\frac{1}{24 \text{ HRS}}) = 2110.0 \text{ TONS}$

B $26,156.4 (3)(0.54)(\frac{1}{24}) = 1765.6 \text{ TONS}$

AVG. BED LOAD = 1937.8 TONS

10-YR :

A $36,077.3 (3)(0.54)(\frac{1}{24}) = 2435.2 \text{ TONS}$

B $30,104.4 (3)(0.54)(\frac{1}{24}) = 2032.0 \text{ TONS}$

AVG. BED LOAD = 2233.6 TONS

25-YR :

A $47,710.6 (3)(0.52)(\frac{1}{24}) = 3101.2 \text{ TONS}$

B $39,623.8 (3)(0.52)(\frac{1}{24}) = 2575.5 \text{ TONS}$

AVG. BED LOAD = 2838.4 TONS

50-YR :

A $72,124.0(3)(0.42)\left(\frac{1}{24}\right) = 3786.5$ TONS

B $59,566.0(3)(0.42)\left(\frac{1}{24}\right) = 3127.2$ TONS

AVG. BED LOAD = 3456.9 TONS

100-YR :

A $82,173.0(3)(0.38)\left(\frac{1}{24}\right) = 3903.2$ TONS

B $67,765.7(3)(0.38)\left(\frac{1}{24}\right) = 3218.9$ TONS

AVG. BED LOAD = 3561.0 TONS

COMPUTATION OF BEDLOAD TRANSPORT BY MEYER-PETER, MULLER FORMULA FOR ONE DISCHARGE

~~River~~ AIRPORT / GOLF COURSE
Date _____
Time _____
Section SAMPLE A

Computed by JGM
Checked by JGM
Date 7/9/82; 7/16/82
Sheet 1 of 12

FORMULA

$$g_s = 1.606 \left[3.306 \left(\frac{Q_s}{Q}\right) \left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} dS - 0.627\ D_m\right]^{3/2} \text{ tons/day/ft. width}$$

CONDITION: DEVELOPED
EVENT: 2 YR, 6 HR

HYDRAULIC DATA

Q - Discharge = ~~29.4~~ 29.8 cfs
d - depth = 0.46 ✓ ft
B - width = 70 ft.
S - slope = 0.06 ft./ft.
n_m - total stream "n" = 0.025
n_w - channel sides "n" = 0.025

SIZE ANALYSIS - BED MATERIAL

Size Fraction mm	Geometric Mean D mm	ΔP Percent	DΔP
< .0625		1	—
.0625 - .125	.0884	2	0.1768
125 - .25	.177	7	1.2390
.25 - .5	.354	13	4.6020
.5 - 1.0	.707	18	12.7260
1.0 - 2.0	1 41	18	25.3800
2.0 - 4 0	2.83	11	31.1300
4 0 - 8.0	5 66	13	73.5800
8.0 - 16 0	11. 32	10	113.2000
16.0 - 32.0	22.63	7	158.4100
Total		100	420.4438

BEDLOAD TRANSPORT COMPUTATIONS

FIGURE 1

$\frac{n_w}{n_m} = \frac{\quad}{\quad} = 1$

$\frac{d}{B} = \frac{0.46}{70} = 0.00656$

n_s (from Nomograph) = 0.025

$D_m = \frac{\Sigma D \Delta P}{100} = 4.2044$

D_{90} = 11 mm

$\left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} = \left(\frac{1.4913}{0.025}\right)^{3/2} = 59.692 = 460.72$

FIGURE 2

$\frac{n_w}{n_s} = \frac{\quad}{\quad} = 1$

$\frac{Q_s}{Q}$ (from Nomograph) = 0.955

$D_{90}^{1/6} = 1.4913$

$g_s = 1.606 \left[3.306 (0.985)(460.72)(0.46)(0.06) - 0.627(4.2044)\right]^{3/2}$

$g_s = 1.606 \left[41.4081 - 2.6362\right]^{3/2}$

$g_s = 1.606 \left[38.7719\right]^{3/2}$

$g_s = 1.606\ (241.4217)$

$g_s = 387.7$ tons / day / ft. width

$G_s = 387.7\ (B) = 27,140.6$ tons / day

COMPUTATION OF BEDLOAD TRANSPORT BY MEYER-PETER, MULLER FORMULA FOR ONE DISCHARGE

~~River~~ AIRPORT/GOLF COURSE
Date
Time
Section SAMPLE B

Computed by JGM
Checked by JGM
Date 7/9/82; 7/16/82
Sheet 2 of 12

FORMULA

$$g_s = 1.606 \left[3.306 \left(\frac{Q_s}{Q}\right) \left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} dS - 0.627\ D_m\right]^{3/2} \text{ tons/day/ft. width}$$

CONDITION: DEVELOPED
EVENT: 2 YR, 6 HR

HYDRAULIC DATA

Q - Discharge = ~~29.4~~ 29.8 cfs
d - depth = 0.46 ✓ ft
B - width = 70 ft.
S - slope = 0.06 ft./ft.
n_m - total stream "n" = 0.025
n_w - channel sides "n" = 0.025

SIZE ANALYSIS - BED MATERIAL

Size Fraction mm	Geometric Mean D mm	ΔP Percent	DΔP
< .0625		1	—
.0625-.125	.0884	4	0.3536
125 - .25	.177	9	1.5930
.25 - .5	.354	18	6.3720
.5 - 1.0	.707	22	15.5540
1.0 - 2.0	1 41	21	29.6100
2.0 - 4 0	2 83	9	25.4700
4 0 - 8.0	5.66	9	50.9400
8.0 - 16 0	11. 32	4	45.2800
16 0 - 32.0	22.63	3	67.8900
Total		100	243.0626

BEDLOAD TRANSPORT COMPUTATIONS

FIGURE 1

$\frac{n_w}{n_m}$ = ——— = 1

$\frac{d}{B}$ = ——— = 0.0066

n_s (from Nomograph) = 0.025

FIGURE 2

$\frac{n_w}{n_s}$ = ——— = 1

$\frac{Q_s}{Q}$ (from Nomograph) = 0.985

$D_m = \frac{\Sigma D \Delta P}{100}$ = 2.4306

D_{90} = 6.3 mm

$D_{90}^{1/6}$ = 1.3590

$\left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} = \left(\frac{1.3590}{0.025}\right)^{3/2} = 54.36^{3/2} = 400.79$

$g_s = 1.606 \left[3.306(0.995)(400.79)(0.46)(0.06) - 0.627(2.4306)\right]^{3/2}$

$g_s = 1.606 \left[36.0218 - 1.5240\right]^{3/2}$

$g_s = 1.606 \left[34.4978\right]^{3/2}$

$g_s = 1.606\ (202.6220)$

g_s = 325.4 tons / day / ft. width

G_s = 325.4 (B) = 22,778.8 tons / day

COMPUTATION OF BEDLOAD TRANSPORT BY MEYER-PETER, MULLER FORMULA FOR ONE DISCHARGE

~~River~~ AIRPORT / GOLF COURSE

Date _____

Time _____

Section SAMPLE A

Computed by JGM

Checked by JGM

Date 7/9/82 ; 7/16/82

Sheet 3 of 12

FORMULA

$$g_s = 1.606 \left[3.306 \left(\frac{Q_s}{Q}\right) \left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} dS - 0.627\ D_m\right]^{3/2} \text{ tons/day/ft. width}$$

CONDITION : DEVELOPED

EVENT : 5YR , 6 HR

HYDRAULIC DATA

Q - Discharge = ~~55.9~~ 56.5 cfs

d - depth = 0.58 ✓ ft

B - width = 70 ft.

S - slope = 0.06 ft./ft.

n_m - total stream "n" = 0.025

n_w - channel sides "n" = 0.025

SIZE ANALYSIS - BED MATERIAL

Size Fraction mm	Geometric Mean D mm	ΔP Percent	DΔP
< .0625		1	—
.0625-.125	.0884	2	0.1768
125-.25	.177	7	1.2390
.25 - .5	.354	13	4.6020
.5 - 1.0	.707	18	12.7260
1.0 - 2.0	1 41	18	25.3800
2 0 - 4 0	2.83	11	31.1300
4 0 - 8.0	5 66	13	73.5800
8.0 - 16 0	11.32	10	113.2000
16 0 - 32.0	22.63	7	158.4100
Total		100	420.4438

BEDLOAD TRANSPORT COMPUTATIONS

FIGURE 1

$\frac{n_w}{n_m}$ = ____ = 1

$\frac{d}{B} = \frac{0.58}{70} = 0.0083$

n_s (from Nomograph) = 0.025

FIGURE 2

$\frac{n_w}{n_s}$ = ____ = 1

$\frac{Q_s}{Q}$ (from Nomograph) = 0.98

$D_m = \frac{\Sigma D \Delta P}{100} = 4.2044$

D_{90} = 11 mm

$D_{90}^{1/6} = 1.4913$

$\left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} = \left(\frac{1.4913}{0.025}\right)^{3/2} = 59.692^{3/2} = 460.72$

$g_s = 1.606 \left[3.306(0.98)(460.72)(0.58)(0.06) - 0.627(4.2044)\right]^{3/2}$

$g_s = 1.606 \left[51.9452 - 2.6362\right]^{3/2}$

$g_s = 1.606 \left[49.3090\right]^{3/2}$

$g_s = 1.606\ (346.2494)$

$g_s = 556.1$ tons / day / ft. width

$G_s = 556.1\ (B) = 38,925.4$ tons / day

COMPUTATION OF BEDLOAD TRANSPORT BY MEYER-PETER, MULLER FORMULA FOR ONE DISCHARGE

~~River~~ AIRPORT / GOLF COURSE
Date
Time
Section SAMPLE B

Computed by JGM
Checked by JGM
Date 7/9/82 ; 7/16/82
Sheet 4 of 12

FORMULA

$$g_s = 1.606 \left[3.306 \left(\frac{Q_s}{Q}\right) \left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} dS - 0.627\ D_m\right]^{3/2} \text{ tons/day/ft. width}$$

CONDITION : DEVELOPED
EVENT : 5 YR , 6 HR

HYDRAULIC DATA

Q - Discharge = ~~55.5~~ 56.5 cfs
d - depth = 0.58 ✓ ft
B - width = 70 ft.
S - slope = 0.06 ft./ft.
n_m - total stream "n" = 0.025
n_w - channel sides "n" = 0.025

SIZE ANALYSIS - BED MATERIAL

Size Fraction mm	Geometric Mean D mm	ΔP Percent	DΔP
< .0625		1	—
.0625 - .125	0884	4	0.3536
125 - .25	.177	9	1.5930
.25 - .5	.354	18	6.3720
.5 - 1.0	.707	22	15.5540
1.0 - 2.0	1 41	21	29.6100
2.0 - 4 0	2 83	9	25.4700
4 0 - 8.0	5.66	9	50.9400
8 0 - 16 0	11. 32	4	45.2800
16.0 - 32.0	22.63	3	67.8900
Total		100	243.0626

BEDLOAD TRANSPORT COMPUTATIONS

FIGURE 1

$\frac{n_w}{n_m}$ = ——— = 1

$\frac{d}{B}$ = ——— = 0.0083

n_s (from Nomograph) = 0.025

$D_m = \frac{\Sigma D \Delta P}{100}$ = 2.4306

D_{90} = 6.3 mm

$D_{90}^{1/6}$ = 1.3590

$\left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} = \left(\frac{1.3590}{0.025}\right)^{3/2} = 54.36^{3/2} = 400.79$

FIGURE 2

$\frac{n_w}{n_s}$ = ——— = 1

$\frac{Q_s}{Q}$ (from Nomograph) = 0.98

$g_s = 1.606 \left[3.306(0.98)(400.79)(0.58)(0.06) - 0.627(2.4306)\right]^{3/2}$

$g_s = 1.606 \left[45.1882 - 1.5240\right]^{3/2}$

$g_s = 1.606 \left[43.6642\right]^{3/2}$

$g_s = 1.606\ (288.5283)$

g_s = 463.4 tons / day / ft. width

G_s = 463.4 (B) = 32,436.4 tons / day

COMPUTATION OF BEDLOAD TRANSPORT BY MEYER-PETER, MULLER FORMULA FOR ONE DISCHARGE

~~River~~ AIRPORT / GOLF COURSE
Date _____
Time _____
Section SAMPLE A

Computed by JGM
Checked by JGM
Date 7/9/82 ; 7/16/82
Sheet 5 of 12

FORMULA

$$g_s = 1.606 \left[3.306 \left(\frac{Q_s}{Q}\right) \left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} dS - 0.627\ D_m\right]^{3/2} \text{ tons/day/ft. width}$$

CONDITION: DEVELOPED
EVENT: 10 YR, 6 HR

HYDRAULIC DATA

Q - Discharge = ~~77.3~~ 78.3 cfs
d - depth = 0.66 ✓ ft
B - width = 70 ft.
S - slope = 0.06 ft./ft.
n_m - total stream "n" = 0.025
n_w - channel sides "n" = 0.025

SIZE ANALYSIS - BED MATERIAL

Size Fraction mm	Geometric Mean D mm	ΔP Percent	DΔP
< .0625		1	—
.0625 - .125	.0884	2	0.1768
125 - .25	.177	7	1.2390
.25 - .5	.354	13	4.6020
.5 - 1.0	.707	18	12.7260
1.0 - 2.0	1.41	18	25.3800
2.0 - 4.0	2.83	11	31.1300
4.0 - 8.0	5.66	13	73.5800
8.0 - 16.0	11.32	10	113.2000
16.0 - 32.0	22.63	7	158.4100
Total		100	420.4438

BEDLOAD TRANSPORT COMPUTATIONS

FIGURE 1

$\frac{n_w}{n_m}$ = ____ = 1

$\frac{d}{B} = \frac{0.66}{70}$ = 0.0094

n_s (from Nomograph) = 0.025

FIGURE 2

$\frac{n_w}{n_s}$ = ____ = 1

$\frac{Q_s}{Q}$ (from Nomograph) = 0.98

$D_m = \frac{\Sigma D \Delta P}{100}$ = 4.2044

D_{90} = 11 mm

$D_{90}^{1/6}$ = 1.4913

$\left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} = \left(\frac{1.4913}{0.025}\right)^{3/2} = 59.692^{3/2} = 460.72$

$g_s = 1606 \left[3.306(0.98)(460.72)(0.66)(0.06) - 0.627(4.2044)\right]^{3/2}$

$g_s = 1.606 \left[59.1100 - 2.6362\right]^{3/2}$

$g_s = 1.606 \left[56.4738\right]^{3/2}$

$g_s = 1.606\ (424.3956)$

g_s = 681.6 tons / day / ft. width

$G_s = 681.6\ (B) = 47,710.6$ tons / day

COMPUTATION OF BEDLOAD TRANSPORT BY MEYER-PETER, MULLER FORMULA FOR ONE DISCHARGE

~~River~~ AIRPORT/GOLF COURSE
Date
Time
Section SAMPLE B

Computed by JGM
Checked by JGM
Date 7/9/82; 7/14/82
Sheet 6e of 12

FORMULA

$$g_s = 1.606 \left[3.306 \left(\frac{Q_s}{Q}\right) \left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} dS - 0.627\ D_m\right]^{3/2} \text{ tons/day/ft. width}$$

CONDITION: DEVELOPED
EVENT: 10-YR, 6 HR

HYDRAULIC DATA

Q - Discharge = ~~77.5~~ 78.3 cfs
d - depth = 0.66 ✓ ft
B - width = 70 ft.
S - slope = 0.06 ft./ft.
n_m - total stream "n" = 0.025
n_w - channel sides "n" = 0.025

SIZE ANALYSIS - BED MATERIAL

Size Fraction mm	Geometric Mean D mm	ΔP Percent	DΔP
< .0625		1	—
.0625 - .125	.0884	4	0.3536
125 - .25	.177	9	1.5930
.25 - .5	.354	18	6.3720
.5 - 1.0	.707	22	15.5540
1.0 - 2.0	1 41	21	29.6100
2.0 - 4 0	2.83	9	25.4700
4 0 - 8.0	5.66	9	50.9400
8 0 - 16 0	11. 32	4	45.2800
16.0 - 32.0	22.63	3	67.8900
Total		100	243.0626

BEDLOAD TRANSPORT COMPUTATIONS

FIGURE 1

$\frac{n_w}{n_m}$ = ——— = 1

$\frac{d}{B}$ = ——— = 0.0094

n_s (from Nomograph) = 0.025

FIGURE 2

$\frac{n_w}{n_s}$ = ——— = 1

$\frac{Q_s}{Q}$ (from Nomograph) = 0.98

$D_m = \frac{\Sigma D \Delta P}{100}$ = 2.4306

D_{90} = 6.3 mm

$D_{90}^{1/6}$ = 1.3590

$\left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} = \left(\frac{1.3590}{0.025}\right)^{3/2} = 54.36^{3/2} = 400.79$

$g_s = 1.606 \left[3.306(0.98)(400.79)(0.66)(0.06) - 0.627(2.4306)\right]^{3/2}$

$g_s = 1.606 \left[51.4211 - 1.5240\right]^{3/2}$

$g_s = 1.606 \left[49.8971\right]^{3/2}$

$g_s = 1.606\ (352.4621)$

g_s = 566.1 tons / day / ft. width

G_s = 566.1 (B) = 39,623.8 tons / day

COMPUTATION OF BEDLOAD TRANSPORT BY MEYER-PETER, MULLER FORMULA FOR ONE DISCHARGE

~~River~~ AIRPORT / GOLF COURSE
Date _____
Time _____
Section SAMPLE A

Computed by JGM
Checked by JGM
Date 7/9/82; 7/14/82
Sheet 7 of 12

FORMULA

$$g_s = 1.606 \left[3.306 \left(\frac{Q_s}{Q}\right) \left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} dS - 0.627\ D_m\right]^{3/2} \text{ tons/day/ft. width}$$

CONDITION: DEVELOPED
EVENT: 25-YR, 6-HR

HYDRAULIC DATA

Q - Discharge = ~~140.9~~ 152.0 cfs
d - depth = ~~0.83~~ 0.85 ft
B - width = 70 ft.
S - slope = 0.06 ft./ft.
n_m - total stream "n" = 0.025
n_w - channel sides "n" = 0.025

SIZE ANALYSIS - BED MATERIAL

Size Fraction mm	Geometric Mean D mm	ΔP Percent	DΔP
< .0625		1	—
.0625 - .125	.0884	2	0.1768
125 - .25	.177	7	1.2390
.25 - .5	.354	13	4.6020
.5 - 1.0	.707	18	12.7260
1.0 - 2.0	1.41	18	25.3800
2.0 - 4.0	2.83	11	31.1300
4.0 - 8.0	5.66	13	73.5800
8.0 - 16.0	11.32	10	113.2000
16.0 - 32.0	22.63	7	158.4100
Total		100	420.4438

BEDLOAD TRANSPORT COMPUTATIONS

FIGURE 1

$\frac{n_w}{n_m} = \frac{\quad}{0.85} \doteq \frac{1}{0.0122}$

$\frac{d}{B} = \frac{\sim\!\!0.83}{70} = \frac{\sim\!\!0.0119}{}$

n_s (from Nomograph) = 0.025

FIGURE 2

$\frac{n_w}{n_s} = \frac{\quad}{} = \frac{1}{}$

$\frac{Q_s}{Q}$ (from Nomograph) = 0.975 ✓

$D_m = \frac{\Sigma D \Delta P}{100} = 4.2044$

D_{90} = 11 mm

$D_{90}^{1/6} = 1.4913$

$\left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} = \left(\frac{1.4913}{0.025}\right)^{3/2} = 59.652^{3/2} = 460.72$

$g_s = 1.606 \left[3.306(0.975)(460.72)(0.83)(0.06) - 0.627(4.2044)\right]^{3/2}$

$g_s = 1.606 \left[73.9561 - 2.63362\right]^{3/2}$

$g_s = 1.606 \left[71.3199\right]^{3/2}$

$g_s = 1.606\ (602.3042)$

$g_s = 967.3$ tons / day / ft. width

$G_s = 967.3\ (B) = 67,711.0$ tons / day

COMPUTATION OF BEDLOAD TRANSPORT BY MEYER-PETER, MULLER FORMULA FOR ONE DISCHARGE

~~River~~ AIRPORT/GOLF COURSE
Date_____
Time_____
Section SAMPLE B

Computed by JGM
Checked by JGM
Date 7/9/82; 7/16/82
Sheet 8 of 12

FORMULA

$$g_s = 1.606 \left[3.306 \left(\frac{Q_s}{Q}\right) \left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} dS - 0.627\ D_m\right]^{3/2} \text{ tons/day/ft. width}$$

CONDITION: DEVELOPED
EVENT: 25-YR, 6 HR

HYDRAULIC DATA

Q - Discharge = ~~140.9~~ 152 cfs
d - depth = ~~0.83~~ 0.85 ft
B - width = 70 ft.
S - slope = 0.06 ft./ft.
n_m - total stream "n" = 0.025
n_w - channel sides "n" = 0.025

SIZE ANALYSIS - BED MATERIAL

Size Fraction mm	Geometric Mean D mm	ΔP Percent	DΔP
< .0625		1	—
.0625 - .125	.0884	4	0.3536
.125 - .25	.177	9	1.5930
.25 - .5	.354	18	6.3720
.5 - 1.0	.707	22	15.5540
1.0 - 2.0	1.41	21	29.6100
2.0 - 4.0	2.83	9	25.4700
4.0 - 8.0	5.66	9	50.9400
8.0 - 16.0	11.32	4	45.2800
16.0 - 32.0	22.63	3	67.8900
Total		100	243.0626

BEDLOAD TRANSPORT COMPUTATIONS

FIGURE 1

$\frac{n_w}{n_m}$ = ____ = 1

$\frac{d}{B}$ = ____ = ~~0.0119~~ 0.0122

n_s (from Nomograph) = 0.025

FIGURE 2

$\frac{n_w}{n_s}$ = ____ = 1

$\frac{Q_s}{Q}$ (from Nomograph) = 0.975 ✓

$D_m = \frac{\Sigma D \Delta P}{100} = 2.4306$

D_{90} = 6.3 mm

$D_{90}^{1/6}$ = 1.3590

$\left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} = \left(\frac{1.3590}{0.025}\right)^{3/2} = 54.36^{3/2} = 400.79$

$g_s = 1.606\ [3.306(0.975)(400.79)(0.83)(0.06) - 0.627(2.4306)]^{3/2}$

$g_s = 1.606\ [64.3359 - 1.5240]^{3/2}$

$g_s = 1.606\ [62.8119]^{3/2}$

$g_s = 1.606\ (497.8097)$

g_s = 799.5 tons / day / ft. width

$G_s = 799.5\ (B) = 55,963.8$ tons / day

COMPUTATION OF BEDLOAD TRANSPORT BY MEYER-PETER, MULLER FORMULA FOR ONE DISCHARGE

~~River~~ AIRPORT / GOLF COURSE
Date _____
Time _____
Section SAMPLE A

Computed by JGM
Checked by JGM
Date 7/9/82 ; 7/16/82
Sheet 2 of 12

FORMULA

$$g_s = 1.606 \left[3.306 \left(\frac{Q_s}{Q}\right) \left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} dS - 0.627\ D_m\right]^{3/2} \text{ tons/day/ft. width}$$

CONDITION: DEVELOPED
EVENT: 50-YR, 6 HR

HYDRAULIC DATA

Q - Discharge = ~~244.0~~ 225.5 cfs
d - depth = ~~LOT~~ 0.99 ft
B - width = 70 ft.
S - slope = 0.06 ft./ft.
n_m - total stream "n" = 0.025
n_w - channel sides "n" = 0.025

SIZE ANALYSIS - BED MATERIAL

Size Fraction mm	Geometric Mean D mm	ΔP Percent	DΔP
< .0625		1	—
.0625 - .125	.0884	2	0.1768
125 - .25	.177	7	1.2390
.25 - .5	.354	13	4.6020
.5 - 1.0	.707	18	12.7260
1.0 - 2 0	1 41	18	25.3800
2 0 - 4 0	2.83	11	31.1300
4.0 - 8 0	5.66	13	73.5800
8.0 - 16 0	11.32	10	113.2000
16 0 - 32.0	22.63	7	158.4100
Total		100	420.4438

BEDLOAD TRANSPORT COMPUTATIONS

FIGURE 1

$\frac{n_w}{n_m} = \frac{\quad}{0.99} = \frac{1}{0.0141}$

$\frac{d}{B} = \frac{\text{LOT}}{70} = 0.0145$

n_s (from Nomograph) = 0.025

$D_m = \frac{\Sigma D \Delta P}{100} = 4.2044$

D_{90} = 11 mm

FIGURE 2

$\frac{n_w}{n_s} = \frac{\quad}{\quad} = 1$

$\frac{Q_s}{Q}$ (from Nomograph) = 0.97 ✓

$D_{90}^{1/6} = 1.4913$

$\left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} = \left(\frac{1.4913}{0.025}\right)^{3/2} = 59.672^{3/2} = 460.72$

$g_s = 1.606 \left[3.306(0.97)(460.72)(1.01)(0.06) - 0.627(4.2044)\right]^{3/2}$

$g_s = 1.606 \left[89.5332 - 2.6362\right]^{3/2}$

$g_s = 1.606 \left[86.8970\right]^{3/2}$

$g_s = 1.606\ (810.0418)$

$g_s = 1300.9$ tons / day / ft. width

$G_s = 1300.9\ (B) = 91,064.9$ tons / day

COMPUTATION OF BEDLOAD TRANSPORT BY MEYER-PETER, MULLER FORMULA FOR ONE DISCHARGE

~~River~~ AIRPORT/GOLF COURSE
Date _____
Time _____
Section SAMPLE B

Computed by JGM
Checked by JGM
Date 7/9/82; 7/14/82
Sheet 10 of 12

FORMULA

$$g_s = 1.606 \left[3.306 \left(\frac{Q_s}{Q}\right) \left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} dS - 0.627\ D_m\right]^{3/2} \text{ tons/day/ft. width}$$

CONDITION : DEVELOPED
EVENT : 50-YR, 6 HR

HYDRAULIC DATA

Q - Discharge = ~~244.0~~ 225.5 cfs
d - depth = LOT 0.99 ft
B - width = 70 ft.
S - slope = 0.06 ft/ft.
n_m - total stream "n" = 0.025
n_w - channel sides "n" = 0.025

SIZE ANALYSIS - BED MATERIAL

Size Fraction mm	Geometric Mean D mm	ΔP Percent	DΔP
< .0625		1	—
.0625 - .125	.0884	4	0.3536
125 - .25	.177	9	1.5930
.25 - .5	.354	18	6.3720
.5 - 1.0	.707	22	15.5540
1.0 - 2.0	1.41	21	29.6100
2.0 - 4.0	2.83	9	25.4700
4.0 - 8.0	5.66	9	50.9400
8.0 - 16.0	11.32	4	45.2800
16.0 - 32.0	22.63	3	67.8900
Total		100	243.0626

BEDLOAD TRANSPORT COMPUTATIONS

FIGURE 1

$\frac{n_w}{n_m}$ = ____ = 1

$\frac{d}{B}$ = ____ = ~~0.0145~~ 0.0141

n_s (from Nomograph) = 0.025

FIGURE 2

$\frac{n_w}{n_s}$ = ____ = 1

$\frac{Q_s}{Q}$ (from Nomograph) = 0.97 ✓

$D_m = \frac{\Sigma D\Delta P}{100} = 2.4306$

D_{90} = 6.3 mm

$D_{90}^{1/6} = 1.3590$

$\left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} = \left(\frac{1.3590}{0.025}\right)^{3/2} = 54.36^{3/2} = 400.79$

$g_s = 1.606 \left[3.306(0.97)(400.79)(1.01)(0.06) - 0.627(2.4306)\right]^{3/2}$

$g_s = 1.606 \left[77.8868 - 1.5240\right]^{3/2}$

$g_s = 1.606 \left[76.3628\right]^{3/2}$

$g_s = 1.606\ (667.3030)$

g_s = 1071.7 tons / day / ft. width

G_s = 1071.7 (B) = 75,018.2 tons / day

COMPUTATION OF BEDLOAD TRANSPORT BY MEYER-PETER, MULLER FORMULA FOR ONE DISCHARGE

~~River~~ AIRPORT / GOLF COURSE

Date _____

Time _____

Section SAMPLE A

Computed by JGM

Checked by JGM

Date 7/9/82; 7/16/82

Sheet 11 of 12

FORMULA

$$g_s = 1.606 \left[3.306 \left(\frac{Q_s}{Q}\right) \left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} dS - 0.627\ D_m\right]^{3/2} \text{ tons/day/ft. width}$$

CONDITION: DEVELOPED

EVENT: 100 YR, 6 HR

HYDRAULIC DATA

Q - Discharge = ~~291.9~~ 238.3 cfs

d - depth = ~~1.08~~ 1.01 ft

B - width = 70 ft.

S - slope = 0.06 ft./ft.

n_m - total stream "n" = 0.025

n_w - channel sides "n" = 0.025

SIZE ANALYSIS - BED MATERIAL

Size Fraction mm	Geometric Mean D mm	ΔP Percent	DΔP
< .0625		1	—
.0625 - .125	.0884	2	0.1768
125 - .25	.177	7	1.2390
.25 - .5	.354	13	4.6020
.5 - 1.0	.707	18	12.7260
1.0 - 2.0	1.41	18	25.3800
2.0 - 4.0	2.83	11	31.1300
4.0 - 8.0	5.66	13	73.5800
8.0 - 16.0	11.32	10	113.2000
16.0 - 32.0	22.63	7	158.4100
Total		100	420.4438

BEDLOAD TRANSPORT COMPUTATIONS

FIGURE 1

$\frac{n_w}{n_m} = \frac{\quad}{1.01} = \frac{1}{0.0144}$

$\frac{d}{B} = \frac{\sim\sim 1.08 \sim\sim}{70} = \sim\sim 0.0155 \sim\sim$

n_s (from Nomograph) = 0.025

FIGURE 2

$\frac{n_w}{n_s} = \frac{\quad}{\quad} = 1$

$\frac{Q_s}{Q}$ (from Nomograph) = 0.97 ✓

$D_m = \frac{\Sigma D \Delta P}{100} = 4.2044$

$D_{90} = 11$ mm

$D_{90}^{1/6} = 1.4913$

$\left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} = \left(\frac{1.4913}{0.025}\right)^{3/2} = 59.652^{3/2} = 460.72$

$g_s = 1.606 \left[3.306 (0.97)(460.72)(1.08)(0.06) - 0.627(4.2044)\right]^{3/2}$

$g_s = 1.606 \left[95.7385 - 2.6362\right]^{3/2}$

$g_s = 1.606 \left[93.1023\right]^{3/2}$

$g_s = 1.606 (898.3399)$

$g_s = 1442.7$ tons / day / ft. width

$G_s = 1442.7 (B) = 100,991.4$ tons / day

COMPUTATION OF BEDLOAD TRANSPORT BY MEYER-PETER, MULLER FORMULA FOR ONE DISCHARGE

~~River~~ AIRPORT/GOLF COURSE
Date _____
Time _____
Section SAMPLE B

Computed by JGM
Checked by JGM
Date 7/9/82; 7/16/82
Sheet 12 of 12

FORMULA

$$g_s = 1.606 \left[3.306 \left(\frac{Q_s}{Q}\right) \left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} dS - 0.627\ D_m\right]^{3/2} \text{ tons/day/ft. width}$$

CONDITION: DEVELOPED
EVENT: 100 YR, 6 HR

HYDRAULIC DATA

Q - Discharge = ~~291.9~~ 238.3 cfs
d - depth = ~~1.08~~ 1.01 ft
B - width = 70 ft.
S - slope = 0.06 ft/ft.
n_m - total stream "n" = 0.025
n_w - channel sides "n" = 0.025

SIZE ANALYSIS - BED MATERIAL

Size Fraction mm	Geometric Mean D mm	ΔP Percent	DΔP
< .0625		1	—
.0625 - .125	.0884	4	0.3536
125 - .25	.177	9	1.5930
.25 - .5	.354	18	6.3720
.5 - 1.0	.707	22	15.5540
1.0 - 2.0	1.41	21	29.6100
2.0 - 4.0	2.83	9	25.4700
4.0 - 8.0	5.66	9	50.9400
8.0 - 16.0	11.32	4	45.2800
16.0 - 32.0	22.63	3	67.8900
Total		100	243.0626

BEDLOAD TRANSPORT COMPUTATIONS

FIGURE 1

$\frac{n_w}{n_m}$ = ____ = 1

$\frac{d}{B}$ = ____ = ~~0.0155~~ 0.0144

n_s (from Nomograph) = 0.025

FIGURE 2

$\frac{n_w}{n_s}$ = ____ = 1

$\frac{Q_s}{Q}$ (from Nomograph) = 0.97 ✓

$D_m = \frac{\Sigma D \Delta P}{100} = 2.4306$

D_{90} = 6.3 mm

$D_{90}^{1/6}$ = 1.3590

$\left(\frac{D_{90}^{1/6}}{n_s}\right)^{3/2} = \left(\frac{1.3590}{0.025}\right)^{3/2} = 54.36^{3/2} = 400.79$

$g_s = 1.606 \left[3.306(0.97)(400.79)(1.08)(0.06) - 0.627(2.4306)\right]^{3/2}$

$g_s = 1.606 \left[83.2849 - 1.5240\right]^{3/2}$

$g_s = 1.606 \left[81.7609\right]^{3/2}$

$g_s = 1.606 (739.2962)$

g_s = 1187.3 tons / day / ft. width

G_s = 1187.3 (B) = 83,111.7 tons / day

DEVELOPED CONDITION : AVG. SEDIMENT YIELD PER EVENT

2-YR :

✓A 27,140.6 (3)(0.54)($\frac{1}{24}$) = 1832.0 TONS

B 22,778.8 (3)(0.54)($\frac{1}{24}$) = 1537.6 TONS

AVG = 1684.8 TONS

5-YR :

✓A 38,925.4 (3)(0.54)($\frac{1}{24}$) = 2627.5 TONS

B 32,436.4 (3)(0.54)($\frac{1}{24}$) = 2189.5 TONS

AVG = 2408.5 TONS

✓10-YR :

A 47,710.6 (3)(0.54)($\frac{1}{24}$) = 3220.5 TONS

B 39,623.8 (3)(0.54)($\frac{1}{24}$) = 2674.6 TONS

AVG = 2947.6 TONS

✓25-YR :

A 67,711.0 (3)(0.42)($\frac{1}{24}$) = 3554.8 TONS

B 55,963.8 (3)(0.42)($\frac{1}{24}$) = 2938.1 TONS

AVG = 3246.5 TONS

50-YR :

A $91,064.9(3)(\frac{0.48}{0.34})(\frac{1}{24}) = \begin{matrix}5463.9\\3870.3\end{matrix}$ TONS

B $75,018.2(3)(\frac{0.48}{0.34})(\frac{1}{24}) = \begin{matrix}4501.1\\3188.5\end{matrix}$ TONS

AVG = $\begin{matrix}4982.5\\3529.3\end{matrix}$ TONS ✓

100-YR :

A $100,991.4(3)(\frac{0.50}{0.32})(\frac{1}{24}) = \begin{matrix}6312.0\\4039.7\end{matrix}$ TONS

B $83,111.7(3)(\frac{0.50}{0.32})(\frac{1}{24}) = \begin{matrix}5194.5\\3324.5\end{matrix}$ TONS

AVG = $\begin{matrix}5753.2\\3682.1\end{matrix}$ TONS ✓

APPENDIX E:

ANNUAL SEDIMENT YIELD CALCULATIONS

EXISTING CONDITION

Average Annual Runoff:

CN = 78 ∴ runoff threshold = 0.7 inches (17, 18)

By extrapolation from Table 9, %R-O = 7.2%

Average annual rainfall = 8.31 inches (11)

$$V_a = \frac{8.31}{12}(0.072)(0.342)(\frac{640 \text{ ac}}{\text{sq. mi.}}) = 10.913 \text{ ac-ft/yr}$$

Average Annual Sediment Yield:

$$SY_a = 2695.7(\frac{10.913}{1.443}) = 20{,}386.8 \text{ tons/yr}$$

where SY_m and V_m are determined by m = 2-yr event (14).

Values obtained from Table 3.

DEVELOPED CONDITION

Average Annual Runoff:

CN = 81 ∴ runoff threshold = 0.6 inches (17, 18)

By extrapolation from Table 9, %R-O = 11%

Average annual rainfall = 3.31 inches (11)

$$V_a = \frac{8.31}{12}(0.11)(0.398)(\frac{640 \text{ ac}}{\text{sq. mi.}}) = 19.403 \text{ ac-ft/yr}$$

Average Annual Sediment Yield:

$$SY_a = 2998.9(\frac{19.403}{2.844}) = 20{,}459.8 \text{ tons/yr}$$

where SY_m and V_m are determined by m = 2-yr event (14).

Values obtained from Table 5.

OBSERVED SEDIMENT YIELD

1. Upstream diked area ≈ 1.2 ac (determined from aerial mapping)

   Depth of sediment ≈ 6 feet (4)

   ∴ Volume = $6(1.2)(43,560)(\frac{1}{27})$ = 11,616 CY

2. Downstream diked area ≈ 1.37 ac (determined from aerial mapping)

   Depth of sediment ≈ 4 feet (4)

   ∴ Volume = $4(1.37)(43,560)(\frac{1}{27})$ = 8,841 CY

TIME PERIOD	YEARS OF RECORD	EVENTS EXCEEDING					
		0.10"	0.25"	0.50"	0.75"	1.0"	1.5"
1891 - 1910	20	414	192	67	29	16	3
1911 - 1930	20	504	233	70	19	11	4
1931 - 1940	10	226*	225	33*	12*	7	2*
1951 - 1970	20	443	260*	62	24*	10	3*

*Extrapolated data

$$\% \text{ Runoff @ } 0.75" = \frac{1}{70}\left[\frac{29}{414}(20)+\frac{19}{504}(20)+\frac{12}{226}(10)+\frac{24}{443}(20)\right] = 5.4\%$$

$$\% \text{ Runoff @ } 0.5" = \frac{1}{70}\left[\frac{67}{414}(20)+\frac{70}{504}(20)+\frac{33}{226}(10)+\frac{62}{443}(20)\right] = 14.7\%$$

TABLE 9: RAINFALL RECORD FROM ALBUQUERQUE RAIN GAGE DATA (22)

3. Total sediment yield for the last two years (4) = 11,616+8,841

= 20,457 CY

This represents an average of 10,230 CY/YR.

Using a specific gravity = 2.65, yields a unit weight = 165 PCF.
[2.65(62.4 PCF) = 165 PCF]

This converts to 2.23 tons/CY or 0.45 CY/ton.
[165 PCF(27 CF/CY)(1-ton/2,000#) = 2.23 tons/CY]

The observed annual sediment yield = 10,230 CY/YR (2.23 tons/CY)
= 22,800 tons/YR

APPENDIX F: HYDROGRAPHS

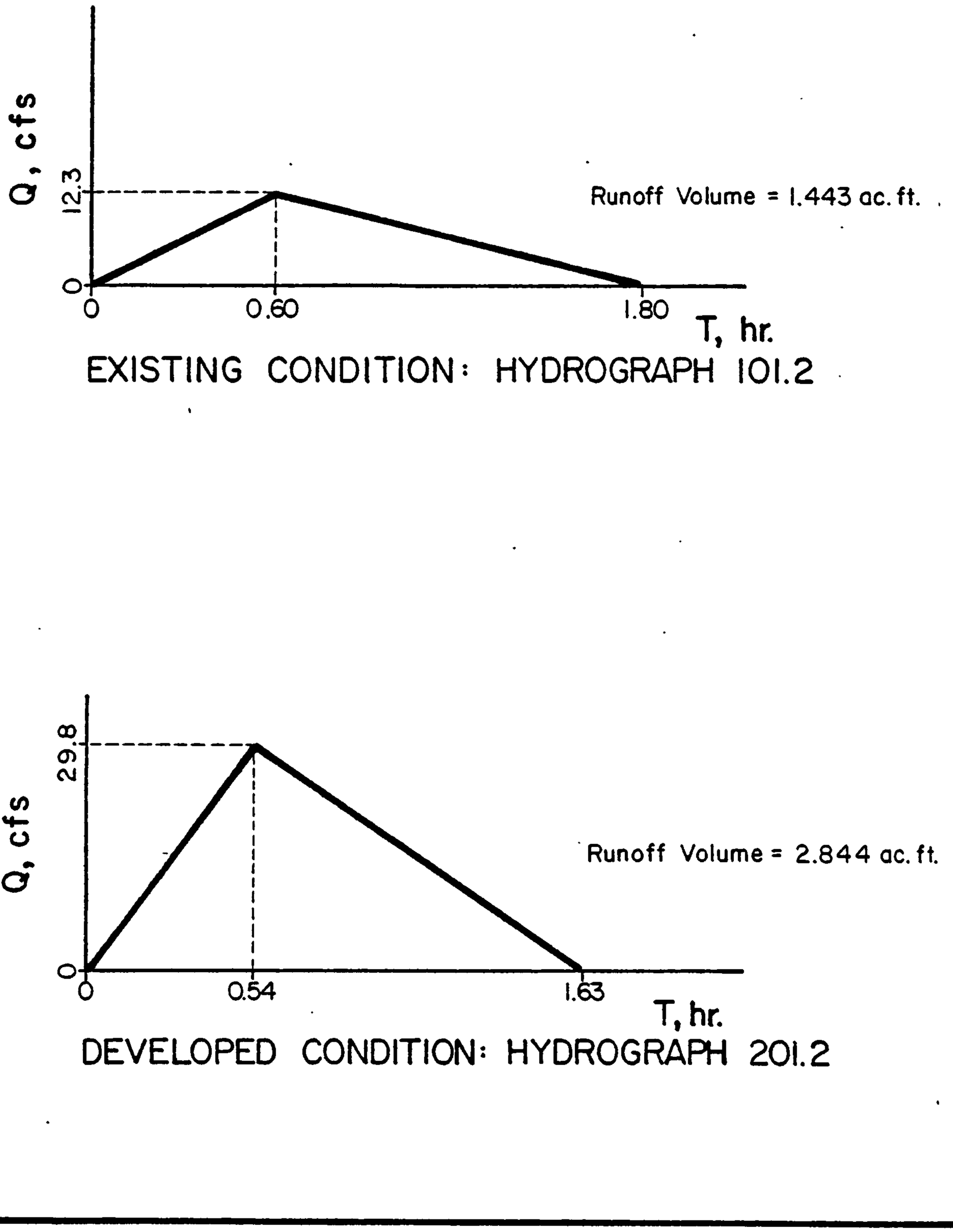
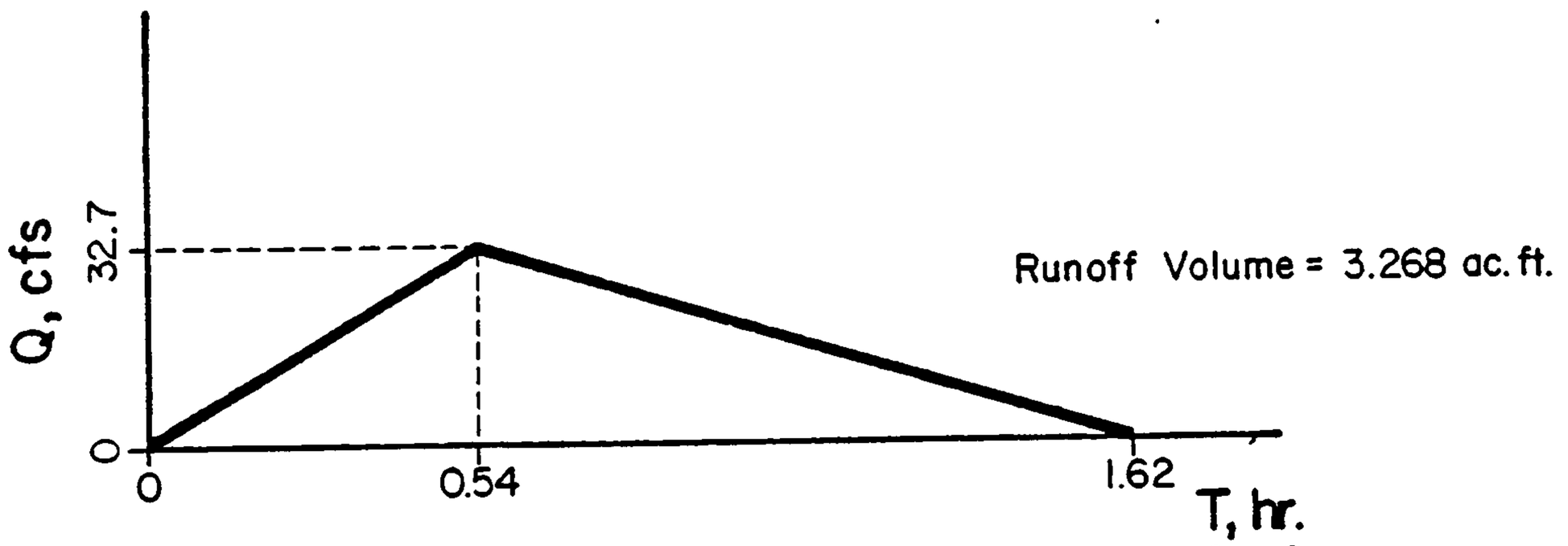
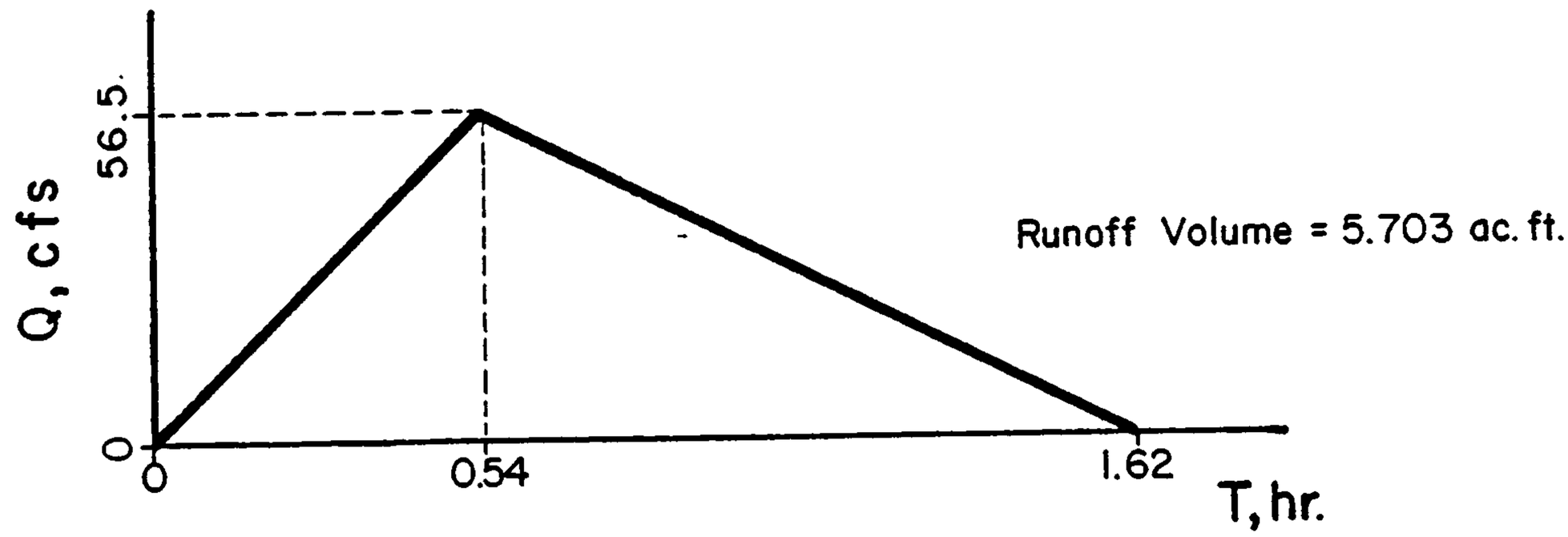


FIGURE 12: 2-YEAR, 6-HOUR HYDROGRAPHS FOR ENTIRE STUDY AREA

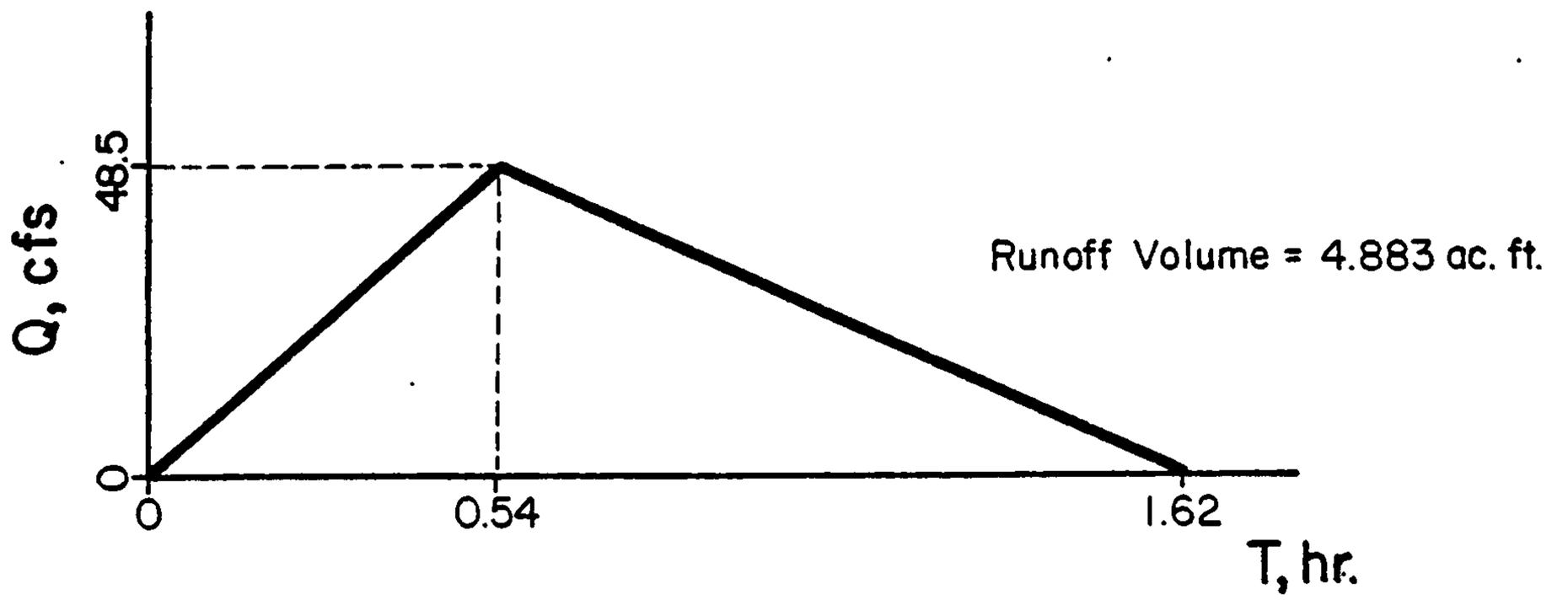


EXISTING CONDITION: HYDROGRAPH 101.2

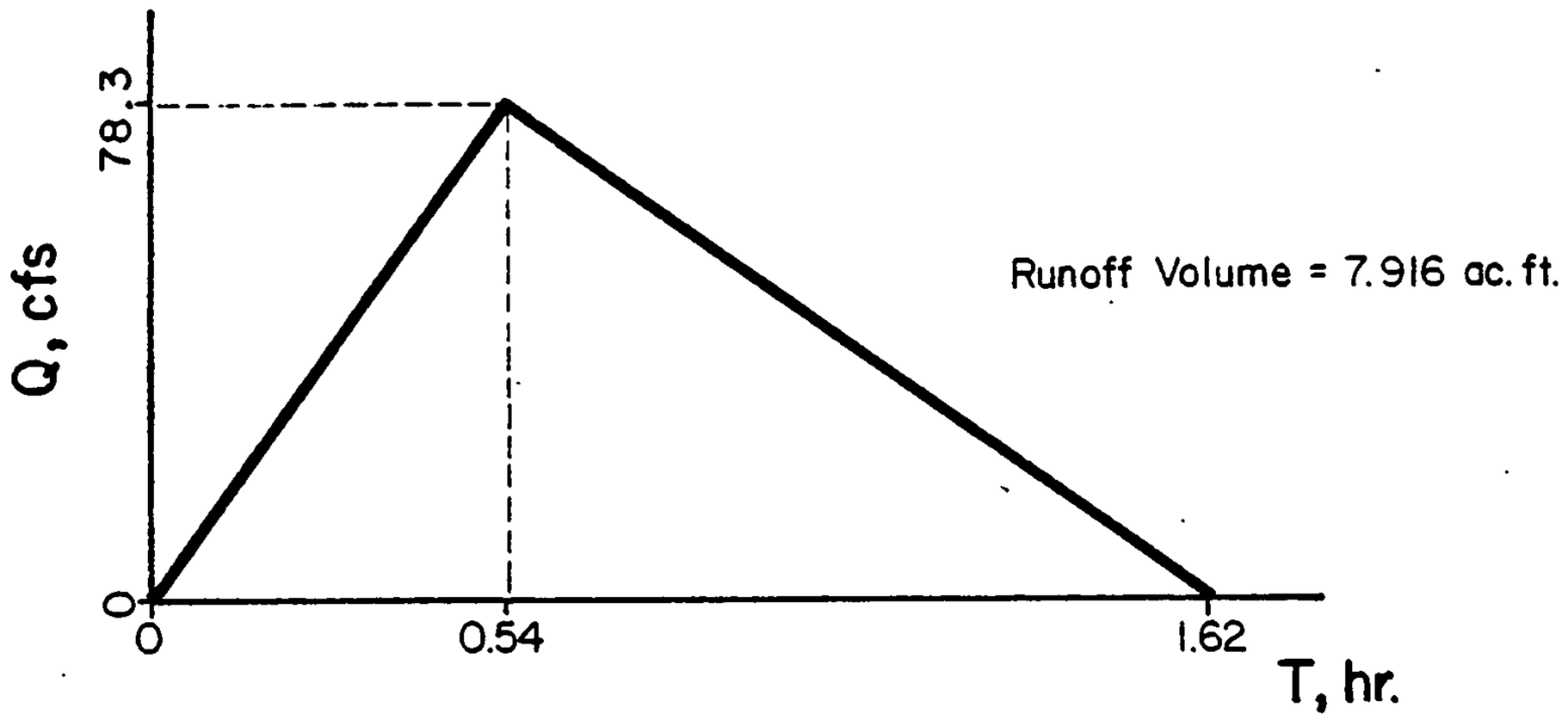


DEVELOPED CONDITION: HYDROGRAPH 201.2

FIGURE 13: 5-YEAR, 6-HOUR HYDROGRAPHS FOR ENTIRE STUDY AREA

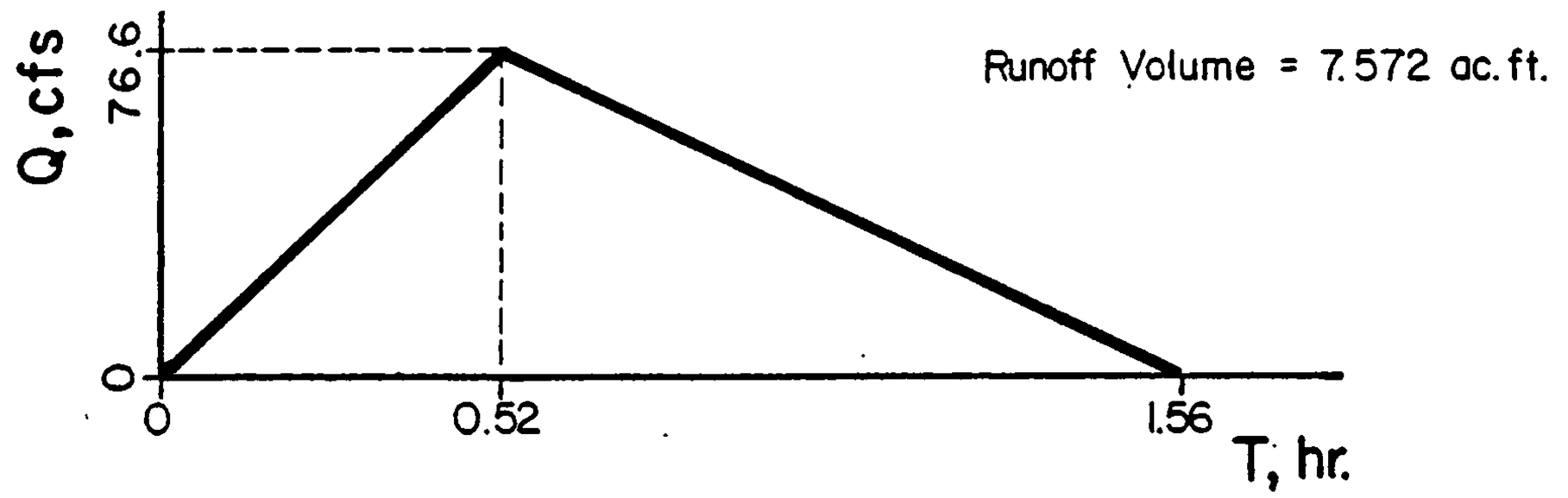


EXISTING CONDITION: HYDROGRAPH 101.2

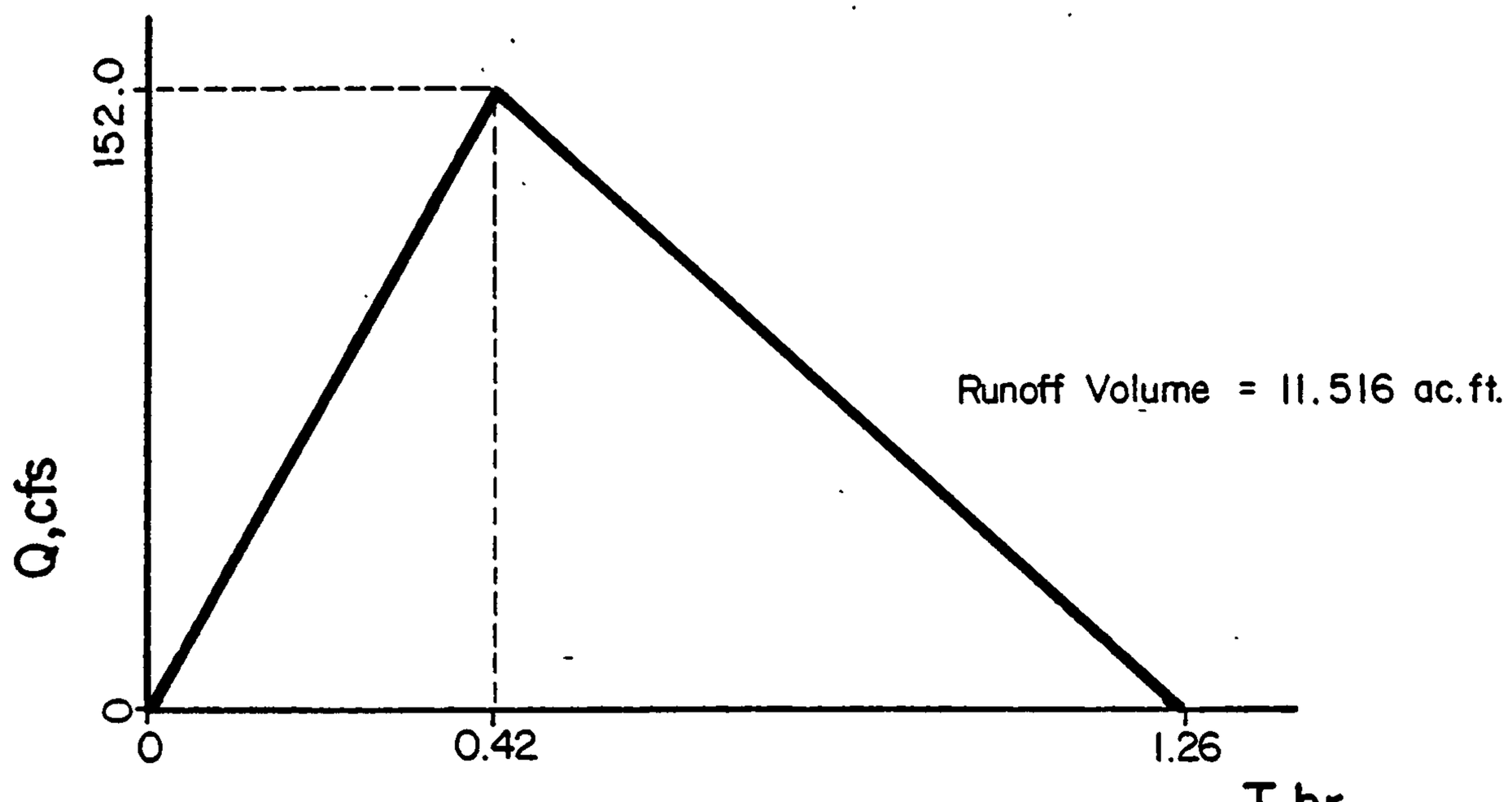


DEVELOPED CONDITION: HYDROGRAPH 201.2

FIGURE 14: 10-YEAR, 6-HOUR HYDROGRAPHS FOR ENTIRE STUDY AREA



EXISTING CONDITION : HYDROGRAPH 101.2



DEVELOPED CONDITION : HYDROGRAPH 201.2

FIGURE 15: 25-YEAR, 6-HOUR HYDROGRAPHS FOR ENTIRE STUDY AREA

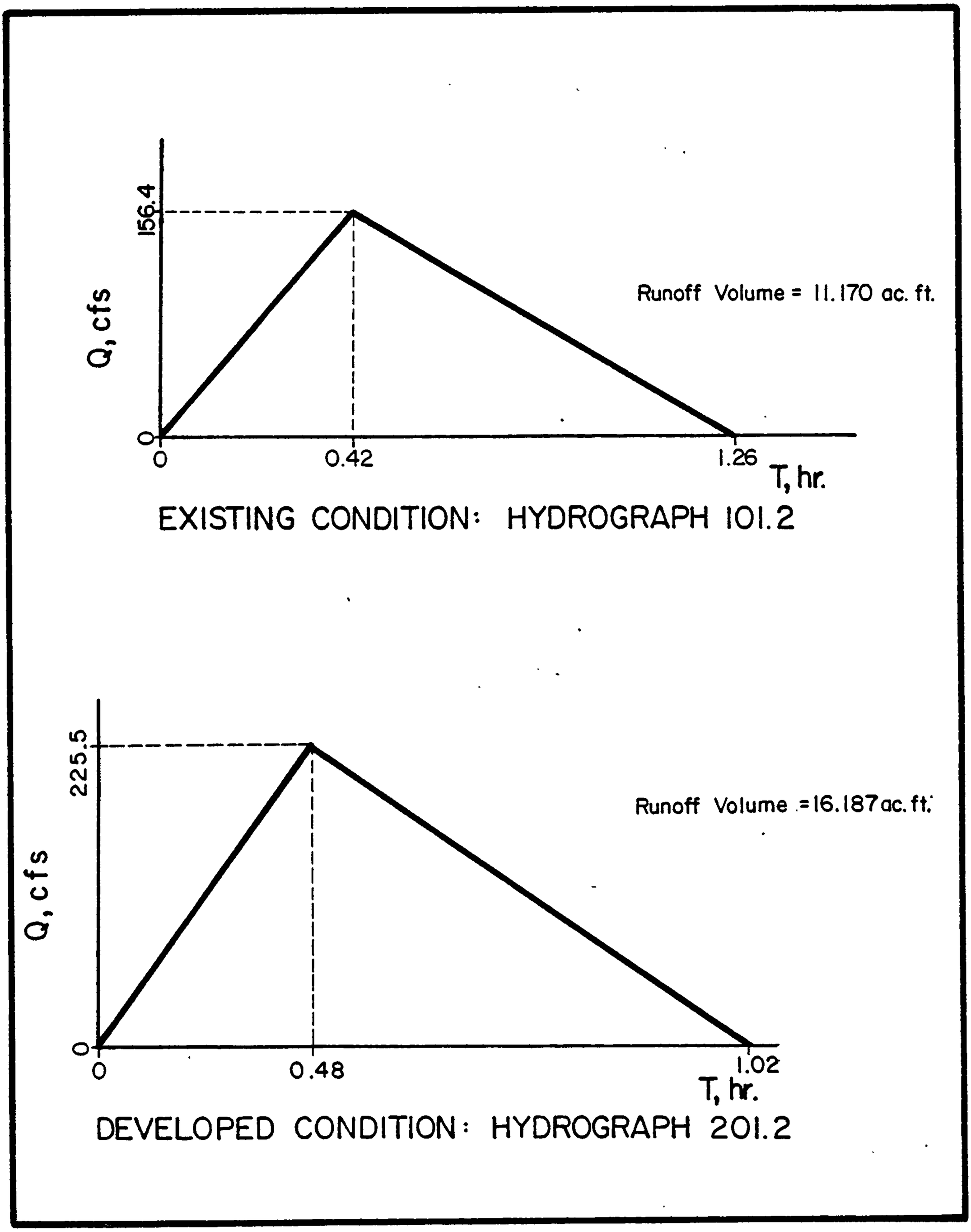
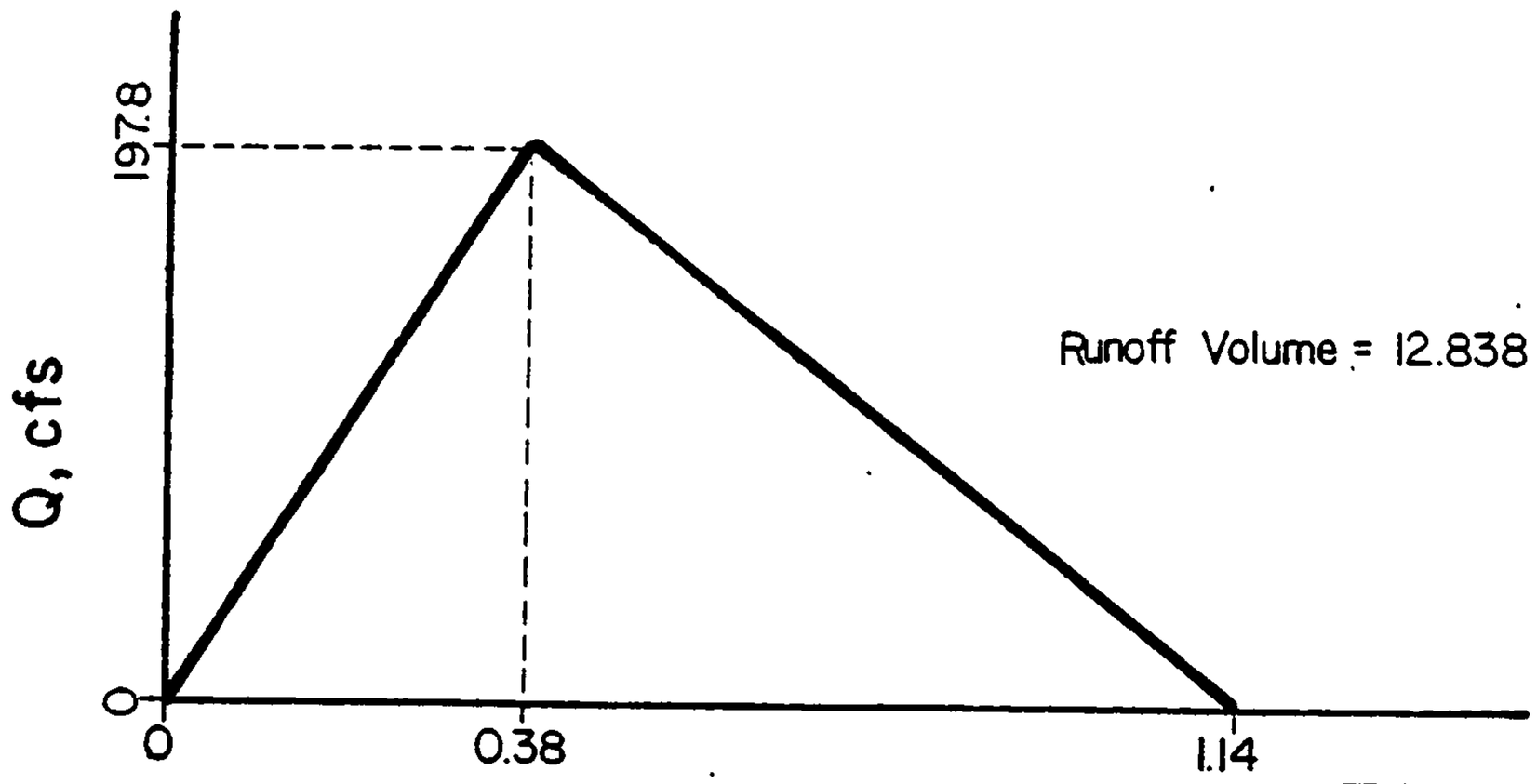
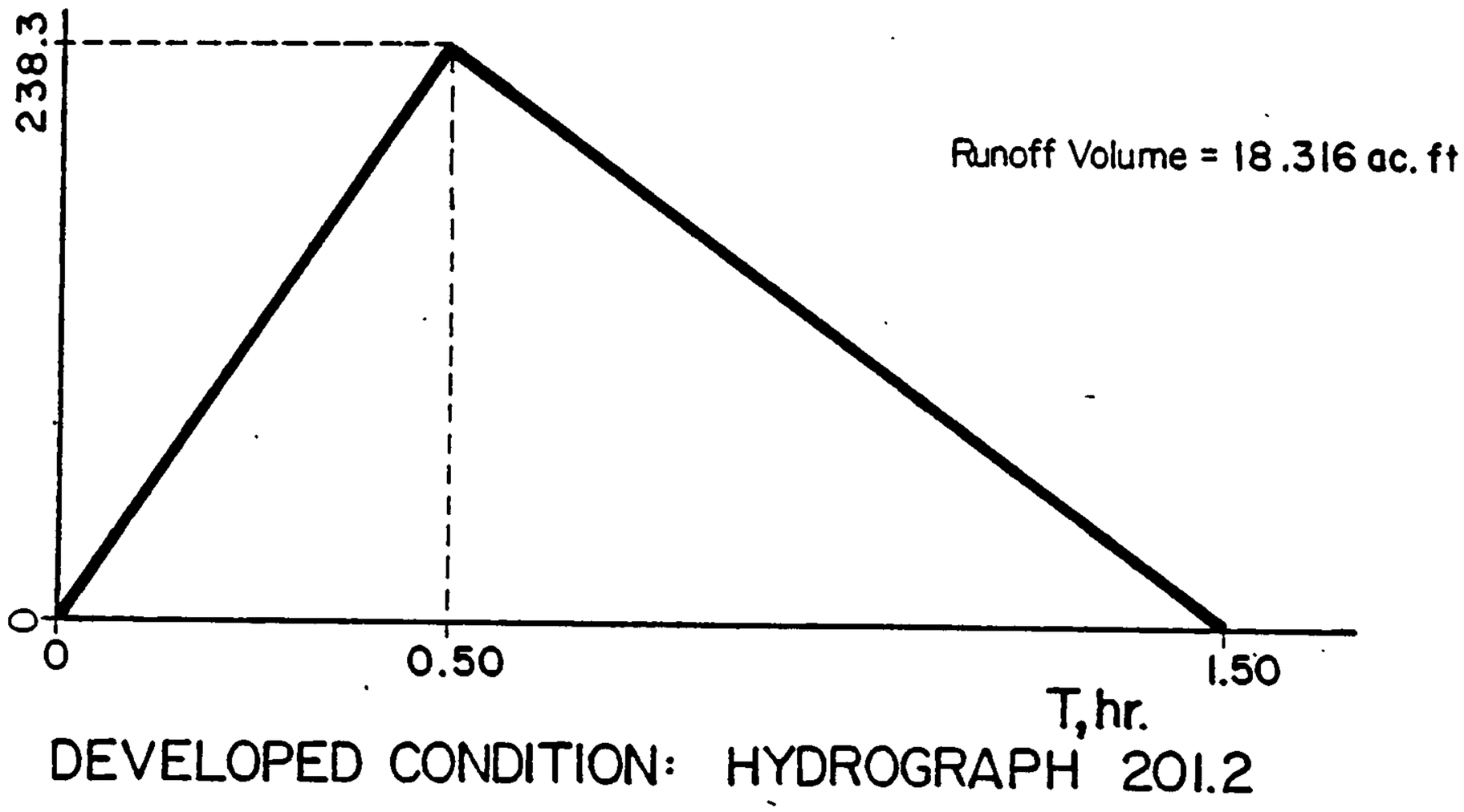


FIGURE 16: 50-YEAR, 6-HOUR HYDROGRAPHS FOR ENTIRE STUDY AREA



EXISTING CONDITION: HYDROGRAPH 101.2



DEVELOPED CONDITION: HYDROGRAPH 201.2

FIGURE 17: 100-YEAR, 6-HOUR HYDROGRAPHS FOR ENTIRE STUDY AREA

APPENDIX G: RAINFALL DATA

Storm No.	Date	Accumulated Rainfall, In.	Comments	Storm No.	Date	Accumulated Rainfall, In.	Comments
1	3-11-65	0.34					
2	6-9-65	0.30		36	6-14-74	0.37	
3	8-1-65	0.25		37	6-31-74	0.58	
4	9-2-65	0.41		38	6-29-76	0.57	
5	10-16-65	0.27		39	8-17-77	0.56	
6	6-27-66	0.75		40	10-6-77	0.73	
7	6-15-66	0.47		41	8-3-77	0.40	
8	6-16-66	0.64		42	8-9-78	0.55	
9	8-18-66	0.34		43	5-25-79	0.53	
10	9-23-66	0.49		44	8-9-79	0.77	
11	10-11-66	0.54		45	8-15-79	0.52	
12	2-26-67	0.29		46	11-7-79	0.32	
13	6-22-67	0.37		47	11-8-79	0.50	
14	12-20-67	0.27		48	12-27-79	0.59	
15	3-10-68	0.33		49	1-11-80	0.39	
16	5-11-68	0.43		50	3-27-80	0.25	
17	7-3-68	0.88		51	4-24-80	0.59	First Dike Washout?
18	8-10-68	0.50		52	8-2-80	0.48	
19	4-11-69	0.72		53	8-14-80	1.75	Second Dike Washout?
20	5-5-69	0.54		54	9-6-80	0.28	
21	6-15-69	0.54		55	9-8-80	0.44	
22	7-16-69	0.50		56	9-9-80	0.62	
23	6-6-69	0.53		57	9-11-80	0.35	
24	8-7-69	0.76		58	12-8-80	0.74	
25	10-22-69	0.56		59	2-9-81	0.51	
26	7-28-70	0.36		60	3-3-81	0.32	
27	8-8-70	0.98		61	7-1-81	0.54	
28	9-13-70	0.44		62	8-5-81	0.27	
29	12-14-70	0.29		63	8-11-81	0.30	
30	8-17-71	0.34		64	8-17-81	0.64	
31	7-4-72	0.42		65	9-5-81	0.38	
32	7-21-72	0.44		66	10-2-81	1.13	Third Dike Washout?
33	8-18-72	0.95		67	3-13-82	0.37	
34	8-29-72	0.73		68	3-22-82	0.39	
35	6-13-73	0.99		69	7-22-82	0.30	

*Data obtained from National Weather Service (12), and Brian G. Burnett (2) for rain gage at Albuquerque International Airport.

TABLE 10: RAINFALL DATA - 1965 TO JULY 1982

1909.11

DESIGN ANALYSIS REPORT

ALBUQUERQUE INTERNATIONAL AIRPORT GENERAL AVIATION AREA, PART 4

PREPARED FOR
CITY OF ALBUQUERQUE, NEW MEXICO

July 1983

LEEDSHILL-HERKENHOFF, INC. COPPER SQUARE P.O. BOX 1217 ALBUQUERQUE, NEW MEXICO 87103

1909.11

DESIGN ANALYSIS REPORT

ALBUQUERQUE INTERNATIONAL AIRPORT GENERAL AVIATION AREA, PART 4

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LEEDSHILL-HERKENHOFF, INC. COPPER SQUARE P.O. BOX 1217 ALBUQUERQUE, NEW MEXICO 87103

LEEDSHILL · HERKENHOFF, INC.
Copper Square
P. O. Box 1217
Albuquerque, New Mexico 87103
(505) 247-0294

1134-1909.11-83

July 11, 1983

Mr. R. L. Pratt, A.A.E.
Director of Aviation
City of Albuquerque
P.O. Box 9022
Albuquerque, N.M. 87119

RE: SOUTH GENERAL AVIATION AREA 'B', PART 4;
DESIGN ANALYSIS REPORT

This final edition of the above referenced report contains the data developed during the design analysis and review phases. Both addenda are also contained in this volume.

Based upon the information contained in this report, construction plans, specifications and bidding documents are being prepared.

CLINT DODGE

jm

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CHAPTER I
PROJECT DEFINITION

A. General:

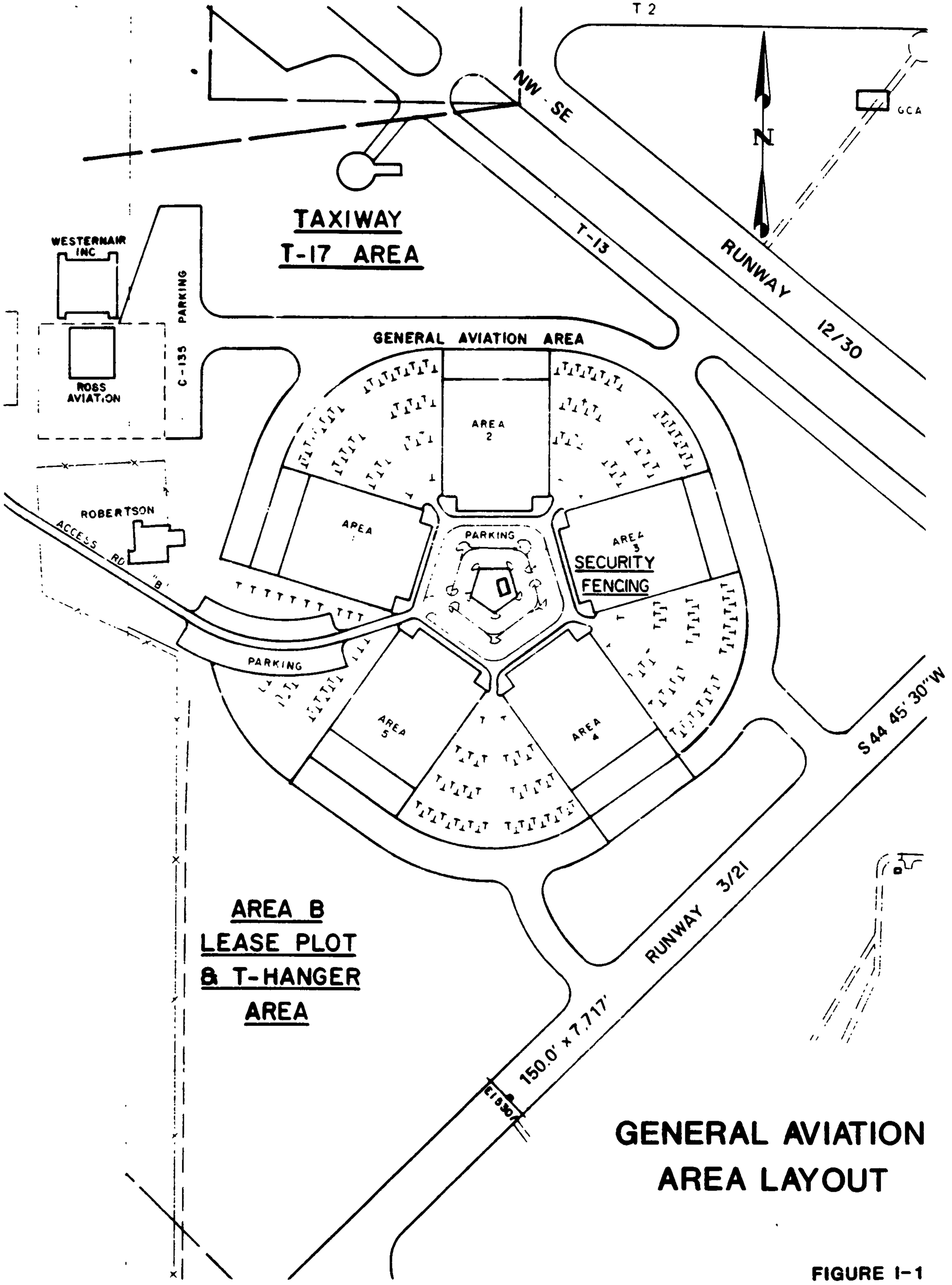
This report is intended to document the design factors considered in the development of this project. The report consists of three general major sections; the first being a project description and consideration of alternates, the selected alternates are more fully analyzed in the second section and supportive calculations, tests, etc. are contained in the appendix. The report is intended to be detailed enough to provide the City with sufficient data so that final design of construction plans can proceed immediately following report approval.

B. Project Description:

The project location and adjacent existing facilities are shown on Figure I-1. The project consists of the following elements:

1. Connecting unlit taxiway from T-13 to the northern edge of the South General Aviation Area. This taxiway is intended to serve two major functions; provide General Aviation (GA) aircraft direct access to T-2 (parallel to runway 8/26) and to provide access by occasional DC-9 type aircraft to the Western Air facility for maintenance work. The location as established by the City in conjunction with Air Traffic Control Tower (ATCT) preferences is from the T-2/T-13 intersection south to the GA area. This taxiway will be identified as T-17.

2. Taxilanes and ramp area for 'T' hangars and lease plots south of the GA area west of runway 3/21. The only lessor presently interested in aviation plots in this area is Robertson Aircraft. Consequently, a reconfiguration of the 100,000 SF lease plot originally described in the lease bid to optimize access, utilities, future expansion and costs is mutually agreeable. The alternates which are identified include the consideration that runway



GENERAL AVIATION AREA LAYOUT

FIGURE I-1

3/21 may be lengthened and upgraded for air carrier use. The entire area south of the GA area, west of the runway 3/21 building restriction line and east of the present fence line is available for potential use, as directed by the City. The tiedown area originally proposed for a portion of this area will be partially superceded by the improvements of this project. The 'T' hangars to be re-located here are those presently situated west of the FAA buildings.

3. Access and utilities are to be extended to this 'T' hangar and lease plot area. An access road as well as sanitary sewer, gas and electrical will be extended from the western edge of the South GA area. Water will be extended from the distribution main in the road along the western property line. Storm drainage will be provided by the existing storm drain system insofar as is feasible.

4. Security fencing will be provided for access control in the G.A. area as part of the relocation of the gate now presently located on the western access road.

C. Design Criteria:

1. Taxiway T-17

a)	Design Aircraft	DC-9
b)	Gross Weight	90,000 #
c)	Frequency	500 operations/year ±
d)	Taxi Speed	10 mph or less
e)	Soil CBR	8 @ 85%, 28 @ 90%, 42 @ 95%
f)	Lighting	None (Retro reflective)

2. G.A. Taxilanes/Ramp

a)	Design Aircraft	G.A. Utility
b)	Gross Weight	12,500 lb.
c)	'T' Hangar Sizes	140' x 50' ± 310' x 50' ±
d)	Soil CBR	1 @ 85%, 20 @ 90%, 23 @ 95%
e)	Lease Area	100,000 SF
f)	Anticipated Lease Hangar	120' x 160'
g)	Lighting	None
h)	Runway Clearance	Air Carrier

3. Access/Utilities

a)	Access Road Category	COA Collector
b)	Traffic Lanes	2
c)	Water Fire Flow Requirement	2000 gpm (8" min.)

4. Security Fencing

a)	Height/Type	5' Chain Link
b)	Anticlimb barbed wire	Yes

CHAPTER II

LAYOUT ALTERNATIVES

The layouts are based on typical dimensional criteria for air carrier runways and for GA T-Hangar areas. Based on recommended FAA standards unless otherwise noted, these criteria are:

Separation-Runway ℄ to Taxiway ℄ (Adequate for all AC fleet)	400 ft.
Building Restriction Line from Runway ℄	750 ft.
Taxiway ℄ to taxilane ℄	325 ft. typ.
T-17 Width (DC-9 class A/C)	50 ft.
T-17 Shoulder width (20' recommended by FAA)	10 ft. min.
T-17 ℄ Radius	100 ft.
T-Hangar separation	70 ft.
G.A. Taxilane (40 ft. recommended by FAA additional width since next to lease boundary)	50 ft. typ.

A. Leased Plot/'T' Hangar Area

The lease plot layout was originally laid out to honor the set aside area for tiedowns south of the main GA area. At the direction of the City, modifications to the original plot boundaries to optimize land use were investigated. Several different layouts were developed using the design criteria. These layouts are presented in schematic form on Figures III-1 through III-5. Elements which were considered in the evaluation included cost, traffic flow, compatability with possible westward expansion, availability of future lease plots, hangar orientation (to minimize winter snow/ice problems), and security.

A brief description of each of the layouts follows:

Plan A: Plan A attempts to maximize the accessibility and leave as much leasable property open as feasible. The facilities are oriented parallel to runway 3/21 which allows compatable future development. Utility extensions are minimized.

Plan B: Plan B is a variation of Plan A with 'T' hangars oriented perpendicular to the runway instead of parallel. There are no significant advantages over Plan A with the disadvantage of winter snow and ice accumulation on the north side of the hangars.

Plan C: Plan C has the lease plot area oriented with the long lease frontage facing the G.A. apron area but situated to avoid the storm drain. Disadvantages include the poor use of potential leasable areas adjacent to the G.A. area and drainage considerations in the infield area.

Plan D: Plan D is similar to Plan C except that the lease plot is located further south in the same locale as the original lease plot layout. The lease plot shown is configured to allow optimum siting of the proposed hangar.

Plan E: Plan E utilizes the original lease configuration. A true north-south orientation for the 'T' hangars is shown (optimum). However, this orientation does not lend itself to future westward expansion which will most probably should be oriented parallel to the runway.

A summary of the parameters for each alternate plan and of the estimated cost is presented on the following table. The recommended alternate is Plan "A" which minimizes cost and maximizes accessability. Plan "A" also represents the best layout from considerations of future airport development.

B. Taxiway T-17

Taxiway T-17 alternates are limited in scope due to the desired connecting point onto T-2/T-13. Three alternates were defined as illustrated on Figures II-6, 7 and 8.

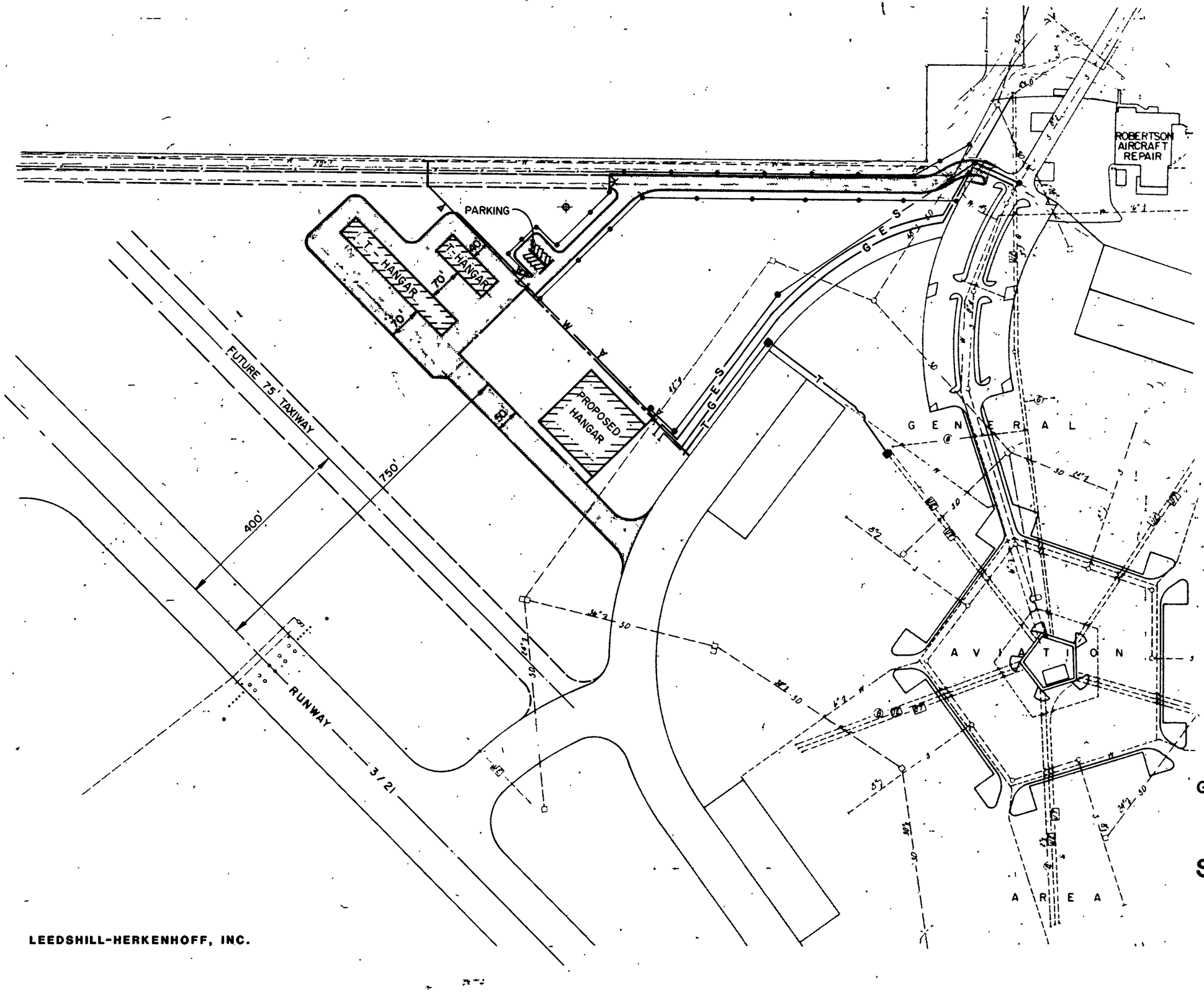
"Plan A" provides the best exit from T-13 onto T-17 for traffic from both directions since it is at right angles. The connection onto the GA ramp taxilane provides good access for all three traffic directions (i.e., east, south and west).

Plan "B" is the minimum cost alternate since it is the shortest distance. However, the connection onto T-13 for traffic east on T-13 is not as suitable, but would be satisfactory. The connection to the G.A. ramp is oriented directly towards a leased hangar site and is not as satisfactory from this viewpoint as Plan "A".

Plan "C" provides ready and direct access to the Western Air/Ross area but does not provide good G.A. traffic flows, a primary purpose of T-17.

A fourth alternate, which is not illustrated is a taxiway connecting onto the south side of the holding bay and thence running south to connect onto the taxilane area in front of Western Air. Ground control (ATCT) recommended the connection at the T-2 intersection to facilitate ground movements and increase the use of T-2.

Plan "A" is recommended as best meeting the design intent of this taxiway. The taxiway will not be lit, but will be provided with retroflective edge markers. Ducts for taxiway lights and future use will be provided.



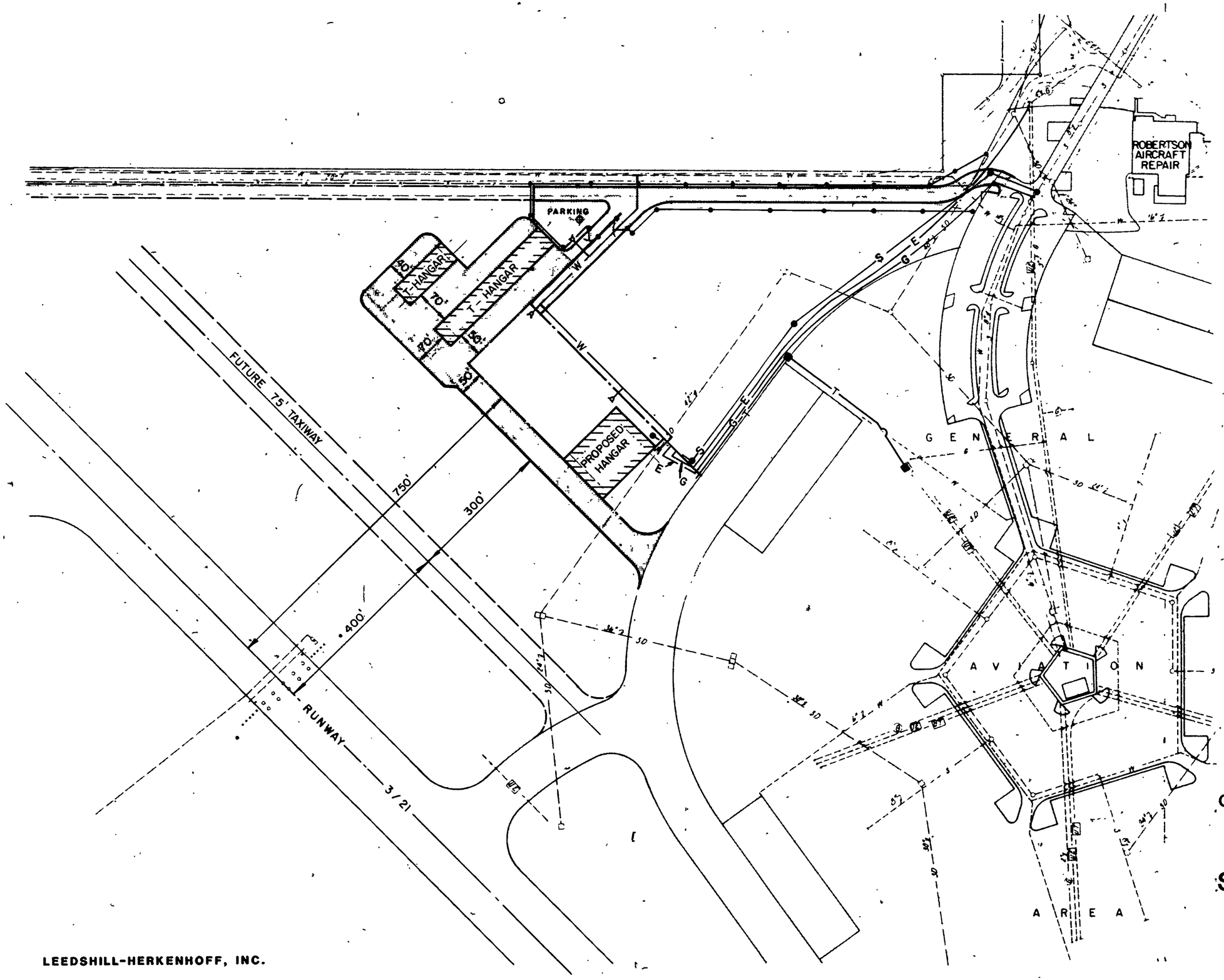
ALBUQUERQUE INTERNATIONAL AIRPORT SOUTH

GENERAL AVIATION AREA, PART 4

AREA B SCHEMATIC LAYOUT PLAN "A"

LEEDSHILL-HERKENHOFF, INC.

FIGURE II-1



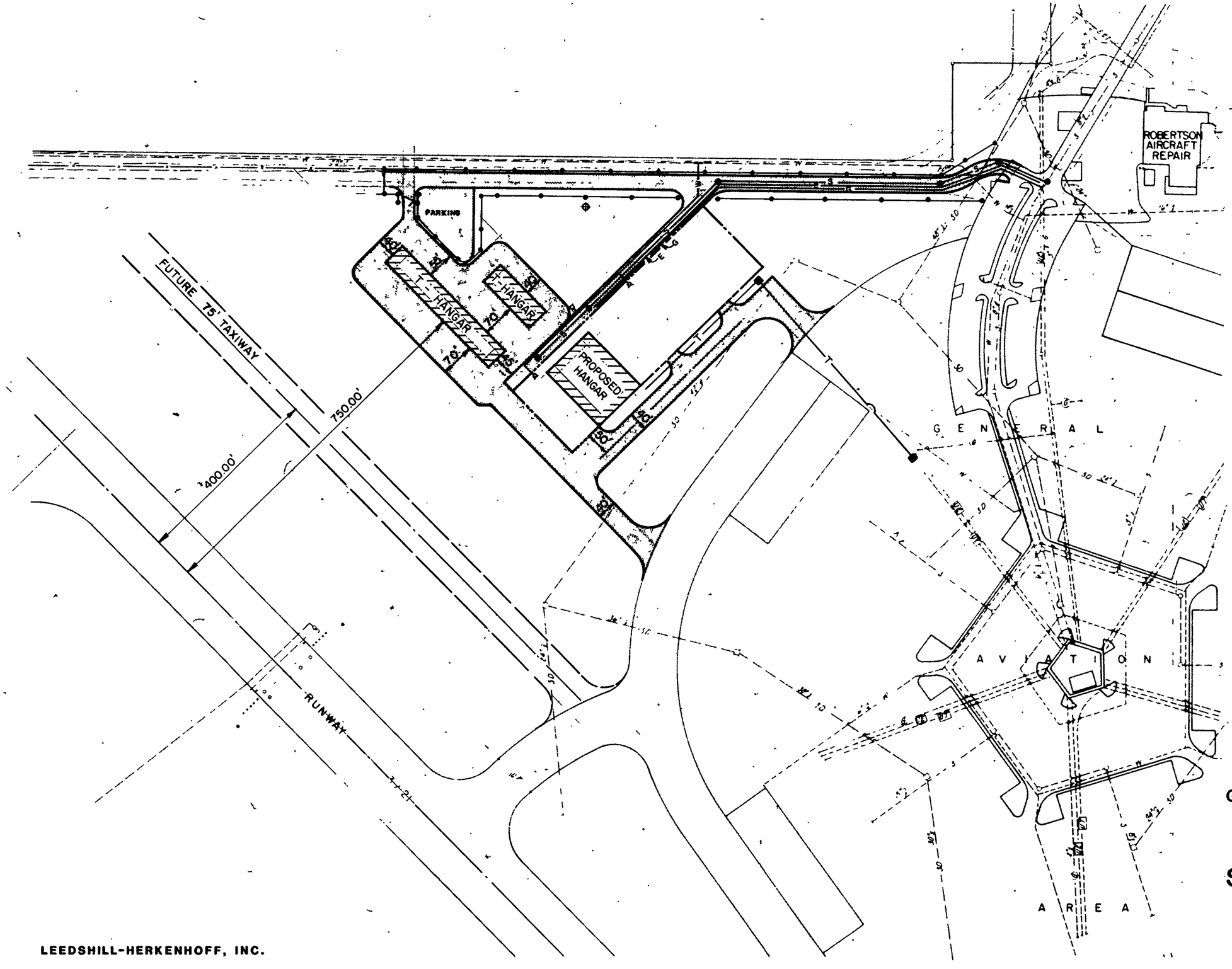
ALBUQUERQUE INTERNATIONAL AIRPORT SOUTH

GENERAL AVIATION AREA, PART 4

AREA B SCHEMATIC LAYOUT PLAN "B"

LEEDSHILL-HERKENHOFF, INC.

FIGURE II-2



N

100 50 0 100 200

SCALE IN FEET

LEGEND

- 32' ROAD PAVEMENT
- APRON PAVEMENT
- S — SANITARY SEWER LINE (M.H.)
- G — GAS LINE
- E — ELECTRICAL LINE
- CHAIN LINK FENCE
- F.H W — WATER LINE
- PROPOSED LEASE BUILDING

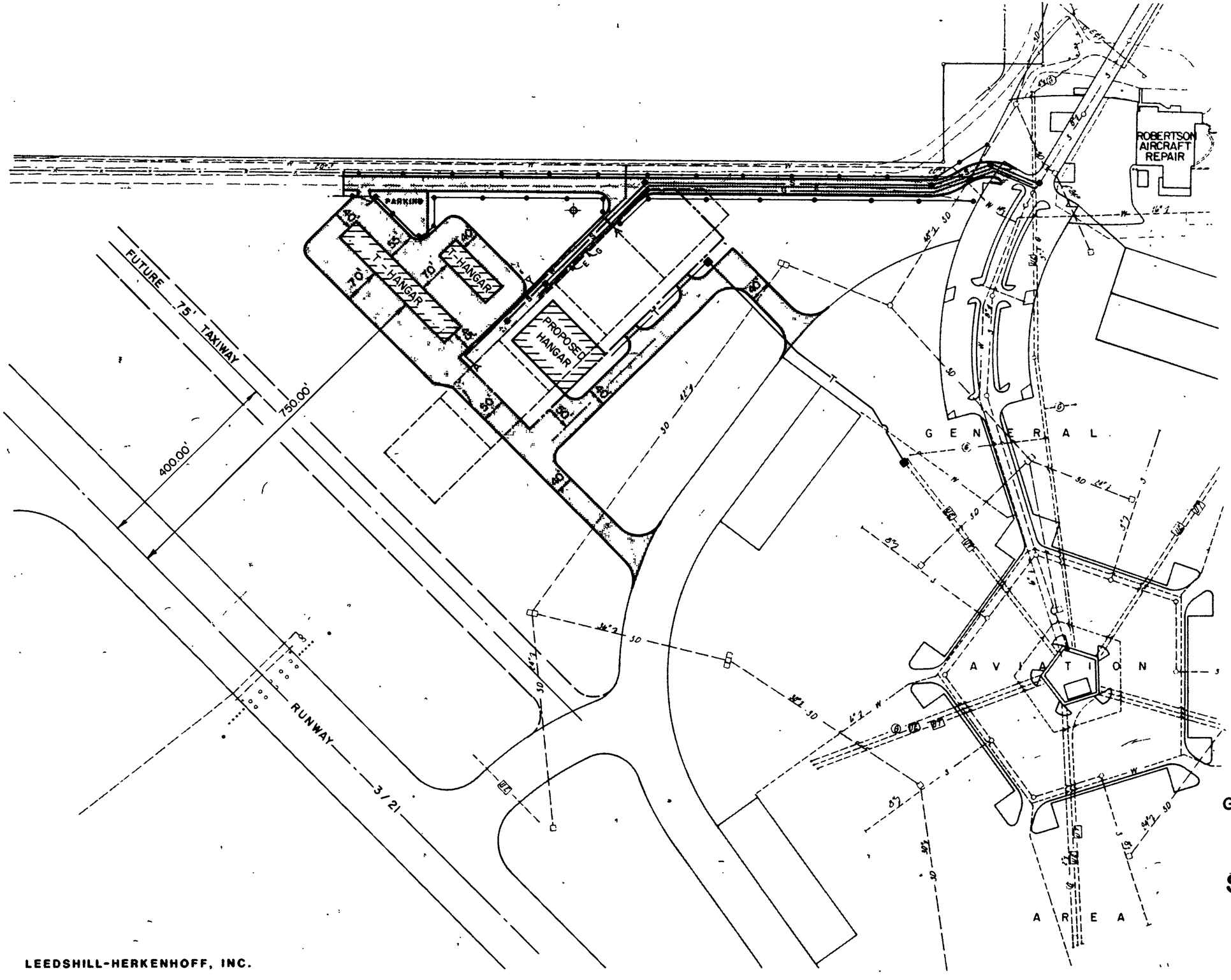
ALBUQUERQUE INTERNATIONAL AIRPORT SOUTH

GENERAL AVIATION AREA, PART 4

AREA B SCHEMATIC LAYOUT PLAN "C"

FIGURE II-3

LEEDSHILL-HERKENHOFF, INC.



LEGEND

- 32' ROAD PAVEMENT
- APRON PAVEMENT
- S — SANITARY SEWER LINE (M.H.)
- G — GAS LINE
- E — ELECTRICAL LINE
- CHAIN LINK FENCE
- W — WATER LINE (F.H.)
- PROPOSED LEASE BUILDING
- ORIGINAL LEASE PLOTS

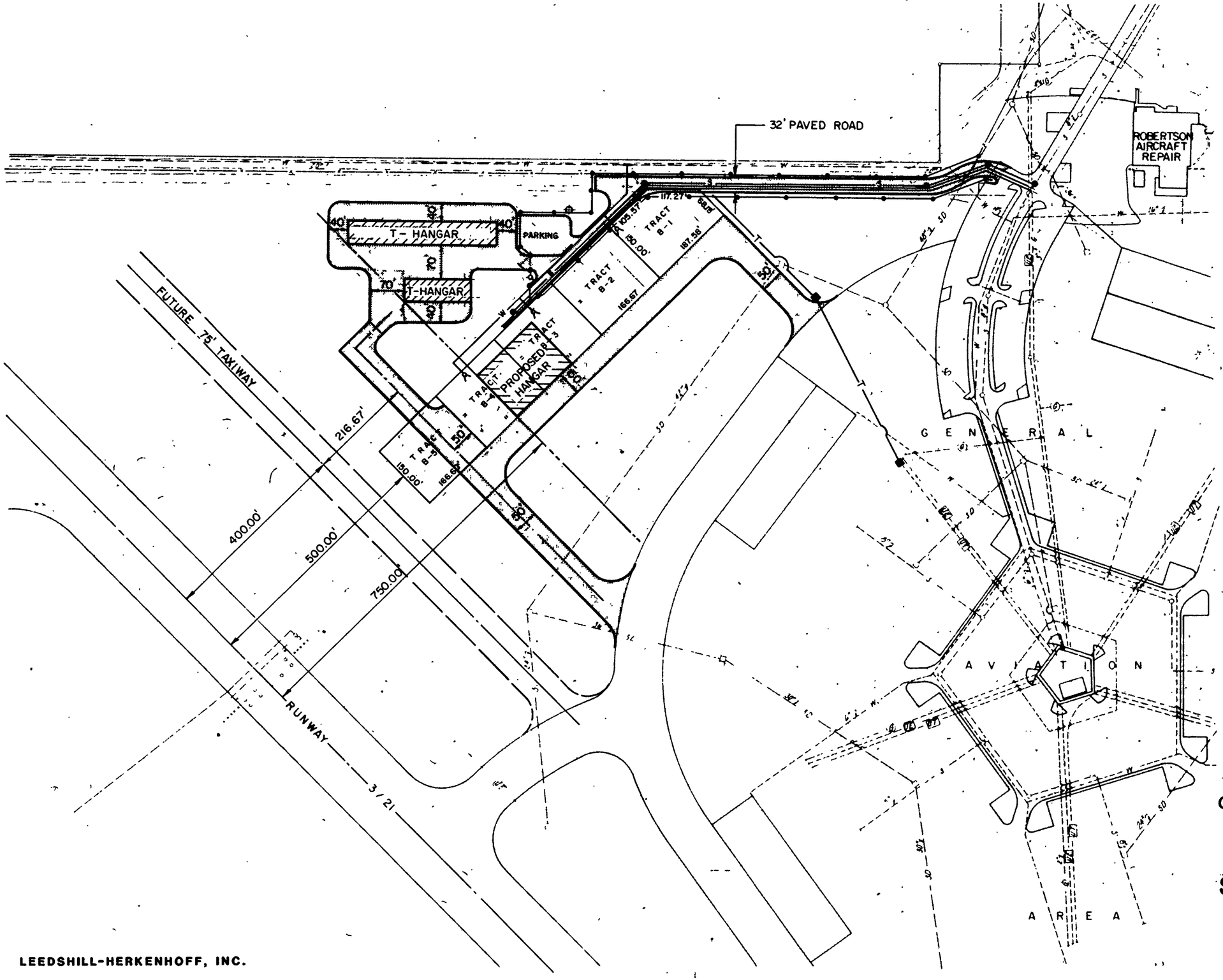
ALBUQUERQUE INTERNATIONAL AIRPORT SOUTH

GENERAL AVIATION AREA, PART 4

AREA B SCHEMATIC LAYOUT PLAN "D"

LEEDSHILL-HERKENHOFF, INC.

FIGURE II-4



ALBUQUERQUE INTERNATIONAL AIRPORT SOUTH

GENERAL AVIATION AREA, PART 4

AREA B SCHEMATIC LAYOUT PLAN "E"

FIGURE II-5

LEEDSHILL-HERKENHOFF, INC.

ROSS AVIATION HANGAR

WESTERNAIR INC. HANGAR

ROBERTSON AIRCRAFT REPAIR

G E N E R A L

A V I A T I O N

A R E A

CONCRETE PAD TO BE REMOVED

HOLDING LANE

100'R

50'

18'

SD

TAXIWAY T-13

RUNWAY 12/30

TAXIWAY T-2

RUNWAY 8/26

℄ CURVE DATA

Δ = 40°00'00"
R = 996.45'
T = 362.68
D = 5°45'

N

100 50 0 100 200

SCALE IN FEET

LEGEND

- TAXIWAY PAVEMENT
- UE — UNDERGROUND ELECTRICAL DUCT
- SD — STORM DRAIN

LEEDSHILL-HERKENHOFF, INC.

ALBUQUERQUE INTERNATIONAL AIRPORT SOUTH

GENERAL AVIATION AREA, PART 4

TAXIWAY T-17 SCHEMATIC LAYOUT PLAN "A"

FIGURE II-6

ROSS AVIATION HANGAR

WESTERNAIR INC. HANGAR

ROBERTSON AIRCRAFT REPAIR

GENERAL AVIATION AREA

HOLDING LANE

100'R

18" SD

50'

150'R

100'R

100'R

TAXIWAY T-13

RUNWAY 12/30

TAXIWAY T-2

RUNWAY 8/26

N

100 50 0 100 200

SCALE IN FEET

LEGEND

- TAXIWAY PAVEMENT
- UE — UNDERGROUND ELECTRICAL DUCT
- SD — STORM DRAIN

ALBUQUERQUE INTERNATIONAL AIRPORT SOUTH

GENERAL AVIATION AREA, PART 4

TAXIWAY T-17 SCHEMATIC LAYOUT PLAN "B"

LEEDSHILL-HERKENHOFF, INC.

FIGURE II-7

ROSS AVIATION HANGAR

WESTERNAIR INC. HANGAR

ROBERTSON AIRCRAFT REPAIR

100'R

150'R

50'

100'R

HOLDING LANE

CONCRETE PAD TO BE REMOVED

100' R

TAXIWAY T-13

RUNWAY 12/30

TAXIWAY T-2

RUNWAY 8/26

GENERAL AVIATION AREA

LEEDSHILL-HERKENHOFF, INC.

100 50 0 100 200

SCALE IN FEET

LEGEND

TAXIWAY PAVEMENT

UE — UNDERGROUND ELECTRICAL DUCT

ALBUQUERQUE INTERNATIONAL AIRPORT SOUTH

GENERAL AVIATION AREA, PART 4

TAXIWAY T-17 SCHEMATIC LAYOUT PLAN "C"

FIGURE II-8

CHAPTER III
UTILITY EXTENSIONS

A. Water:

There is an existing 16 inch water line in the perimeter loop road in the G.A. area. It is proposed that a connection be made to this line to supply service to the T hangars and lease plot. Flow requirements are based upon some domestic water use (drinking water, restrooms, washdowns, etc.) and provisions for fire flow. Based upon City of Albuquerque criteria, a minimum size 8 inch line must be provided for this light commercial use area. Preliminary fire flow requirements are based upon total square foot area of buildings. The total square foot area of a potential lease plot hangar is estimated to be 20,000 S.F. The formula used in determining the total fire flow is:

$$F=18C(A)^{0.5}$$

where F = required fire flow in gallons per minute

C = coefficient for building construction, assumed to be 0-8 in this case

A = total floor area

In this case, the fire flow is 2,000 gallons per minute. To supply this, two hydrants each capable of handling 1,000 gallons per minute are proposed for the lease plot area. Two hydrants will also be provided adjacent to the T-hangar area.

B. Natural Gas:

A natural gas line is located in the access road to the South G.A. area, with extension into the Fixed Base Operator areas. A natural gas service will be required in the lease plot area. The Gas Company of New Mexico will install the line. This should be done in conjunction with installation of other utilities so that these utilities are in place prior to paving.

C. Sanitary Sewer:

There is an 8 inch sanitary sewer system in the access road and Fixed Base Operator area. The proposed extension into the lease plot area is via an 8 inch line to this area with 4 inch service lines as required. This sewer line extension will tie into an existing manhole located in the access road. This high strength V.C.P. sewer line will be extended into the lease plot area in this contract with individual service lines the responsibility of the leasors. The minimum slope of this line will be 0.0060 feet/foot. Manholes will be installed at 350 feet maximum spacings for access and cleaning purposes. The manhole located in front of the present Robertson hangar is 21.55 feet deep which allows a maximum average slope for the sewer of 1.9%. Drop manholes will be utilized to minimize the depth of the sewer and to prevent supercritical flow.

D. Electrical:

The proposed electrical extension to the lease plot area will ultimately be the responsibility of the lease plot individuals. The work proposed under this contract would be to construct concrete encased duck work, traffic bearing pull boxes and concrete pads for pad-mounted transformers. The primary power duct work would begin at the existing power supply with three phase power and consist of two-four inch P.V.C. concrete encased ducts. The ducts would enter traffic bearing pull boxes at regular intervals. The pull boxes would be built in accordance with Public Service Company of New Mexico specifications. The four inch duct work would lead to a concrete pad located in the utility easement west of the lease plot area. The concrete pad would be built per PNM specifications to accommodate one 120/208 V 3Ø transformer and one 120/240 V 1Ø transformer. From the transformer pads the two 4 inch ducts would continue to a pull box located near the T hangars. From this pull box, two 2 inch concrete encased PVC conduits would go to each T hangar. These conduits would be provided with steel elbows and risers to five feet above finish grade and capped. This duct work and pull box system would

allow for power to be supplied to the lease plot area as required by the leasors. It would then be their responsibility to obtain the necessary electrical service as well as metering devices.

The taxiway T-17 connection to T-13 will cross the taxiway lighting cables. Two 4 inch concrete encased ducts will be installed at the return and new 5KV L-824 cable will be installed. The circuit supplying power to the taxiway lgihts will be connected at the adjacent lights and through cables will be spliced with 5KV connectors. One duct will be kept as a spare.

Two 4 inch concrete encased ducts will also be supplied at the T-17 connection to the G.A. ramp for future use.

E. Telephone:

Duct work and pull boxes will also be provided for underground telephone service to the lease plot areas. Extension and installation of services will again be the responsibility of the lessors. The telephone trunk cable will be extended from the terminal area into the "utilities building" in the center of the FBO area. Service for the lease plot will be extended from the existing telephone duct in the FBO area. A pull box will be located at this point to allow service to the FBO hangar as well as service to the lease area. A 4'x 2' watertight pull box will be required to protect the splice. Other pull boxes will be 3'x 3' as required for turns at a maximum of 350'.

All utilities will be installed in a utility "corridor" that roughly parallels the Fixed Base Operator ramp area, then turns and extends southwest adjacent to the lease plot. This configuration allows for future expansion to the west of this area.

CHAPTER IV

PAVEMENT DESIGN

The preliminary pavement design is based upon soils data gathered at the sites in March 1983 and the FAA Advisory Circular entitled "Airport Pavement Design and Evaluation" (AC 150/5320-6C). A copy of the soils report is contained in the Appendix.

A. Taxiway T-17:

The design loading for taxiway T-17 is for a dual gear DC-9 with a gross vehicle weight of 90,000 pounds and an estimated 500 operations per year. Pavement design for aircraft is based upon the California Bearing Ration (CBR) value of the existing soil. Tests performed also included determination of Atterburg limits, in-place densities and gradation.

The soil in the area of the proposed taxiway is a silty sand, non-cohesive and well drained. The liquid limit and plasticity index are both zero. In place densities ranged from 101.7 pounds per cubic foot at a depth of 7 feet to 118.8 pounds per cubic foot at a depth of 1 foot below some existing pavement. The optimum density as determined by a Modified Proctor was 124.4 pounds per cubic foot.

California Bearing Ratio Tests were conducted and yielded the following results:

at 85% relative compaction, CBR = 8
at 90% relative compaction, CBR = 28
at 95% relative compaction, CBR = 42

Based upon these CBR values, a preliminary pavement analysis was done assuming a dual wheel geared aircraft. Using the criteria in the referenced circular with a minimum CBR of 20, it was determined that the total pavement thickness required was 9.5 inches with a minimum bituminous surface course of

4 inches and a minimum asphalt treated base course of 4 inches. This method of analysis assumes removal and recompaction of the native material to a depth of 54".

To analyze this design in another manner, a CBR value of 8 for the native soil was used. In this case the critical assumption used is that the native material has an in place density of 85% of the maximum. From the field investigation this does not seem an unreasonable assumption. Using this CBR value, a total thickness of 18.5 inches is required. The thickness of the pavement required is then 9.5 inches and with a compacted subgrade with a CBR value of 20+, no subbase is required. Using this information, the pavement section for proposed Taxiway T-17 consists of 4 inches of asphalt concrete surface course, (1800 pounds stability) tack coat if required, 6 inches of asphalt treated base course (1800 pounds stability), 6 inches of prepared subgrade (100% compaction) and 6 inches of prepared subgrade (95% compaction). This pavement section exceeds the minimum standards. Because of the high CBR values of the compacted subgrade, costs of removal and compaction of excessive subgrade excavation can be avoided using the subgrade preparation specified. The asphalt treated base course will be used in lieu of cement treated base course to minimize reflective cracking.

B. Taxilane and Aprons in Lease Plot Areas:

The design loading for this paved area is for aircraft having a gross vehicle weight of 12,500 pounds or less. Again, this pavement design is based upon the CBR value of the subgrade material. The same tests performed for the taxiway were performed in the lease plot area.

The soil in the lease plot area was slightly coarser than that in the taxiway area. The liquid limit was found to be 23, while the plasticity index was found to be zero. The in-place densities ranged from 95 to 105.9 pounds per cubic foot. The optimum density was determined by Modified Proctor to be 123.0 pounds per cubic foot and the optimum moisture was 9.6%.

The CBR values were found to be:

at 85% relative compaction, CBR = 1
at 90% relative compaction, CBR = 20
at 95% relative compaction, CBR = 23

Using the design curves in the advisory circular, and the minimum CBC value of 20, the minimum allowable thickness is 2 inches of surface course pavement.

Base course is normally required under the minimum surface. However, the minimum surface can be reduced if bound base is used. In this instance, it is proposed that the City of Albuquerque standard design for residential pavement be used. This pavement section consists of 1 inch of asphalt concrete surface course (with 1500 pound stability), 2 inches asphalt treated base course (with 1500 pounds stability), and 6 inches of prepared subgrade, compacted to 95% maximum density. The reasons for using this pavement section are twofold, there is additional strength for occasional heavy loads (such as trucks) and there is additional grade control in two pavement mats. This section will also meet FAA requirements.

CHAPTER V

DRAINAGE

The drainage analysis of this report is limited to the investigation of the effect of this project on the existing drainage system. The outfall of the improvements installed as part of the G.A. area is into the historic drainage path which flows westward, crosses University Blvd., UNM Golf Course, I-25 and is intercepted by the AMAFCA South Diversion Channel. A report of this system was prepared by Tom Mann and Associates titled "Albuquerque Golf Course Drainage Study (A/E 32-14) dated July 30, 1982 (referred to as AIA/UNM). A subsequent companion study for UNM by Denny-Gross and Associates is in preparation. The first report concentrates on the hydrologic and hydraulic changes brought about by the construction of the G.A. area, including sediment transport. The second report concentrates on the means of conveying this flow across the golf course.

The present plan is to convey the runoff across the golf course in a closed conduit which has sufficient capacity to carry runoff from future developments in the escarpment area below the airport. It is assumed that this plan will be put into effect.

There is an existing strom drain system in both areas of this project. This system consists of double and triple "D" inlets and 36" to 48" reinforced concrete pipe and is shown on thee layout plans. This system was constructed in 1979 as part of the Fixed Base Operations area. The collected water discharges to the arroyo on the west.

Discussion with city hydrology staff indicates a preference to pond the runoff generated from development of the lease plot area and new taxiway at the existing inlets. This would allow a gradual "bleed off" of the collected water. The inlets could be partially closed off with plates if desired to slow down the discharge rate. Current City criteria states that all ponding areas must be drained completely within 24 hours.

Hydrologic investigations were accomplished following the Development Process Manual. Outlined procedures were used to estimate rainfall volume and intensities as well as runoff volumes and peak rates.

The design storm for this project is the 6 hour 100 year storm. The volume of rainfall for this storm at the airport is 2.33 inches.

Soil in this area was classified as wink fine sandy loam having 0-5 percent slopes by the Soil Conservation Service's "Soil Survey of Bernalillo County and Parts of Sandoval and Valencia Counties, New Mexico". This soil was then classified as soil type B. Coefficients of runoff were then determined from land use descriptions as:

Pasture or range land, fair condition C = 0.34

Paved parking lots, roofs C = 0.98

Rainfall intensities were determined using the time of concentration and total 6 hour rainfall volumes. The times of concentrations were determined using the formula:

$$Tc = 0.0078 \ \frac{(L)^{0.77}}{(S)^{0.33}}$$

where Tc = time of concentration, in minutes
L = length of flow path of runoff, in feet
S = slope of flow path, ft/ft

The minimum Tc allowed for design by the DPM is 10 minutes. Actual times were calculated and averaged under 10 minutes, so a time of concentration of 10 minutes is used. These calculations are given in the Appendix.

Rainfall intensity as determined from the DPM was 5.01 in./hr. This value was used to determine the peak runoff by the rational formula, Q = CIA.

Taxiway Area: In the taxiway area, contributing runoff to the existing triple D inlet is approximately 18 acres. There exists a concrete trim pad and abandoned paved taxi of 1 acre. The proposed taxiway would consist of approximately 1.4 acres of paved area. The existing pad and strip will be removed. Using a C value of 0.39 for the developed area, a peak flow rate of 35.2 cfs was calculated. Flow will be collected on both sides of the taxiway, via the existing inlet and a new, single "B" inlet which will be constructed directly across from and tie into the existing inlet with an 18" RCP line. A rough grading plan of the proposed construction is shown in Figure V-1. Because of the existing pavement in the area, which will be removed, the peak runoff from new construction is expected to increase by approximately 1.4 cfs over what now exists. According to the Albuquerque International Airport/University Golf Course Drainage Study (AE 82-14), this leg of the storm drainage system can accommodate the 100 year runoff without significant ponding on the inlets. Therefore, the effect of the proposed improvement will be to increase the peak flow rate for the 100 year event from the entire northern storm drain system from the estimated 122.9 cfs from the current condition to a peak rate of 124.3 cfs, an increase of 1.1%. Total volume increases from the reported 6.699 A-F to 6.819 A-F.

Area B: Area B presently drains both into the inlet system and into a large shallow closed basin. The existing contours on the Rough Grading Plan (Figure V-2) show this existing divide. The area which does drain to the existing inlets was originally graded as part of the G.A. ramp area in preparation for possible expansion of the tiedown area. This divide roughly corresponds to the drainage divide shown in the AIA/UNM Study (Figure 7 of the study).

One concept for handling the runoff following development is to honor this drainage divide, drain the T-hangar area as it presently flows, and continue to allow ponding in this approximately 6 acre area between runway 3/21 and the improvements. According to the City Drainage Ordinance, use of permanent ponding in this manner requires waiver by the City Engineer. We

have been informed by the hydrology section staff of the City Engineer's office that such a waiver is very unlikely and their preference is to drain the entire area of development into the strom drainage system.

Consequently, the proposed drainage system which is analyzed in the following paragraphs utilizes the existing inlets. This system adds about 7 acres of drainage area to the storm drain system (identified as area 205 in the AIA/UNM study). The developed drainage in Area B will be divided into two contributing areas which flow to the existing inlets.

The west area currently drains approximately 6.3 acres. As developed approximately 1 acre will be paved access road. The undeveloped coefficient of runoff of 0.34 increases to 0.44 and the theoretical peak discharge for a 100 year event increases from 10.7 to 13.9 cfs. Runoff volume increases from 0.416 A-F to 0.538 A.F.

The east inlet presently drains approximately 8.7 acres. In the developed state, this inlet will drain approximately 15.7 acres of which 5.4 acres will be impervious. The theorectical peak rate of runoff for the 100 year storm at the inlet increases from 14.8 cfs to 44 cfs with a coefficeint of runoff change from 0.34 to 0.56. Runoff volume increases from .574 A-F to 1.707 A-F.

The Albuquerque International Airport/University Golf Course Drainage Study (A/E 82-14) investigated the storm drain capacities of this leg of the system. The report states that the storm drain system is inadequate to convey peak discharge from the 25 year 6 hour storm, as well as less frequent storms, causing water to pond on the inlets in both the G.A. area and the inlets effected by this project. Therefore, the peak runoff rate for these less frequent storms from the storm drain system will not be increased by this project. Therefore, no throttling devices on the inlets to limit peak discharges are proposed. It should be noted that if either inlet were to be completely blocked (by debris or otherwise) the storage pool from the 100 year event does not innundate any paved areas.

ROSS AVIATION HANGAR

WESTERNAIR INC. HANGAR

ROBERTSON AIRCRAFT REPAIR

See fig V-2

CONCRETE PAD TO BE REMOVED

HOLDING LANE

DA = 18 Ac.
Q_{P100} = 35.2 cfs

GENERAL AVIATION AREA

TAXIWAY T-13

TAXIWAY T-2

RUNWAY 12/30

RUNWAY 8/26

N

100 50 0 100 200

SCALE IN FEET

LEGEND

- 07 — EXISTING CONTOURS
- 07 — FINISH CONTOURS
- TAXIWAY PAVEMENT AREA

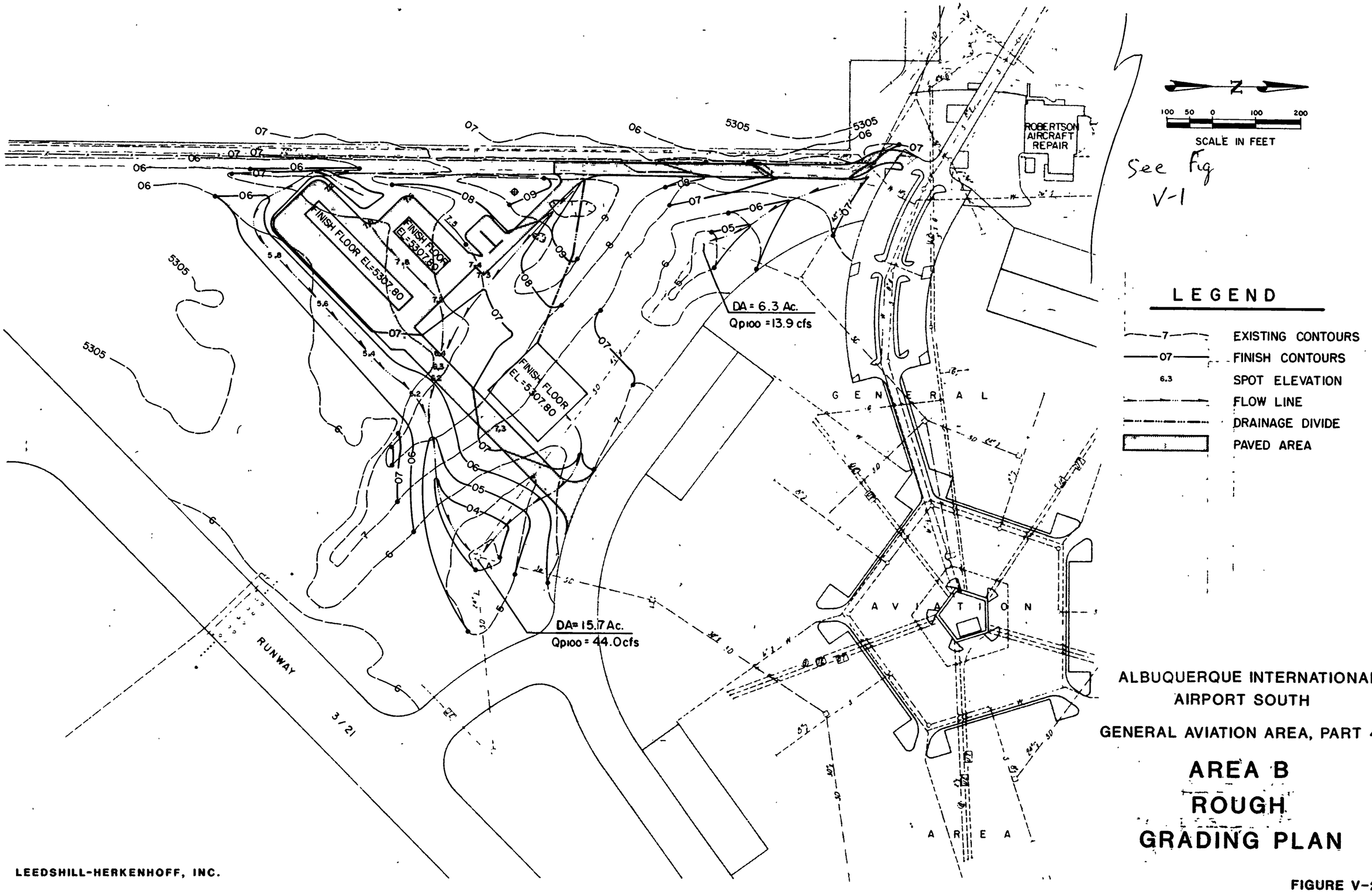
LEEDSHILL-HERKENHOFF, INC.

ALBUQUERQUE INTERNATIONAL AIRPORT SOUTH

GENERAL AVIATION AREA, PART 4

TAXIWAY T-17 ROUGH GRADING PLAN

FIGURE V-1



ALBUQUERQUE INTERNATIONAL AIRPORT SOUTH

GENERAL AVIATION AREA, PART 4

AREA B ROUGH GRADING PLAN

LEEDSHILL-HERKENHOFF, INC.

FIGURE V-2

CHAPTER VI

SUMMARY

The recommendations for each element of the project are contained in the section dealing with the design variables. In summary, the recommended project and preliminary construction cost estimates are as follows:

Taxiway T-17: Plan A connecting the T-2/T-13 intersection to the northwest edge of the FBO-GA ramp taxi lane is the preferred alternate. It offers the best traffic flow of all the alignments.

Area B: Plan A is the recommended alternate. It involves re-aligning the lease plot area to maximize utilization of the available land and optimize access.

Security Fencing: Security fencing will be provided to control automobile access for those FBO areas which are not scheduled for immediate development (all areas except Cutter).

The contruction cost estimates for the project are:

T-17	$244,000
Area B	346,000
Security Fence	40,000
Total Construction	$630,000

(See Appendix for detailed takeoff.)

The proposed schedule for this project is:

Approval of Design Analysis	April 27, 1983
Submittal of Final Review Plans	June 1, 1983
Approval of Final Plans/Advertise	June 16, 1983
Open Bids	June 30, 1983
Award and Notice to Proceed	July 7, 1983
Complete Construction	Oct. 1, 1983

This is a very ambitious schedule particularly for the review and award sequences and will require a concerted effort by everyone involved.

LEEDSHILL · HERKENHOFF, INC.
Copper Square
P. O. Box 1217
Albuquerque, New Mexico 87103
(505) 247-0294

683-1909.11-83

April 22, 1983

Mr. Lowell Pratt, AAE
Director of Aviation
City of Albuquerque
P.O. Box 1293
Albuquerque, New Mexico 87103

RE: ALBUQUERQUE INTERNATIONAL AIRPORT SOUTH GENERAL AVIATION AREA - PART 4
DESIGN ANALYSIS REPORT ADDENDUM

During in-house review of the Design Analysis report submitted April 12, 1983; another possible layout for Area B was uncovered. This alternate has potential advantages over the recommended layout and this addendum is submitted to allow City review of this additional data.

The schematic layout is shown on the attached Figure labeled Plan "F". This layout is similar to Plan "A" except the access taxilane is located on the western side of the lease plot area with subsequent minor modification of the circulation in the T-hangar area. The lease plot layout, T-hangar orientation access road and utility extensions all remain the same as Plan "A".

The advantage of this layout over Plan "A" is the possibility for additional lease or T-hangar area west of the taxilane. The aircraft access would already be in place thus reducing the cost of such utilization. Plan "F" will cost essentially the same as Plan "A".

The disadvantage of Plan "F" is the control of access into the T-hangar and lease plot areas from the access road. The access control shown is for a single gate which would lead into both areas and which crosses the taxilane. Due to the lack of separation of ground and air traffic, we still recommend Plan "A" as the best layout.

If there are any questions, please do not hesitate to call me.

Sincerely,

CLINT DODGE

er

Encs.

N

100 50 0 100 200

SCALE IN FEET

ROBERTSON AIRCRAFT REPAIR

PARKING

T-HANGAR

T-HANGAR

PROPOSED HANGAR

FUTURE 75' TAXIWAY

750.00'

400.00'

RUNWAY 3/21

GENERAL AVIATION AREA

LEGEND

- 32' ROAD PAVEMENT
- APRON PAVEMENT
- S — SANITARY SEWER LINE (M.H.)
- G — GAS LINE
- E — ELECTRICAL LINE
- CHAIN LINK FENCE
- T — TELEPHONE LINE
- PROPOSED LEASE BUILDING
- W — WATER LINE (F.H.)

ALBUQUERQUE INTERNATIONAL AIRPORT SOUTH

GENERAL AVIATION AREA, PART 4

AREA B

SCHEMATIC LAYOUT PLAN "F"

LEEDSHILL-HERKENHOFF, INC.

FIGURE II-5A

LEEDSHILL·HERKENHOFF, INC.
500 Copper Avenue N.W.
P. O. Box 1217
Albuquerque, New Mexico 87103
(505) 247-0294

955-1909.11-83

July 7, 1983

Mr. Lowell Pratt, AAE
Director of Aviation
City of Albuquerque
P.O. Box 9022
Albuquerque, New Mexico 87119

RE: ALBUQUERQUE INTERNATIONAL AIRPORT SOUTH GENERAL AVIATION AREA - PART 4
DESIGN ANALYSIS REPORT ADDENDUM

During the review process of the above referenced project, the question of how to deal with the runoff from the area was a continual point of contention. The best solution seemed to be to use the existing inlets and storm drain system to relieve the runoff from the area in the lease plot area. This however, would increase the frequency and duration of the runoff that is now causing problems at the golf course downstream. A solution to the problem downstream is being negotiated and until this is finalized, the Hydrology department of the City has strongly recommended that no additional runoff be allowed to flow to the existing inlets. In order to proceed with the project, an interim solution of a retention pond was suggested. An existing ponding area will be utilized for this. At the time of the golf course improvements completion, the ponded area would be regraded to drain to the inlets as shown in Figure V-2 of the Design Analysis Report.

To estimate the area and depth of the retention pond, calculations similar to those contained in the Design Analysis Report were performed and a rough interim grading plan was established.

The first drainage area examined is to the north of the proposed Robertson hangar. The runoff from this area would drain to the existing inlet to the east of the hangar. The undeveloped runoff Q_{100} = 7.66 cfs, while the developed runoff Q_{100} = 9.69 cfs, an increase of 2.03 cfs. The undeveloped volume ¥ = 0.297 acre-feet and the developed volume ¥ = 0.376 acre-feet, an increase of 0.079 acre-feet. This runoff will be directed toward the inlet because the regrading required at a later date would be extensive if the runoff were ponded.

The second area considered is to the south of the proposed Robertson hangar and contains the T-hangar area. The total area is 9.18 acres. The undeveloped runoff Q_{100} = 28.75 cfs, while the developed runoff Q_{100} = 44.94, an increase

of 16.2 cfs. The undeveloped volume ∀ = 1.13 acre-feet, and the developed volume ∀ = 1.75 acre-feet, an increase in volume of 0.62 acre-feet. To determine the approximate size and depth of the ponding area, the grading plan shown in Figure V-2A, attached, was used. By planimetering the area in one inch of depth increments, a total area of 4.04 acres is required, with a maximum pond depth of 8 inches.

In the event that the proposed taxiway paralleling Runway 3/21 is constructed before the golf course improvements are completed, the ponding area could be modified to run parallel between the taxiway and lease plot area.

Support calculations and the previously mentioned grading plan are attached outlining the specifics of the ponding area. We recommend that this interim/final grading approach be taken so that the project may proceed.

If there are any questions, please call me.

Sincerely,

CLINT DODGE

er

cc: Dwayne Sheppard, City of Albuquerque-City Engineer
Steve Boberg, City of Albuquerque-Engineering
Bob Gill, City of Albuquerque-Aviation
Jim Fink, City of Albuquerque-Hydrology

ADDENDUM #2 CALCULATIONS

GORDON HERKENHOFF & ASSOCIATES, INC.
ENGINEERING COMPUTATIONS

NAME OF PROJECT: Abq. airport | SHEET NUMBER: 1 OF
COMPUTED BY | CHECKED BY | JOB NUMBER | DATE: 20 May 83

Examination of drainage utilizing ponding

Area 1 = south of Robertsons that drains to east inlet

planimeter reading 1.303 (100,000 sf)
= 130,300 sf
≈ 3 ACRES

developed area = 26400 sf = 0.61 AC.

drainage area ≈ 1.5 AC

Area 2 - south of Robertsons, enclosing Thangars

planimeter reading 3.35 (100,000 sf
= 335,000 sf
= 7.7 AC.

developed area = 220,040 sf = 5.05 ac

Look at times of concentration

Area 1 $T_c = 0.0078 \frac{L^{0.77}}{S^{0.385}}$

$T_{c_1} = 0.0078 \frac{(600)^{0.77}}{(0.0067)^{0.385}} = 7.4$ min

$T_{c_2} = 0.0078 \frac{(300)^{0.77}}{(0.0042)^{0.385}} = 5.2$ min

use $T_c = 10$ min per DPW

Area 2 $T_{c_1} = 0.0078 \frac{(630)^{0.77}}{(0.0063)^{0.385}} = 7.8$ min

$T_{c_2} = 0.0078 \frac{(570)^{0.77}}{(0.0061)^{0.385}} = 7.4$ min

again use $T_{c_{21}} = 10$ min.

ADDENDUM #2 CALCULATIONS

GORDON HERKENHOFF & ASSOCIATES, INC.
ENGINEERING COMPUTATIONS

NAME OF PROJECT

SHEET NUMBER 2 OF

COMPUTED BY | CHECKED BY | JOB NUMBER | DATE

Rainfall intensity = 2.33"

Actual rainfall intensity based on $\bar{T}c = 2.15$

$2.33(2.15) = 5.01$ in/hr

Calculate coefficients of runoff and volumes of runoff

Area 1: undeveloped condition $C = 0.34$

developed condition

$C = 0.34(3.89/4.5) + 0.98(.61/4.5) = 0.43$

$Q_{developed} = CIA$

$= 0.43(5.01)(4.5) = 9.69$ cfs

$Q_{undeveloped} = 0.34(5.01)(4.5) = 7.66$ cfs

$\Delta Q = 2.03$ cfs

$V_{developed} = CRA$

$= 0.34(2.33)(4.5/12) = 0.297$ A-F

$V_{developed} = 0.43(2.33)(4.5/12) = 0.376$ A-F

$\Delta V = 0.079$ A-F

Area 2: $C_{undeveloped} = 0.34$

$C_{developed} = 0.34(2.65/3.7) + 0.98(3.05/3.7) = 0.76$

$Q_{developed} = 0.76(5.01)7.7 = 29.3$ cfs

$Q_{undeveloped} = 0.34(5.01)7.7 = 13.11$ cfs

$\Delta Q = 16.2$ cfs

$V_{developed} = 0.76(2.33)7.7/12 = 1.14$ A-F

$V_{undeveloped} = 0.34(2.33)7.7/12 = 0.52$ A-F

$\Delta V = 0.62$ A-F

ADDENDUM #2 CALCULATIONS

GORDON HERKENHOFF & ASSOCIATES, INC.
302 Eighth Street, N.W.
Albuquerque, New Mexico 87102
(505) 247-0004
ENGINEERING COMPUTATIONS

NAME OF PROJECT

SHEET NUMBER 3 OF

COMPUTED BY | CHECKED BY | JOB NUMBER | DATE

ADD PONDING AREA RUNOFF

C = 0.34
A = 9.18 AC

Q = 0.34(5.01")(9.18 AC) = 15.64 cfs

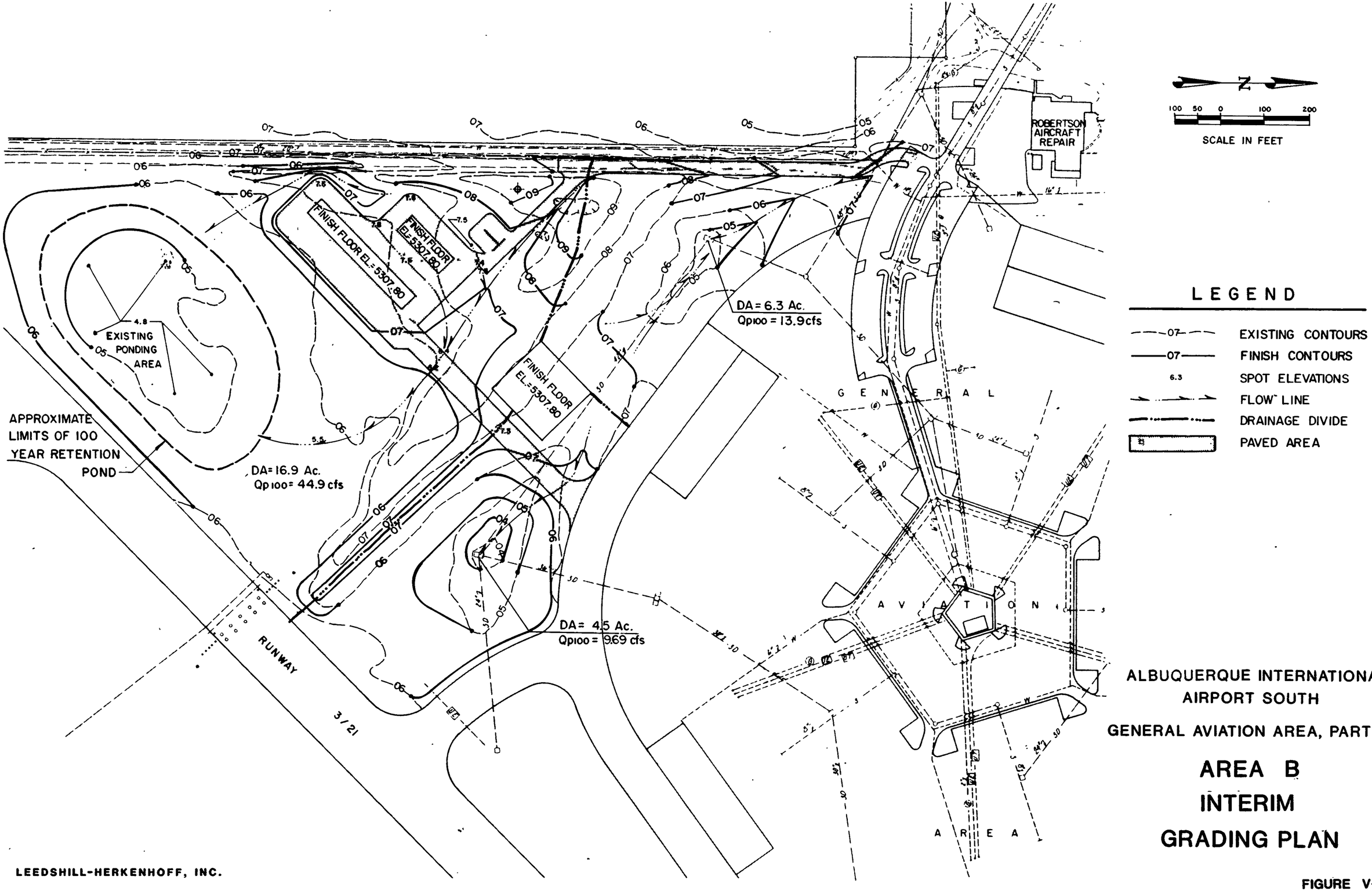
V = 0.34(2.33) 9.18/12 = 0.61 AC-FT

TOTAL VOLUME TO POND = VOLUME FROM DEVELOPED AREA
+ VOLUME FROM PONDING AREA

= 1.14 AC-FT + 0.61 AC-FT

= 1.75 AC-FT

TOTAL DEVELOPED RUNOFF = 15.64 + 29.3 = 44.94
FROM TABLE ON P. 5, THE POND VOLUME OF 1.75 AC-FT
WILL OCCUR AT APPROX EL. 5.46; POND DEPTH = 0.6625'
SURFACE AREA 190,250 SF = 4.37 AC



LEEDSHILL-HERKENHOFF, INC.

ALBUQUERQUE INTERNATIONAL AIRPORT SOUTH

GENERAL AVIATION AREA, PART 4

AREA B INTERIM GRADING PLAN

FIGURE V-2A

LEEDSHILL-HERKENHOFF, INC.
500 Copper Avenue N.W.
P. O. Box 1217
Albuquerque, New Mexico 87103
(505) 247-0294

1197-1909.11-83

July 25, 1983

Mr. Lowell Pratt, AAE
Director of Aviation
City of Albuquerque
P.O. Box 9022
Albuquerque, New Mexico 87119

RE: ALBUQUERQUE INTERNATIONAL AIRPORT SOUTH GENERAL AVIATION AREA PART 4-
DESIGN ANALYSIS REPORT ADDENDUM

During review by the Federal Aviation Administration staff of the proposed layouts for Taxiway T-17, an alternate not previously shown was requested. This layout involves a straight taxiway from Taxiway T-13 to the Fixed Base Operator Area that is on the east side of the existing storm drain inlet. A schematic of this layout is shown in Figure 11-8A. Since this was the layout preferred by the FAA and the City staff at the review, this layout is the preferred layout for final design.

Sincerely,

CLINT DODGE

er

ROSS AVIATION HANGAR

WESTERNAIR INC. HANGAR

ROBERTSON AIRCRAFT REPAIR

HOLDING LANE

100' R

50'

150' R

100' R

100' R

UE

SD

UE

GENERAL AVIATION AREA

TAXIWAY T-13

RUNWAY 12/30

TAXIWAY T-2

RUNWAY 8/26

100 50 0 100 200

SCALE IN FEET

LEGEND

- TAXIWAY PAVEMENT
- UE — UNDERGROUND ELECTRICAL DUCT
- SD — STORM DRAIN

ALBUQUERQUE INTERNATIONAL AIRPORT SOUTH

GENERAL AVIATION AREA, PART 4

TAXIWAY T-17 SCHEMATIC LAYOUT PLAN "D"

FIGURE II-8A

LEEDSHILL-HERKENHOFF, INC.

APPENDIX

CALCULATIONS	A-1 through A-17
SOILS REPORT	B-1 through B-5

A1

GORDON HERKENHOFF & ASSOCIATES, INC.
ENGINEERING COMPUTATIONS

NAME OF PROJECT: ALBQ. SOUTH GA AREA
SHEET NUMBER: 1 OF 5
COMPUTED BY: JAE
CHECKED BY:
JOB NUMBER: P09.11
DATE: 11 MAR 83

CONCEPT COST ESTIMATES FOR DESIGN ALTERNATIVES

PLAN A: RECOMMENDED ALTERNATIVE

1) RELOCATION OF TEE HANGARS	BY OTHERS
2) PAVED ACCESS TO TEE HANGARS & ROBERTSON'S 11,000 SY @ $12/SY	$ 132,000
3) ADDITIONAL PARKING 500 SY @ $10/SY	5,600
4) EXTENSION OF SANITARY SEWER 1100 LF OF 8" PIPE @ $12/LF	13,200
4 MANHOLES @ $1250/EA	5,000
5) EXTENSION OF ELECTRICITY 1100 LF OF POWER LINES (UNDERGROUND) MISC. SECURITY LIGHTING	20,000
6) EXTENSION OF NATURAL GAS 1100 LF OF 2" LINE @ $3/LF	3,300
7) EXTENSION OF WATER LINE 1050 LF OF 6" LINE @ $17/LF	11,050
8) PAVED ACCESS ROAD 3300 SY @ $10/SY	33,000
9) FENCING 4000 LF @ $10/LF	40,000
10) CURB AND GUTTER 1600 LF @ $8/LF	12,800
SUBTOTAL	$ 335,950
CONTINGENCIES (20%)	67,190
TOTAL	$403,140

NOTE: SEE SHEETS A-18 & A-19 FOR "PRELIMINARY COST ESTIMATE"

A2

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ENGINEERING COMPUTATIONS

NAME OF PROJECT

SHEET NUMBER 2 OF 5

COMPUTED BY | CHECKED BY | JOB NUMBER | DATE

PLAN B

1) RELOCATION OF TEE HANGARS	BY OTHERS
2) PAVED ACCESS TO TEE HANGARS & ROBERTSONS 116,500 SY @ $12/SY	$ 198,000
3) ADDITIONAL PARKING 560 SY @ $10/SY	5,600
4) EXTENSION OF SANITARY SEWER 1100 LF OF 8" PIPE @ $12/LF	13,200
4 MANHOLES @ $1250/EA	5,000
5) EXTENSION OF ELECTRICITY 1100 LF OF POWER LINES (UNDERGROUND) MISC SECURITY LIGHTING	20,000
6) EXTENSION OF NATURAL GAS 1100 LF OF 2" GAS LINE @ $3/LF	3,300
7) EXTENSION OF 6" WATER LINE 700 LF OF 6" WL @ $17/LF	11,900
8) PAVED ACCESS ROAD 3030 SY @ $10/SY	30,300
9) FENCING 4000 LF @ $10/LF	40,000
10) CURB & GUTTER 1650 LF @ $8/LF	13,200
SUB-TOTAL	$ 340,500
CONTINGENCIES	68,100
TOTAL	$ 408,600

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ENGINEERING COMPUTATIONS

NAME OF PROJECT			SHEET NUMBER 3 OF 5
COMPUTED BY	CHECKED BY	JOB NUMBER	DATE

PLAN C:

1) RELOCATION OF TEE HANGARS	BY OTHERS
2) PAVED ACCESS TO TEE HANGARS & ROBERTSON'S 17,000 SY @ $12/SY	$ 204,000
3) ADDITIONAL PARKING 500 SY @ $10/SY	5,600
4) EXTENSION OF SANITARY SEWER 960 LF OF 8" LINE @ $12/LF	11,520
4 MANHOLES @ $1250/EA	5,000
5) EXTENSION OF ELECTRICITY 800 LF OF POWER LINES (UNDERGROUND)	17,000
MISC SECURITY LIGHTING	
6) EXTENSION OF NATURAL GAS 800 LF OF 2" LINE @ $3/LF	2,400
7) EXTENSION OF WATER LINE 800 LF OF 6" LINE @ $17/LF	13,600
8) PAVED ACCESS ROAD 3910 SY @ $10/SY	39,100
9) FENCING 3000 LF @ $10/LF	30,000
10) CURBS & GUTTER 1700 LF @ $8/LF	13,600
SUB TOTAL	$ 341,820
CONTINGENCIES (20%)	68,364
TOTAL	$ 410,184

A4

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ENGINEERING COMPUTATIONS

NAME OF PROJECT			SHEET NUMBER 4 OF 5
COMPUTED BY	CHECKED BY	JOB NUMBER	DATE

PLAN D:

1) RELOCATION OF TEE HANGARS	BY OTHERS	
2) PAVED ACCESS TO TEE HANGARS & ROBERTSON'S 15,500 SY @ $12/SY	$186,000	
3) ADDITIONAL PARKING 500 SY @ $10/SY	5,600	
4) EXTENSION OF SANITARY SEWER 1050 LF @ $12/LF	12,600	
MANHOLES @ $1250/EA	5,000	
5) EXTENSION OF ELECTRICITY 1000 LF OF POWER LINES (UNDERGROUND)	19,000	
MISC. SECURITY LIGHTING		
6) EXTENSION OF NATURAL GAS 1000 LF OF 2" LINE @ $3/LF	3,000	
7) EXTENSION OF WATER LINE 450 LF OF 6" LINE @ $17/LF	7,650	
8) PAVED ACCESS ROAD 4300 SY @ $10/SY	43,000	
9) FENCING 4500 LF @ $10/LF	45,000	
10) CURB & GUTTER 1550 LF @ $8/LF	12,400	
SUB-TOTAL	$339,250	
CONTINGENCIES	67,850	
TOTAL	$407,100	

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ENGINEERING COMPUTATIONS

NAME OF PROJECT			SHEET NUMBER
			5 OF 5
COMPUTED BY	CHECKED BY	JOB NUMBER	DATE

PLAN E:

1) RELOCATION OF TEE HANGARS	BY OTHERS
2) PAVED ACCESS TO TEE HANGARS & ROBERTSON'S 16,000 SY @ $ 7L/SY	$ 112,000
3) ADDITIONAL PARKING 560 SY @ $ 10/SY	5,600
4) EXTENSION OF SANITARY SEWER 1250 LF OF 8" LINE @ $12/LF	15,000
5 MANHOLES @ $1250/EA	6,250
5) EXTENSION OF ELECTRICITY 1200 LF OF POWER LINES (UNDERGROUND) MISC. SECURITY LIGHTING	21,000
6) EXTENSION OF NATURAL GAS 1200 LF OF 2" LINE @ $3/LF	3,600
7) EXTENSION OF WATER LINE 500 LF OF 6" LINE @ $17/LF	8,500
8) PAVED ACCESS ROAD 2850 SY @ $10/SY	28,500
9) FENCING 4000 LF @ $10/LF	40,000
10) CURB & GUTTER 2000 LF @ $8/LF	16,000
SUBTOTAL	$ 456,450
CONTINGENCIES	91,290
TOTAL	$ 547,740

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ENGINEERING COMPUTATIONS

NAME OF PROJECT			SHEET NUMBER 1 OF 1
COMPUTED BY	CHECKED BY	JOB NUMBER	DATE

CONCEPT COST ESTIMATE OF TAXIWAY ALTERNATIVES

PLAN A:

1) KEEL SECTION:
   450054 @ $30/SY — $ 135,000

2) SHOULDER SECTION
   2,550 SY @ $13/SY — 33,150

3) DUCTS
   190 LF @ $10/LF — 1900

SUBTOTAL — $ 170,050
CONTINGENCIES (20%) — 34010
TOTAL — $ 204,060

PLAN B

1) KEEL SECTION
   27005Y @ $30/SY — $ 81,000

2) SHOULDER SECTION
   2500 SY @ $13/SY — 32,500

3) DUCTS
   150 LF @ $10/LF — 1500

SUB TOTAL — $115,000
CONTINGENCIES (20%) — 23,000
TOTAL — $138,000

PLAN C

1) KEEL SECTION
   4400 SY @ $30/SY — $132,000

2) SHOULDER SECTION
   37005Y @ $12/SY — 44,400

3) DUCTS
   220 LF @ $10/LF — 2,200

SUBTOTAL — $178,600
CONTINGENCIES (20%) — 35,720
TOTAL — $214,320

GORDON HERKENHOFF & ASSOCIATES, INC.
ENGINEERING COMPUTATIONS

NAME OF PROJECT			SHEET NUMBER
ALBQ AIRPORT			OF
COMPUTED BY	CHECKED BY	JOB NUMBER	DATE
JAE			17 MAR 83

WATER LINE EXTENSION TO SOUTH GA AREA

GENERAL NOTES:

1) THIS IS NOT A MASTER PLAN LINE
2) 8" MIN. SIZE, MAY BE LARGER DUE TO FIRE FLOW
3) MUST BE LOCATED WITHIN PAVED ROW, 25' WIDTH MIN.
4) MUST BE MIN. OF 10' FROM SAN. SEWER
5) MAX. 1200' BETWEEN IN LINE VALVES (GATE VALVES)
6) AIR RELIEF VALVES NECESSARY ON VALVES 7 8"
7) INDIVIDUAL LEASORS WILL BE RESPONSIBLE FOR THEIR METERED SERVICE
8) ASSUME THAT THIS IS A HEAVY COMMERCIAL LINE, SO FIRE HYDRANT SPACING IS 300' MAX. BETWEEN HYDRANTS AND BETWEEN HYDRANT & BUILDING.
9) FIRE FLOW REQUIREMENTS BASED UPON

$$F = 18C(A)^{0.5}$$

WHERE F = REQ'D FIRE FLOW IN G.P.M
A = TOTAL FLOOR AREA
C = ASSUMED 1.0 FOR ORDINARY CONSTRUCTION

A = 20,000 SF, (120X160 assumed)

F = 2000 gpm

FROM DPM P. 16, REQUIRE 4 HYDRANTS MIN. THIS AREA

10) ALL REQUIRED FIRE HYDRANTS SHALL SUPPLY A MIN. OF 1000 gpm AT MIN. 20 P.S.I. RESIDUAL PRESSURE FROM 4" OUTLET.
11) 8" LINE IS ALLOWABLE (V ~ 12-18 fps.)

AB

GORDON HERKENHOFF & ASSOCIATES, INC. 302 Eighth Street, N.W. Albuquerque, New Mexico 87102 (505) 247-0294 ENGINEERING COMPUTATIONS	NAME OF PROJECT A1A			SHEET NUMBER OF
	COMPUTED BY CD	CHECKED BY	JOB NUMBER	DATE

Check Sewer Velocities

$n = .013$ 8" pipe

$$V = \frac{1.5}{n} R^{2/3} \sqrt{S} = \frac{1.5}{n} (D/4)^{2/3} \sqrt{S} = 34.9 \sqrt{S}$$

$V < \text{critical} < \sqrt{gy}$ = 4.6 fps

$\sqrt{S} = .132$ $S_{max} = .0174$

available slope $S_{avail} \approx .019$

therefore, will require drop manholes

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ENGINEERING COMPUTATIONS

NAME OF PROJECT			SHEET NUMBER
SOUTH GA AREA			1 OF 5
COMPUTED BY JAE	CHECKED BY GD	JOB NUMBER	DATE 28 MAR 83

PAVEMENT DESIGN - TAXIWAY T-17 AREA

SOIL INFORMATION: COMPOSITE SAMPLES

GRADATION

SIZE	PERCENT PASSING
1"	—
3/4"	100%
1/2"	94.5
3/8"	99.0
no. 4	98.0
no. 10	95.8
no. 40	84.8
no. 80	56.8
no. 200	15.3

LIQUID LIMIT = 0
P.I. = 0

OPTIMUM DENSITY 124.4 pcf
MOISTURE 9.7%

CBR VALUES @ 85% RELATIVE COMPACTION CBR = 8
90% 28
95% 42

IN PLACE DENSITY AND MOISTURE

TEST HOLE #1	@ 1'	118.8 pcf 11.2%		@ 6'-7'	101.1 pcf 18.3%
" " #2	@ 1'	107.6 pcf 9.8%		@ 8'	114.6 pcf 11.2%

USE AC 150/5320-6C FOR PRELIMINARY DESIGN

A10

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ENGINEERING COMPUTATIONS

NAME OF PROJECT			SHEET NUMBER 2 OF 5
COMPUTED BY	CHECKED BY	JOB NUMBER	DATE

DESIGN FOR DUAL WHEEL AIRCRAFT, GROSS WEIGHT 90000 lb

SUBGRADE COMPACTION REQUIREMENTS (p.41)

USE CURVE FOR NON-COHESIVE SOIL

PERCENT COMPACTION	DEPTH REQ'D	IN PLACE ACTUAL #1	IN PLACE ACTUAL #2
100%	16"	95.5%	86.5%
95%	29.5"		
90%	42"		
85%	54"		
	81.3%		
	96"		92.1%

∴ WILL NEED TO REMOVE AND REPLACE MATERIAL TO ATTAIN REQUIRED COMPACTION.

DEPTHS INDICATED ARE BELOW FINISHED SUBGRADE

CHECK TOTAL PAVEMENT THICKNESS REQUIREMENTS (p.45)
USE MIN. ANN DEPARTURES = 1200
REQUIRED THICKNESS 4" CRITICAL AREAS } SURFACE COAT
3" NON-CRITICAL AREAS }

CBR VALUE	RELATIVE COMPACTION	THICKNESS REQ'D
8	85%	18.5"
28	90%	7"
42	95%	4.6"

TOTAL MINIMUM PAVEMENT THICKNESS SHALL NOT BE LESS THAN THAT REQUIRED FOR CBR = 20, IN THIS CASE 9.5"
MINIMUM SURFACE COURSE = 4"
MINIMUM CTB COURSE = 6"
MINIMUM ATB COURSE = 4"

FAA REGULATIONS REQUIRE BOUND BASE COURSE FOR AIRCRAFT HAVING WEIGHTS > 100,000 lb, SINCE OUR DESIGN WEIGHT = 90000 lb, WHICH IS IN REASONABLE PROXIMITY, USE BOUND BASE COURSE.

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ENGINEERING COMPUTATIONS

NAME OF PROJECT			SHEET NUMBER 3 OF 5
COMPUTED BY	CHECKED BY	JOB NUMBER	DATE

LOOK AT USING MIN. CBR VALUE RATHER THAN MASSIVE REMOVAL AND REPLACEMENT OF SUBGRADE MATERIAL. CRITICAL ASSUMPTION IS THAT NATIVE MATERIAL IS 85% MAX. DENSITY (CBR=8)

18.5" THICKNESS (TOTAL) REQUIRED

THICKNESS OF SUBBASE REQUIRED (FOR CBR=20) 9.5"

18.5" − 9.5" = 9"

NEED 9" OF SUBBASE

THICKNESS OF BASE COURSE REQ'D (FROM P. 52) 6" MIN.

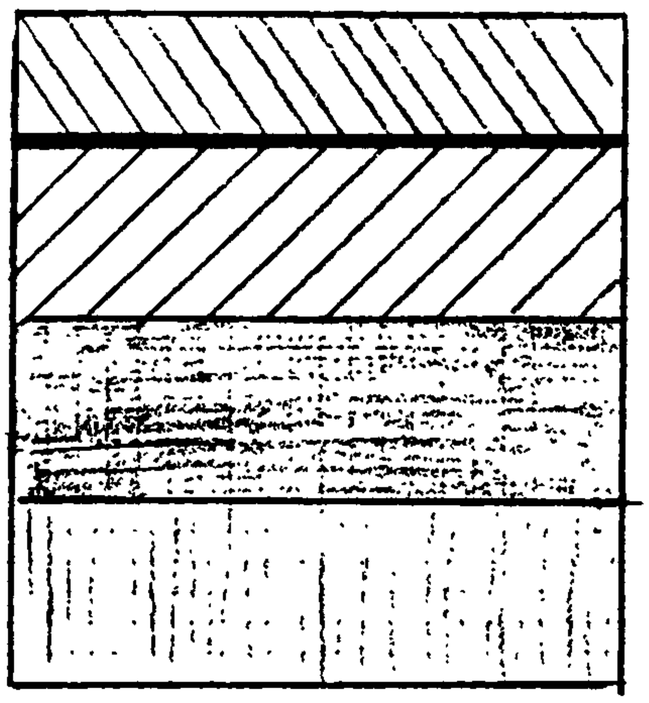
PAVEMENT SECTION WOULD THEN BE

SURFACE COURSE	4"
A.T. BASE COURSE	6"
COMPACTED SUBGRADE	9"

COMMENTS:

1) USE ATB RATHER THAN CTB TO MINIMIZE REFLECTIVE CRACKING
2) SUBGRADE COMPACTED TO 90% HAS CBR = 28. RECOMMEND USING THIS RATHER THAN SUBBASE, HOWEVER REQUIRE 100% COMPACTION ON UPPER 6"

RECOMMENDED PAVING SECTION IS:



4" ASPHALT SURFACE COURSE P-401 (STABILITY 1800)

6" ASPHALT TREATED BASE COURSE P-201 (STABILITY 1800)

6" PREPARED SUBGRADE (100% COMPACTION)

6" PREPARED SUBGRADE (95% COMPACTION)

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ENGINEERING COMPUTATIONS

NAME OF PROJECT			SHEET NUMBER 4 OF 5
COMPUTED BY	CHECKED BY	JOB NUMBER	DATE

PAVEMENT DESIGN - SOUTH GA AREA

SOIL INFORMATION - COMPOSITE SAMPLES

GRADATION

SIZE	PERCENT PASSING
1"	100%
3/4"	99.3%
1/2"	99.2
3/8"	98.9
no. 4	98.1
no. 10	96.2
no. 40	84.3
no. 80	50.3
no. 200	22.3

LIQUID LIMIT = 23
P.I = 0

OPTIMUM DENSITY 123.0 pcf
MOISTURE 9.6%

CBR VALUES @ 85% RELATIVE COMPACTION CBR = 1
@ 90% " " = 20
95% = 23

IN PLACE DENSITY AND MOISTURE

TEST HOLE #3 @ 1' 97.6 pcf (79%) @ 5' 95.0 pcf (77%)
9.7% 7.0%

#4 @ 1' 96.9 pcf (79%) @ 8' 105.9 pcf (86%)
7.8% 6.0%

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ENGINEERING COMPUTATIONS

NAME OF PROJECT			SHEET NUMBER 5 OF 5
COMPUTED BY	CHECKED BY	JOB NUMBER	DATE

DESIGN FOR LIGHT AIRCRAFT, GROSS WEIGHT 12,500 lb.

P.125

CBR VALUE	RELATIVE COMPACTION	THICKNESS REQ'D
1	N/A	N/A
20	{ MIN. THICKNESS 2" SURFACE COURSE	
23	3" BASE COURSE	

∴ CAN PROBABLY USE COMPACTED SUBGRADE AS BASE COURSE

PAVEMENT SECTION WOULD LOOK LIKE:

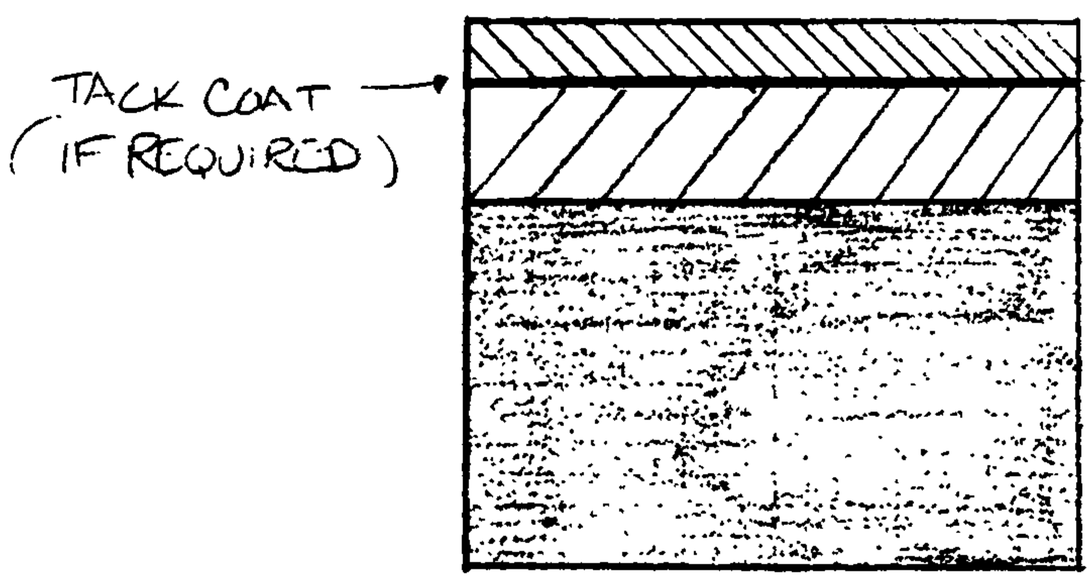
SURFACE COURSE 2"

6" COMPACTED SUBGRADE (95% MAX) AS BASE.

COMMENTS:

5320-6C IS AMBIGUOUS REGARDING REQUIREMENT FOR BASE COURSE. IT IS POSSIBLY ADVANTAGEOUS TO USE THE CITY OF ALBQ STANDARD FOR A RESIDENTIAL PAVING SECTION AND ASPHALT MIX DESIGN. THIS WOULD SATISFY FAA REQUIREMENTS AND GIVE A LITTLE MORE GRADE CONTROL AS WELL AS ADDITIONAL STRENGTH OVER MINIMUM OF 2"

RECOMMENDED PAVING SECTION IS:



NOTE: @1.5" AGGREGATE BASE = 1" ATB, WE SATISFY THE 3" BASE COURSE REQUIREMENT FROM FAA.

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ENGINEERING COMPUTATIONS

NAME OF PROJECT			SHEET NUMBER
ALBQ AIRPORT			1 OF
COMPUTED BY: JAE	CHECKED BY: CD	JOB NUMBER:	DATE:

DRAINAGE CALCULATIONS:

SOIL TYPE "B" FROM SOIL SURVEY OF BERNALILLO COUNTY BY SCS.

COEFFICIENTS OF RUNOFF (PLATE 22.2 C-2)
UNDEVELOPED LAND = 0.34
PAVED AREAS, BUILDINGS = 0.98

PEAK 6 HR 100 YR STORM = 2.33" (PLATE 22.2 D-1)

DRAINAGE IN AREA A - TAXIWAY T-17

WANT TO DETERMINE DIFFERENCE IN FLOW VALUES FROM WHAT IS NOW EXISTING COMPARED TO DEVELOPED AREA.

TOTAL AREA CONTRIBUTING FLOW TO EXISTING INLET ~ 18 AC.
EXISTING PAVED AREA AND HELIPORT = 1 AC
AREA OF NEW TAXIWAY = 1.4 AC.

$$T_C = 0.0078 \frac{L^{0.77}}{S^{0.385}}$$

TIME OF CONCENTRATION (AVG) OF EXISTING CONDITION

$$T_{C_1} = 0.0078 \frac{(550)^{0.77}}{(0.011)^{0.385}} = 5.72 \text{ min}$$

$$T_{C_2} = 0.0078 \frac{(500)^{0.77}}{(0.01)^{.385}} = 5.5 \text{ min}$$

DPM CRITERIA SAYS MIN T_C = 10 MIN FOR DESIGN

TIME OF CONCENTRATION (AVG) OF DEVELOPED CONDITION

$$T_{C_1} = \frac{0.0078(600)^{0.77}}{(0.008)^{.385}} = 6.78 \text{ min}$$

$$T_{C_2} = \frac{0.0078(500)^{0.77}}{(0.01)^{.385}} = 5.5 \text{ min}$$

AGAIN USE T_C = 10 min.

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NAME OF PROJECT			SHEET NUMBER 2 OF
COMPUTED BY	CHECKED BY	JOB NUMBER	DATE

COMPARING I = 2.33" AGAINST T_c = 10 min IN PLATE 22.2 D-2
TO GET ACTUAL INTENSITY, MULTIPLY RAINFALL VOLUME
(2.33) BY VALUE ON CURVE FOR T_c = 10 min (2.15)

I = 2.33(2.15) = 5.01" IN/HR

CALCULATE ACTUAL COEFFICIENTS OF RUNOFF

EXISTING C = $^{17}/_{18}$(.34) + $^{1}/_{18}$(.98) = .375

DEVELOPED C = $^{16.6}/_{18}$(.34) + $^{1.4}/_{18}$(.98) = .390

$Q_{DEVELOPED}$ = CIA
= .390(5.01")(18 AC) = 35.2 cfs

$Q_{UNDEVELOPED}$ = .375(5.01)18
= 33.6

ΔQ = 1.4 cfs

= CRA

$Volume_{DEVELOPED}$ = .375(2.33)(18) $^{1}/_{12}$ = 1.31 A-F

$Volume_{UNDEV}$ = .34(2.33)(18) $^{1}/_{12}$ = 1.19 A-F

ΔV = .12 AF

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NAME OF PROJECT			SHEET NUMBER 3 OF
COMPUTED BY	CHECKED BY	JOB NUMBER	DATE

DRAINAGE IN AREA B- LEASE PLOT AREA

WATER WILL DRAIN TO TWO EXISTING INLETS
WEST INLET CONTRIBUTING AREA = 6 3 AC
EAST INLET CONTRIBUTING AREA = 8.7AC (existing)

EXAMINE EAST INLET FIRST:
BUILDING AREAS = 41,700 sf ~ 1 AC
PAVED AREAS = 187,740 sf ~ 4.3 AC
NATURAL GROUND COVER = 1.7 AC. (additional)

LOOK AT RUNOFF NOW EXISTING ON UNDEVELOPED PROPERTY

$A = 8.7$ Ac

$C = 0.34$

$$T_{c_2} = 0.0078 \left(\frac{300}{(0.01)}\right)^{.77} = 371 \text{ min}$$

$$T_{c_3} = 0.0078 \frac{(500)^{.77}}{(0.0072)^{.385}} = 6.24 \text{ min}$$

MIN. $T_c = 10$ min

FROM PREVIOUS CALCULATIONS $I_{max} = 2.33(2.15) = 5.01"/HR$

$Q_{UNDEVELOPED} = CIA$
$= 0.34(5.01)(8.7)$
$= 14.8$ cfs

C OF DEVELOPED AREA
$C = 5.3/15.7(.98) + 10.4/15.7(.34) = .56$

$Q_{DEVELOPED} = CIA = 0.56(5.01)(15.7)$
$= 44$ cfs $\qquad \Delta Q = 29.2$ cfs

RUNOFF VOLUME = CRA/12 =

$V_{DEV} = 1.707$ A-F $\qquad V_{UNDEV} = .574$ A-F

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NAME OF PROJECT			SHEET NUMBER 4 OF
COMPUTED BY	CHECKED BY	JOB NUMBER	DATE

EXAMINE WEST INLET

TOTAL DRAINAGE AREA = 6.3 AC
TOTAL PAVED AREA = 1 AC

CALCULATE AVG T_c

$$T_{c_1} = 0.0078 \frac{(350)^{.77}}{(0.0086)^{.385}} = 4.43 \text{ min}$$

$$T_{c_2} = 0.0078 \frac{(400)^{.77}}{(0.009)^{.385}} = 4.82 \text{ min}$$

$$T_{c_3} = 0.0078 \frac{(400)^{0.77}}{(0.005)^{.385}} = 6.05 \text{ min}$$

$T_{c_{AVG}} = 5.1$ min

Must use $T_c = 10$ min

CALCULATE $I_{max} = 2.33"(2.15) = 5.01"/1$ HR

$Q_{UNDEVELOPED} = 0.34(5.01")(6.3 \text{ AC})$
$= 10.7$ cfs

$C_{DEVELOPED} = 1/6.3(.98) + 5.3/6.3(.34)$
$= .44$

$Q_{DEVELOPED} = .44(5.01)(6.3)$
$= 13.9$ cfs

$\Delta Q = 3.2$ cfs

$V_{UND} = .034(2.33)(6.3)\,1/12 = .416$ A-F

$V_{DEV} = .44(2.33)(6.3)\,1/12 = .538$ AF

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NAME OF PROJECT AIA			SHEET NUMBER OF
COMPUTED BY CD	CHECKED BY	JOB NUMBER	DATE 4/8/93

AREA 'B'

PRELIMINARY COST ESTIMATE

	UNIT	PRICE	QUANTITY	AMOUNT
1-4" ELECTRICAL DUCT	LF	10	1200	12000
1-3" TELEPHONE DUCT	LF	9	750	6750
2" GAS	LF	7.00	1100	7700
8" SEWER - ALL DEPTHS	LF	20.00	1100	22000
SANITARY MANHOLES	EA	1700	4	6800
MANHOLE CONNECTION	EA	100	1	100
8" WATER LINE	LF	16	750	12000
TAPPING SADDLE	EA	2500	1	2500
FIRE HYDRANTS	EA	1000	4	4000
FENCING	LF	14	2800	39,200
DRIVE GATES	EA	500	2	1000
PEDESTRIAN GATES	EA	150	1	150
RESIDENTIAL PAVING	SY	7.00	15,300	107,100
EARTHWORK	CY	2.00	45,000	90,000
R&R ASPHALT PVMT	SY	12	200	2400
8" GATE VALVE & BOX	EA	800	1	800
		SUBTOTAL		$ 314,500
10%		CONTINGENCIES		31,450
		TOTAL		$345,950

call $346,000

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NAME OF PROJECT A/A.			SHEET NUMBER OF
COMPUTED BY	CHECKED BY	JOB NUMBER	DATE 4/8/93

TAXIWAY T-17
PRELIMINARY COST ESTIMATE

DESC.	UNIT	PRICE	QUANTITY	AMOUNT
Electrical Duct -2 way	LF	10	160	$ 1600
18" RCP	LF	30	200	6000
SINGLE 'D' CATCH BASIN	EA	1900	1	1900
CONNECTION TO INLET	EA	150	1	150
5 KV L-824 CABLE & TRENCH	LF	2.00	1200	2400
ASPHALT PAVEMENT	SY	20.00	5150	103,000
SHOULDER PAVEMENT	SY	6.00	2000	12,000
REMOVAL OF PCC PVMT	SY	5.00	2300	11,500
REMOVAL OF ASPHALT PVMT	SY	2.50	2800	7,000
GRADING	SY	2.00	38000	76,000
			SUBTOTAL	$ 221,550
			CONTINGENCY 10%	22,155
			TOTAL	$243,705

call $244,000

SECURITY FENCING

FENCE	LF	14.00	2500	$ 35,000
ACCESS GATE	EA	500	1	500
PEDESTRIAN GATE	EA	150	2	300
				$ 35,800
			10% CONTINGENCIES	3,580
			TOTAL	$ 39,380

call $40,000

B-1

FOX Consulting Engineers and Geologists

F. M. FOX & ASSOCIATES, INC
3412 BRYN MAWR DRIVE, N.E.
ALBUQUERQUE, NEW MEXICO 87107
(505) 884-0900

March 30, 1983

Leedshill-Herkenhoff, Inc.
P.O. Box 1217
Albuquerque, NM 87103

Attention: Ms. Joann Azar-English

Dear Ms. Azar-English:

This letter presents the results of the subsoil investigation conducted at Albuquerque International Airport. The investigation was performed to provide subgrade information for a proposed new ramp and taxiway to be constructed at the airport.

Four (4) test holes were drilled at locations determined by Leedshill-Herkenhoff, Inc. The subsoils in each test hole were visually classified and undisturbed samples were taken to obtain in-situ densities. All density results are presented in Table 1.

The test holes generally encountered 5 to 7 feet of loose to medium dense, silty sand, overlying medium dense, clayey to very clayey sand. The subsoils between the depths of about 2 to 8 feet were very calcareous with some highly cemented lenses.

Two (2) composite bulk samples were obtained by combining the shallow subsoils from test holes #1 and #2 and from test holes #3 and #4. Each composite sample was tested to determine its gradation, Atterberg Limits, Proctor density, and California Bearing Ratio (CBR). The gradation and Atterberg Limits results are presented in Table 2, and the Proctor test and CBR results are presented in Figures 1 and 2.

If any questions remain, or if we can be of further service, please do not hesitate to call us.

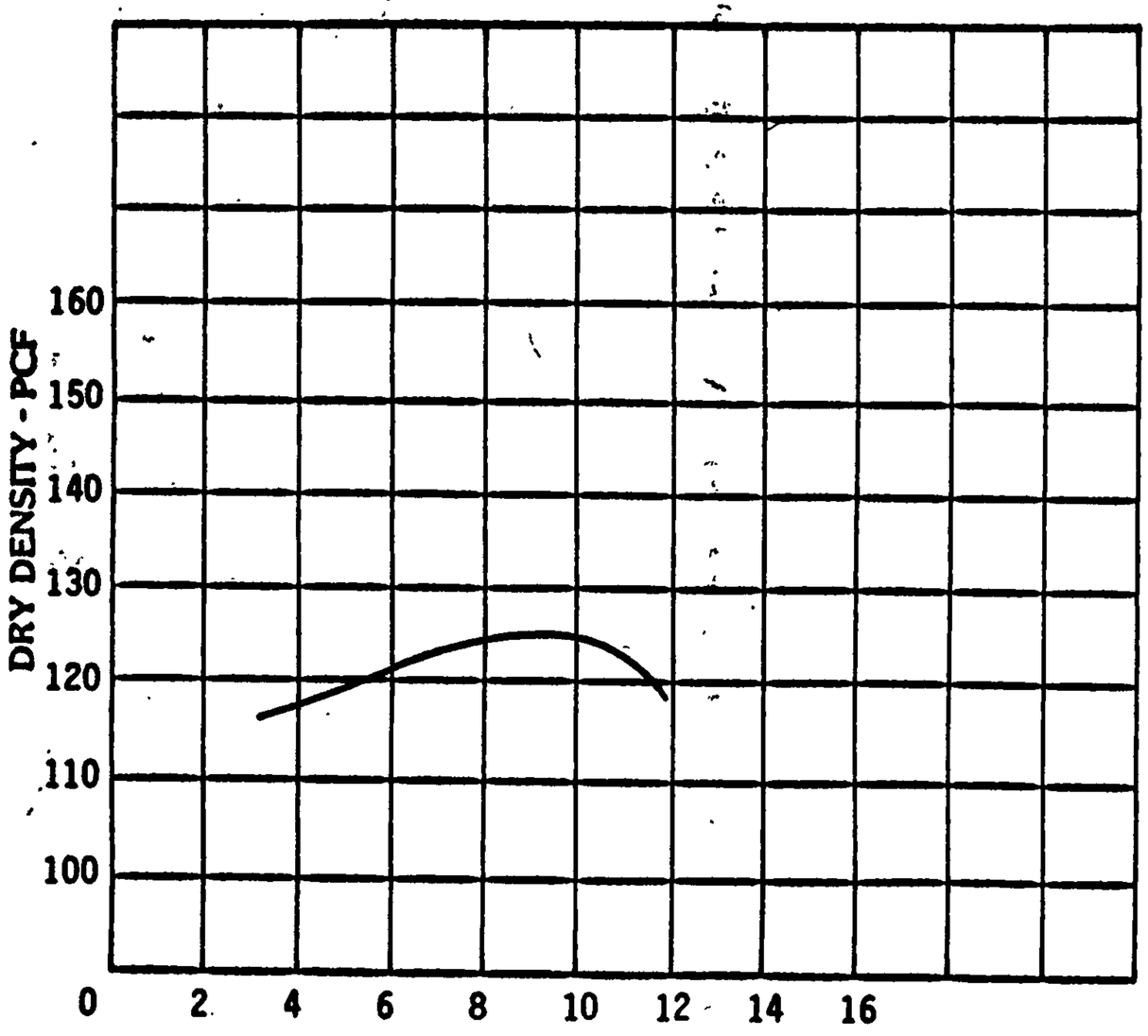
Sincerely,
F. M. Fox & Associates, Inc.

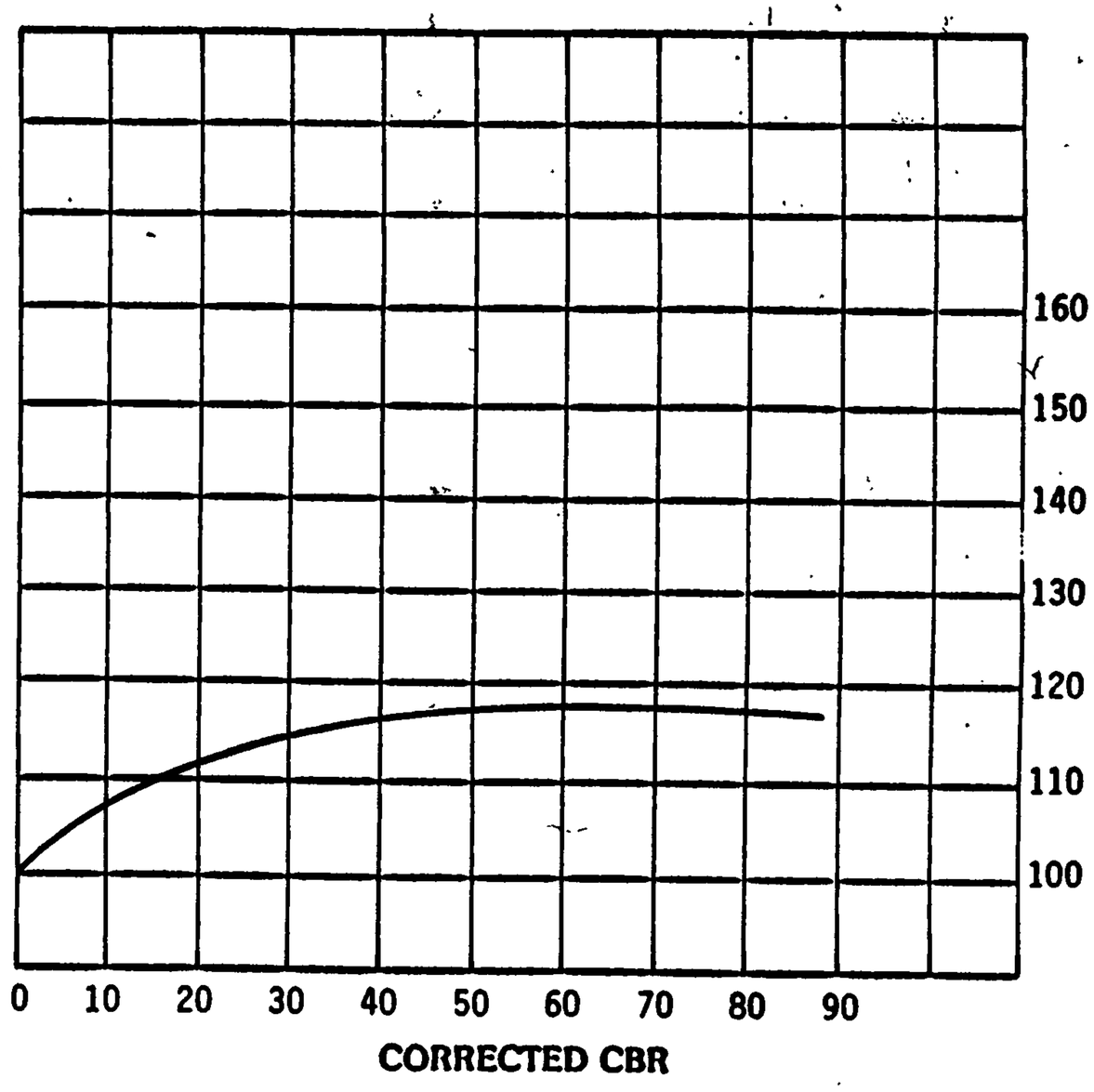
James T. McCord
Staff Geotechnical Engineer

MARTIN D. VINYARD
NEW MEXICO
8261
REGISTERED PROFESSIONAL ENGINEER

Martin D. Vinyard, P.E.
Geotechnical Division Manager

BEARING RATIO OF LABORATORY COMPACTED SOILS TEST RESULTS





Composite Sample of silty SAND from test holes 1 & 2 at a depth of 0 to 5 feet.

SURCHARGE 12.5POUNDS DURING SATURATION AND PENETRATION

AASHTO T-180 Method B
Maximum Dry Density = 124.5 pcf
Optimum Moisture Content = 9.7%

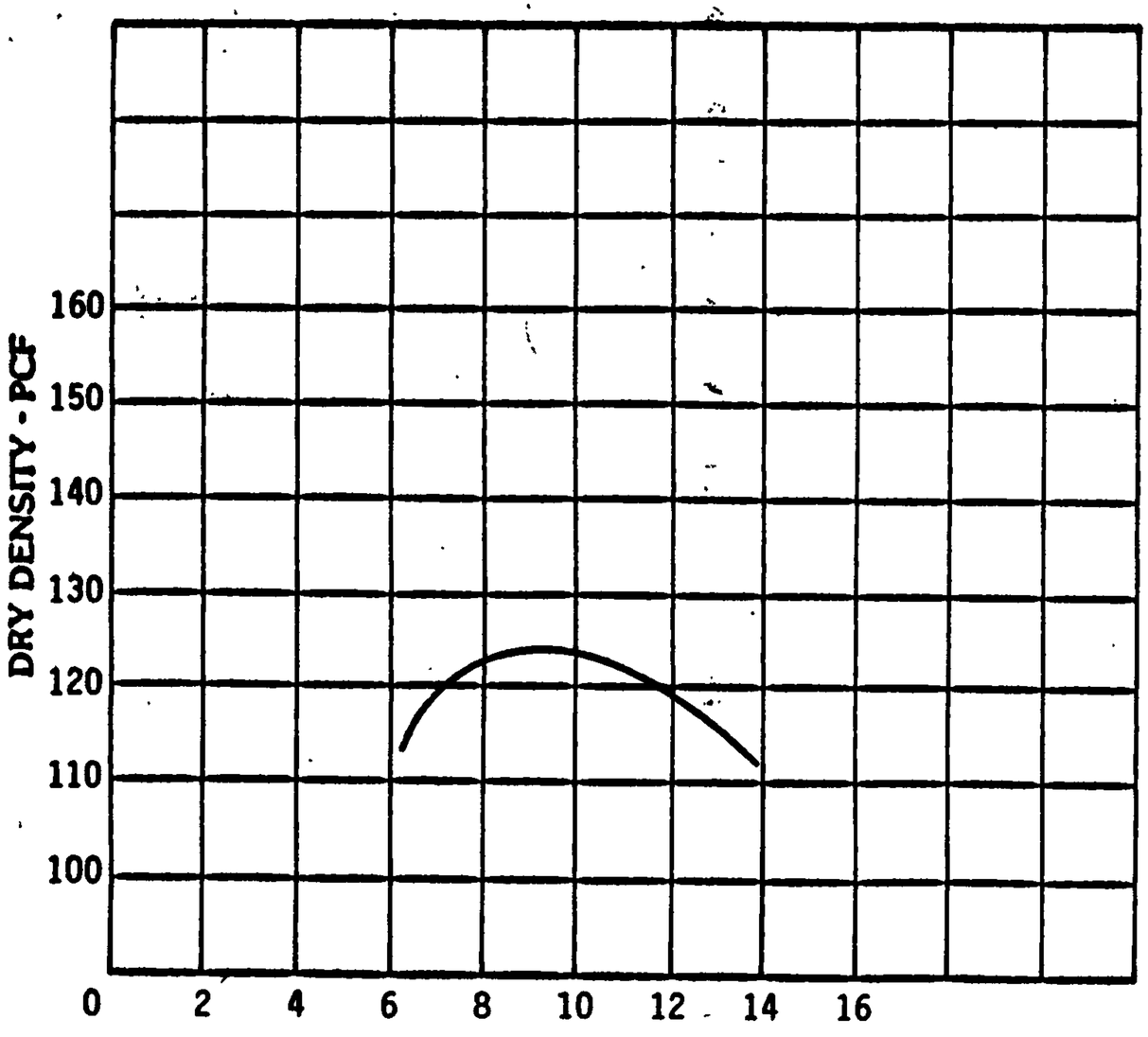
CBR = 8 @ 85% of Proctor Density
= 28 @ 90% of Proctor Density
= 42 @ 95% of Proctor Density

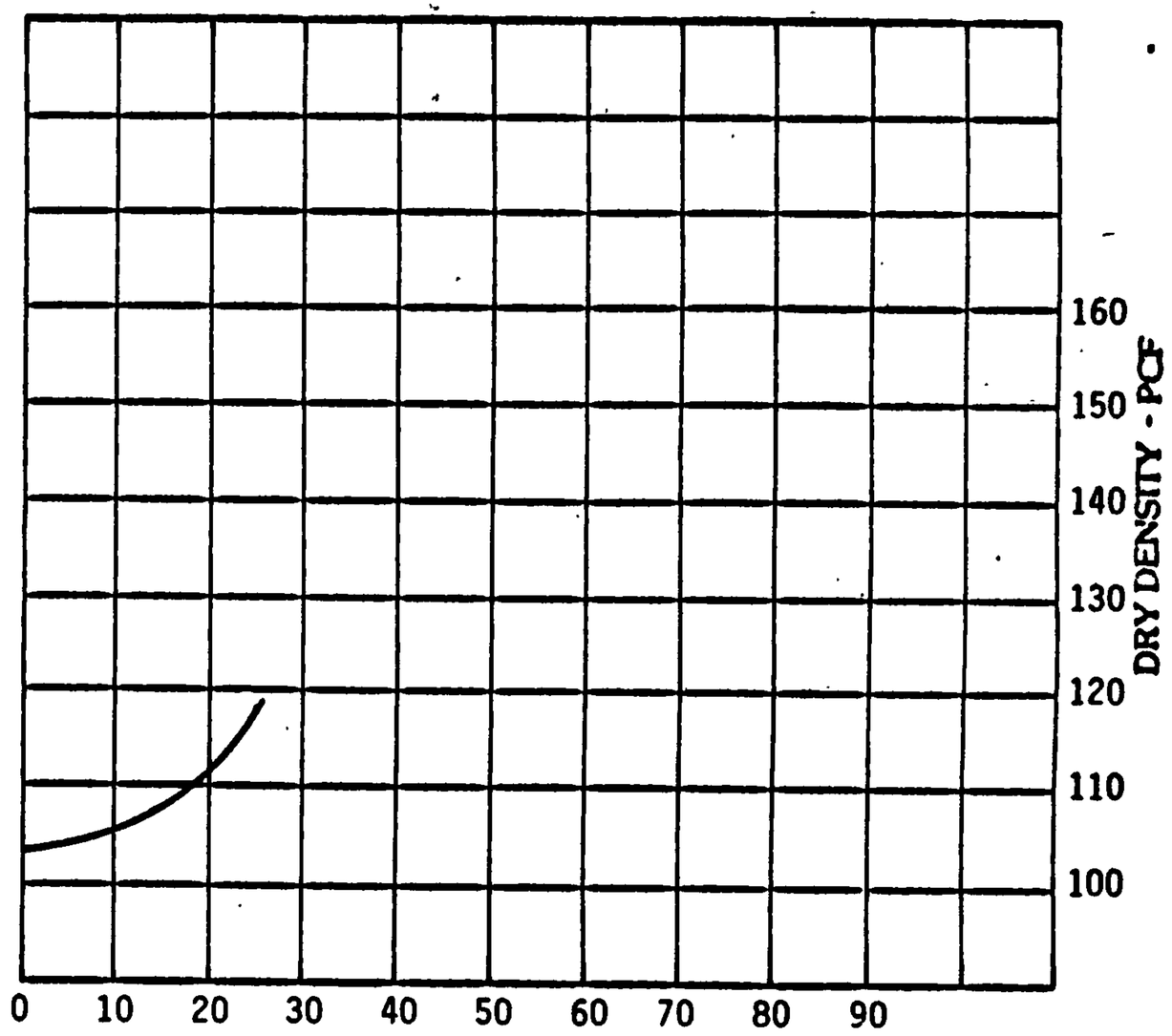
B-3

FOX Consulting Engineers and Geologists

Job No:	0111120
Date:	March 31, 1983
Figure	1

BEARING RATIO OF LABORATORY COMPACTED SOILS TEST RESULTS





Composite Sample of very silty SAND from test holes 3 & 4 at a depth of 0 to 4 feet.

SURCHARGE 12.5 POUNDS DURING SATURATION AND PENETRATION

AASHTO T-180 Method B
Maximum Dry Density = 123.0 pcf
Optimum Moisture Content = 9.6%

CBR = 1 @ 85% of Proctor Density
= 20 @ 90% of Proctor Density
= 23 @ 95% of Proctor Density

B-4

FOX Consulting Engineers and Geologists

Job No:	0111120
Date:	March 31, 1983
Figure	2

SUMMARY OF LABORATORY TEST DATA

Table 1

Hole No.	Depth of Sample (ft)	Natural Dry Density (pcf)	Natural Moisture Content (%)	Soil Description
1	1	119	11.2	silty SAND
1	6	101	18.3	clayey SAND
2	1	108	9.8	silty SAND
2	8	115	11.2	SAND-CLAY
3	1	98	9.7	silty SAND
3	5	95	7.0	silty, clayey SAND
4	1	97	7.9	silty SAND
4	8	106	6.0	silty SAND

Table 2

Test No.	Depth of Sample (ft)	Atterberg Limits LL	PI	Sieve Analysis % Passing No.4	No.10	No.40	No.200	Soil Description	Remarks
1 & 2	0-5	NV	NP	98	96	85	15	silty SAND (SM)	Fig. 1
3 & 4	0-4	23	NP	98	96	84	22	very silty SAND (SM)	Fig. 2

M-16/D024

ALBUQUERQUE INTERNATIONAL AIRPORT

DRAINAGE MASTER PLAN

June, 1994

Prepared for:

City of Albuquerque
Aviation Department
Po Box 9022
Albuquerque, New Mexico 87119

Prepared by:

MOLZEN-CORBIN & Associates

2701 Miles Road SE
Albuquerque, New Mexico 87106
(505) 242-5700

Project No. ABQ11-14 P-01

I, John M. Provine, New Mexico Registered Professional Engineer No. 10999, do hereby certify that this document was prepared under my direct supervision and is true and correct to the best of my knowledge and belief.

John M. Provine — Date

JOHN M. PROVINE NEW MEXICO 10999 REGISTERED PROFESSIONAL ENGINEER

DRAINAGE MASTER PLAN
Project No. 4255.01

✈ Albuquerque International Airport ✈

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SECTION I

INTRODUCTION

A. **General.** For many years, capital improvement projects at the Albuquerque International Airport (AIA) have been planned and completed without the aid of a formalized master plan for storm drainage. Numerous runway, taxiway, apron and terminal improvement projects have been completed along with storm drainage system components, generally designed within each project's limits. This system of individual project development has led to some problems and various inefficiencies in expenditures for storm sewer improvements. Recent increase in capital improvement activities has led to an increased need for understanding the overall nature of drainage at the AIA. While paving and/or building projects can normally be restricted to specific project limits, drainage improvements usually extend well beyond the limits of an individual project area, to include upstream runoff contribution areas and downstream runoff impacts.

There are several reasons which contributed to this situation. Individual projects came to life in rather short time frames based on need and available funding. While one project was being completed, it was uncertain just what the next project would encompass or when it would occur. However, in recent years, better planning efforts have been formulated to develop not only current construction projects but to line up several future projects. The need to better master plan all drainage network system components reached a peak in 1991 during the design of a project to reconstruct Taxiway E which is the southern parallel taxiway to Runway 8-26. At this time it was determined that reconstruction of Taxiway E would be immediately followed by a similar reconstruction project for Taxiway A which is the northern parallel taxiway to Runway 8-26. Since Runway 8-26 is the primary runway at the AIA, it was felt that Taxiway E could be reconstructed, leaving Runway 8-26 and Taxiway A in service, resulting in little disruption to air traffic. After Taxiway E was completed, Taxiway A could then be reconstructed leaving Runway 8-26 and Taxiway E in service, again resulting in little disruption of air traffic. Recognizing this planned construction sequence, it was soon discovered that the drainage system for the Taxiway E project should not be confined to the specific project limits, but should be tailored to accommodate the planned improvements for the future Taxiway A project as well. In addition, much of the contributing drainage area for the Taxiway E drainage structures actually extends northward through the Taxiway A location and much of the Kirtland Air Force Base (KAFB) areas.

There are numerous other activities at the airport which have also stressed the need for better planning of storm drainage at the AIA.

The major terminal renovation project during 1988 significantly changed the topographical nature of the airport. A significant portion of the airside infrastructure (runways, taxiways and aprons) drain through the terminal area. In addition, the terminal area contributes runoff to adjacent commercial development areas and the City of Albuquerque's storm drain system.

The AIA Airport Master Plan (AIA AMP) is to be completed in 1994. This master plan proposed substantial changes in the entire airport area which will be implemented in the very near future. The most significant changes include reconstruction of Runway 3-21 to be the primary air carrier cross wind runway (and to allow for simultaneous operations) and closure of Runway 17-35 and using the areas of Runway 17-35 to be abandoned for further expansion of the terminal facilities, and relocation of air freight operations to the southern and eastern area of Runway 3-21.

The Sunport Corridor, consisting of a new principal arterial and interchange to link the terminal area to Interstate 25 has been designed and will soon be under construction. This new roadway will change drainage patterns in the western portions of the AIA.

Construction of Spirit Drive has stimulated numerous developments on the west side of the AIA on properties owned by numerous private individuals including large areas owned by the University of New Mexico. Each of these development areas have potential impacts on the AIA drainage system and could be affected by AIA drainage.

It is now very clear that construction activities on and near the AIA property will increase significantly. A master planned storm drainage system is needed for orderly development of these proposed projects. Therefore, this document has been prepared in order to guide Aviation Department and City of Albuquerque Hydrology staff, developers and designers in implementing individual projects without adverse drainage impacts. An overall master plan such as this is not intended to final design all the details of the storm drainage system, however it is intended that the basic network character and size be determined at this time so that pipes, channels, ponds and other outfalls can be constructed at this time which will accommodate other future improvements.

B. **<u>Agencies.</u>** In master planning the storm drainage system it is necessary to first have an understanding of the numerous agencies affected by the drainage system. These agencies not only are interested in the drainage system, but will likely be involved in funding, constructing, operating and maintaining these systems.

The City of Albuquerque, Aviation Department is obviously the principal agency involved in implementing and maintaining the airport drainage system. The Aviation Department is the lead agency in overseeing all activities at the airport. KAFB currently has joint use of the AIA and also owns much of the lands that surround the airport. Some of the drainage system components will be owned and maintained by KAFB personnel. Since drainage runoff does not recognize property boundaries, the entire watersheds in many areas are also affected. This means that the City of Albuquerque Public Works Department is also involved. Much of the Gibson Boulevard storm drainage system as well as other City owned systems, is affected by this master plan. The University of New Mexico owns most of the lands west of Spirit Drive. These areas are the downstream recipient of much of the AIA flows. The New Mexico State Engineer's Office has been involved in the review and approval of at least two detention ponds constructed on or near the AIA. The Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA) has review authority over many of the drainage plans prepared for airport area development. In addition, the South Diversion Channel and Tijeras Arroyo which represent primary outfalls for airport drainage area under AMAFCA jurisdiction. The New Mexico State Highway and Transportation

Department (NMSHTD) was involved in the design of the Sunport Corridor project. In addition, much of the airport drainage system is dependent upon the capacity of culverts under I-25. The Federal Aviation Administration funds most of the airside construction projects and establishes criteria for airport design and is an integral part of this and nearly all project developments.

C. **Abbreviations.** In the following pages, numerous abbreviations will be utilized. The following list is given for a quick reference to these abbreviations:

a-f	acre-feet
AIA	Albuquerque International Airport
AMAFCA	Albuquerque Metropolitan Arroyo Flood Control Authority
cfs	cubic feet per second
CLV	Culvert
DPM	City of Albuquerque Development Process Manual
FAA	Federal Aviation Administration
FAA AC	FAA Advisory Circular
KAFB	Kirtland Air Force Base
NMSHTD	New Mexico State Highway and Transportation Department
SEO	New Mexico State Engineer's Office
SCS	Soil Conservation Service
SDC	South Diversion Channel
SD	Storm Drain

SECTION II

CRITERIA

A. General. Selection of specific design criteria is difficult for a project of this complexity. As discussed in Section I of this report, there are numerous agencies directly affected by the AIA drainage system, each with their own specific drainage design criteria. However, the criteria used in this study has been specifically tailored for the conditions at the AIA, but based upon a combination of both FAA and City of Albuquerque Criteria. While this criteria development entails a complex combination of variables, **simply stated, the criteria used was to contain the 10-year event in underground networks (pipes) and contain the 100-year event with storage.** The 10-year event was used to size all pipes and then checked against the 100 year event. If the pipes could not keep the 100 year event from ponding on paved surfaces, pipe sizes were increased to avoid ponding on paved areas when considering future development.

As stated, both FAA and City of Albuquerque criteria were used and modified in developing the criteria for the AIA DMP. Neither of these two criteria were directly useable in and of themselves. Therefore, a combination of these criteria, along with an understanding of the nature of the AIA area and operations was necessary.

City of Albuquerque storm drainage criteria states that the 10 year event should be handles in storm sewers. In addition, this criteria further states that the 100 year event shall be conveyed in the storm sewer and the street section such that one traffic lane in each direction remain dry during the 100 year event. The AIA has adopted this same criteria for the 10 year event. However, for the 100 year event some modifications were necessary. Obviously, on airports there is no curb and gutter, thereby eliminating the surface conveyance system available on city streets. On airports, the pavement surfaces must remain free of runoff. However, it must be noted that the airport does contain some street sections and furthermore, some runoff from the airport does enter City of Albuquerque street sections such as on Gibson Boulevard. Therefore, this criteria cannot be totally cast aside.

Drainage on airport areas is dictated by FAA Advisory Circular 150\5320-5B, "Airport Drainage". This AC does not give mandatory requirements, but rather provides suggestions for a reasonable approach to airport drainage design. Most of the AC is dedicated to charts, nomographs and formulas to aid designers in drainage design. However, it is prudent at this point to review some of the recommendations given in the AC since FAA funding is crucial in future airport development. Following the FAA criteria is important to ensure that this funding is not jeopardized. Some important points provided in the AC are listed here.

1. Airports shall have smooth, well drained operations areas to permit safe movement of aircraft under all weather conditions. In drainage design, there area two basic purposes, to permit movement of air traffic and to maintain stable pavement. It is important to remember that not only does drainage affect traffic but it is crucial in protecting the life of pavement areas. This is particularly difficult on airports due to the fact that airport

airside development is, by design, very flat. These flat grades, by their nature are difficult to drain.

2. Proper drainage does not assure an all-weather airport but it does shorten the interval of non-use. This requires an economically designed system to realize the full value of the investment made. More frequent storms are by definition less severe and require smaller, less expensive drainage systems. Conversely, less frequent storms are much more severe (for example, the 100 year event) and require significantly larger, more expensive storm drainage facilities. The challenge in drainage design is to properly locate the optimal point in terms of storm frequency, severity and cost of the systems.

3. Inlets are the principal problem on airports because there is normally differential settlement around the inlet. This can be prevented by careful compaction in these areas and use of expansion joints if located in concrete pavement. Medium or high head ponding will be unacceptable to personnel servicing aircraft with baggage, fuel, food, etc. This could also obscure pavement markings and thus inhibit parking of aircraft at gate positions. Aprons serving instrument or all weather air carrier operations warrant a head limitation of 0.4 feet. also airports where continuing operations are regularly conducted in severe weather conditions would warrant use of a similar limitation.

4. There are a few important points to remember in using the FAA AC in drainage design. This AC is intended to be used for all airports, from general aviation to air carrier. The AIA operates continuously on a 24 hour a day schedule and in adverse weather conditions. Therefore, FAA criteria should be followed as closely as possible but should most importantly be tailored to the specific needs of the AIA.

B. **Hydrology.** The FAA AC suggests using the 2, 5, or 10 year event for design. Storm sewers should be designed to drain ponding areas based on total storm volume and keeping the maximum water surface elevation out of pavement areas in order to keep water from infiltrating pavement bases which would cause pavement failures. This design should then be modified in order to ensure that ponding areas drain in the shortest time possible. Furthermore, the FAA AC also recommends using rainfall events of 30 minutes, 1 hour or 2 hour intensities. These are much shorter storm durations than are normally used in New Mexico.

City of Albuquerque criteria requires use of the 10 year and 100 year events as stated in previous sections of this report. In addition, these events are based on rainfall 6 hours in duration. This is not a great departure from the FAA design suggestions. While the events used by the City are less frequent and more severe than those suggested by FAA, they are also of longer duration which balances the differences. In other words, the 5 year 30 minute storm is roughly equivalent to a 10 year 6 hour storm when comparing the resulting peak flows from a watershed. Again, when considering the critical nature of operations at the AIA, the City of Albuquerque criteria can be justified to ensure maximum use of the airport, minimization of non-use times, and reasonable cost of drainage facilities.

The criteria used for this study is therefore the 10 year 6 hour event for storm sewer design, the 100 year 6 hour event for ponding and storm sewers (keeping the pavement dry) and the 100 year 24 hour event for sizing detention ponds (which is necessary for NM SOE

approvals). This criteria is not held for the Sunport Blvd. Basin which required using a 100 year, 24-hour event.

C. **Soils.** A complete summary of soils in the study area will not be presented in this report. The SCS Soil Survey for Bernalillo County was utilized in preparing this study. This survey shows that the AIA area has mostly well drained soils with some areas of impervious clays, particularly on the east end of Runway 8-26. If further information is needed regarding the soils in the area, the reader should refer to this SCS Soil Survey.

D. **Drainage Areas.** Drainage area determinations were made utilizing several topographical maps of the airport area obtained through aerial photographs prepared for this study. Areas were delineated using these topo maps and were verified through extensive field surveys and visual observation. Area maps area included in this report in the plates provided.

E. **Modelling.** This entire drainage master plan is based upon hydrologic and hydraulic modelling of the study area utilizing AMAFCA's AHYMO491 computer program. Complete printouts of the input and output data files are available under separate cover.

Computer printouts for a project of this magnitude are quite voluminous and comprehensive. In order to more accurately and rapidly illustrate the computer model methodology, flow charts and summary tables were developed to graphically show drainage area characteristics (area, peak flows, times of concentration, etc.), how this area was routed downstream, where it was added to other areas, and the resulting outfall data. In most cases, an entire basin network is shown on a single sheet and the reader can trace the theory and pertinent data without extensive revue of the computer printouts. These flowcharts and summary tables are contained in the appendices.

SECTION III

EXISTING CONDITIONS

A. General. This study began with an analysis of existing conditions at the AIA. Existing conditions for the purpose of this study includes the completion of the reconstruction of Taxiways A and E which were completed in 1994. Existing conditions does not include reconstruction of Runway 3-21 to air carrier standards and abandonment of Runway 17-35. Existing conditions includes Runway 3-21 as general aviation and the freight apron located west of Runway 3-21. It further includes completion of Spirit Drive. A basic description of the major basins is given in the following sections.

B. **Basin G17.** This 0.10140 square mile basin encompasses the northern end of Runway 17-35 and developed areas west of the runway end. Drainage from the US Customs area ramps is collected in a storm sewer, routed around the north end of the runway and through developed commercial areas west of the runway end and just south of Gibson Blvd. The system experiences problems just south of Gibson Blvd. near the old potato chip factory where the storm sewer makes a 90 degree bend to connect to the 36 inch storm sewer in Gibson. Because of this bend, the hydraulic grade line surfaces and flows leave the storm sewer and surface in the street.

TABLE III-1
Basin G17

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
101						18" SD	US Customs Apron
102						18" SD	E. of R/W 17-35 N. End
103						24" SD	N. of R/W 17-35
104						24" SD	W. of R/W 17-35 N. End
105						27" SD	Offices NW of R/W 17-35
106						Surface	Columbia Drive
107						30" SD	Offices (SAIC) NW of R/W 17-35
108						21" SD	Commercial area on Gibson
Σ	0.1014	66.97	102.89	4.599	8.215	30"SD	to Gibson Blvd. Storm Drain

C. **Basin N17.** This 0.4119 square mile basin extends along Runway 17-35 from the Runway 8-26 intersection up to the north runway end. This area also includes the main parking structure, car rental parking area, the loop road, the main terminal ramp and commercial development areas just north of the parking structures to Alamo Drive. Drainage is conveyed to the Yale Blvd. storm sewer system and eventually discharged into the Kirtland Channel. Problems with storm sewer flows surfacing have been experienced just north of the parking structure near the commercial development. Furthermore, the terminal expansion project included installation of a restrictor plate in the storm sewer conveying flows from the ramp through the terminal area. This restrictor plate causes ponding on the aircraft parking ramp south of the terminal. The restrictor plate was installed to correct the first problem described above, that is the flooding near the commercial development caused by stormwater exiting the storm drain.

TABLE III-2
Basin N17

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
201						24" SD	E. Side T/W D
202							eliminated
203						24" SD	Between R/W 17-35 & T/W D
204						24" SD	Between R/W 17-35 & T/W D
205						24" SD	Between R/W 17-35 & T/W D
206						36" SD	Between R/W 17-35 & T/W C
207						36" SD	Between R/W 17-35 & T/W C
208						24" SD	Between R/W 17-35 & T/W C
209						42" SD	Between T/W C & Access Road
210						42" SD	Between Access Road & Girard
211						24" SD	Girard Airport Entrance
212						24" SD	Rental Car Parking
213						24" SD	Rental Car Parking
214						24" SD	S. Side Airport Loop Road
214.1						10" SD	S. Side Airport Loop Road
214.2						36" SD	S. Side Airport Loop Road
215						surface	W. side of Girard
216						Surface	W. Side of Girard; N. of 215
217						Surface	Business complex
217.1						Surface	S. Side of Alamo Drive

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
218						24" SD	Lower parking lot
219						Surface	Parking Structure
220						36" SD	E. Terminal Ramp
221						36" SD	Terminal Entrance, East
222						24" SD	Terminal Entrance, West
223						60" SD	Areas E. of Alamo/Yale
224						24" SD	Amfac Hotel, East
225						Surface	Amfac Hotel, North
226						Surface	Amfac Hotel, West
227						24" SD	Airport Entrance by Yale
228						24" SD	E. side of Yale
Σ	0.4119	328.37	514.53	20.169	35.293	2-42" CMP	to Basin KC

D. **Basin KC.** This 0.6810 square mile basin extends from Yale Blvd to the South Diversion Channel and includes commercial development on each side of Randolph. The principal drainage feature of this basin is the Kirtland Channel which begins just west of the Yale/Alamo intersection and discharges to the South Diversion Channel via a baffle chute drop structure west of Mulberry. The principal problems experienced in this basin are in the lower reaches of the Kirtland Channel where the concrete lined channel makes two rather severe horizontal bends. Very high velocities which exist in the channel have resulted in the flows overtopping the channel banks in this area.

The Kirtland Channel has experienced some problems near the box culvert crossing on Mulberry. These problems have been noted through visual observations and some damages that have occurred during past rainfall events. The Kirtland Channel in this area makes two horizontal bends on what appears to be rather severe horizontal curves. Both of these bends occur just upstream from the Mulberry to form an "S" curve. The downstream curve immediately upstream of the Mulberry crossing has experienced overtopping along the southern channel bank encroaching on some existing homes in this area. In addition to this curvilinear alignment, there is a side channel discharging into the Kirtland Channel just upstream of the Mulberry crossing. The Mulberry crossing itself is a two cell 6'X6' box culvert. Downstream of this culvert is a short segment of channel and then a baffle drop spillway discharging into the South Diversion Channel.

From visual observation it appears that the problem of flow overtopping the channel is caused by superelevation of flows with high velocities around this second curve. Additional freeboard was added to this area by constructing a one foot high concrete retaining wall along this channel

edge. However, the side channel inflow from the residential areas to the north cannot be modified in this same way as these flows cannot be blocked. It also appears that the box culvert may be causing some of the problem. If the box culvert is of insufficient capacity, then backwater might be causing some of the overtopping problem. These problems were analyzed and are discussed in the next section.

TABLE III-3
Basin KC

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
301						2-54"	NE & SE Cor. Alamo/Yale
302						2-54"	N. Side Randolph, W. of 301
303						Surface	W. Side Yale, S. Side Randolph
304						Surface	W. Side Yale, S. of Randolph
305						Surface	S. of Randolph, W. of Yale
306						Surface	W. Side Yale @ Yale Airport Entrance
307						Surface	N. Side Kirt. Chan., W. of 301
308						Surface	BDM Complex
309						Surface	S. Side Randolph, W. of 303
310						Surface	S. Side Kirt. Chan., W. of 308
311						Surface	N. Side Kirt. Chan.
312						24" SD	S. Side Kirt. Chan. E. Side Univ.
313						Surface	S. Side Randolph
313.1						36" SD	SE Cor. Randolph/Univ.
314						Surface	W. of 306
314.1						Surface	E. Side Univ., S. of Randolph
315						Surface	N. of Postal Facility
316						Surface	N. Side Kirt. Chan., W. of Univ.
317						Surface	N. Side Kirt. Chan., Univ. to SDC
Σ	0.6810	568.6	961.5	30.96	54.893	Kirt. Chan.	to South Diversion Channel

E. **Basin SP.** The Sunport Corridor basin consisting of 0.2010 square miles is currently undergoing alterations. The existing conditions of this basin has been modelled but will change with the construction of the Sunport Corridor which will be a new principal arterial connecting I-25 to the AIA. The proposed construction of the roadway was accompanied by a drainage report specifically for this roadway project and is further addressed in the next section of this report.

This basin originates at the west edge of the terminal area including the new postal facility. The flows are conveyed through earth channels south of George Road where they eventually cross University in culverts. Areas west of University drain primarily by surface conveyance and pass beneath I-25 via culverts and eventually discharge to the South Diversion Channel.

TABLE III-4
Basin SP

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
401						24" SD	Post Office Site
402						CLV	Adjacent to George Road
403						SD	W of Univ, S of Randolf
404						48" SD	University to I-25
405						RR Bridge	University to RR tracks
406						CLV	SE of I-25/SDC
407						3-36" CLV	RR tracks to I-25
408						CLV	I-25
409						48" SD	SW of I-25/SDC
410						3-36" CLV	I-25
411						48" SD	I-25 to SDC
Σ	0.2010	60.19	150.6	3.846	9.445	Open Chan.	to South Diversion Channel

F. **WE Basin.** This 0.38 square mile basin includes the southern portion of the terminal area, the west ends of Runway 8-26 and the Taxiway's A and E, and the undeveloped escarpment area between the airport and University Boulevard. The existing drainage situation consists of a collection system that is throttled into a 24" outfall to the escarpment area, the overland flow within the escarpment area, and a 2-60" culvert structure at University Boulevard. In addition to being throttled at the outfall, the collection system contains many sections that are inadequate to handle the 5 year design storm. The existing peak discharge reaching the culvert crossing is 101 cubic feet per second (cfs). The capacity of the 2-60" structure is 370 cfs

The proposed system for this basin includes many upgrades to the existing collection system necessary to handle the 10 year design storm, and to solve existing drainage problems. To

accommodate future, additional flows under the master plan, a portion of the system has been upsized to accept runoff from areas scheduled for future development. The 24" outfall has been replaced with a 60" outfall that is sized to meet master plan requirements. Until the downstream drainage system and the matter of the unplatted arroyo is settled, an orifice plate should be installed to restrict the flow to existing levels. The storage capacity in the islands will be utilized to hold the peak discharge rate that exceeds the capacity of the orifice plate with this stored volume being released after the discharge rate falls below the orifice capacity. This 60" pipe discharges to a rip-rap lined channel that eventually outfalls at the University Boulevard culvert crossing. Several small arroyos within the escarpment area are intercepted by this channel. As a result of this project, the design flow in the channel is 166 cfs. The master plan discharge to be carried by the channel to the crossing is 347 cfs. Under the master plan, the additional runoff volume generated by the 100 year, 6 hour design storm will be contained in the islands.

Currently, subbasin 504, located between concourse C and B, has ponding problems during typical storms. A shallow swale carries the stormwater southwest from this portion of the terminal apron to an inlet and 24" drain in Subbasin 505. The flat grade and resulting shallow depth of the swale do not allow adequate drainage. Concern expressed by the Aviation Department over the life of the pavement with substantial ponding occurring concrete bring this situation up for resolution.

A tee installed during the Taxiway E project near the west end of Runway 8 can be extended to this area. See Section IV for specific details on this improvement.

TABLE III-5
WE Basin

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
501						24" SD	N side of 17-35/8-26
502						54" SD	N side 8-26 W of 17-35
503						30" SD	Terminal N ramp
504						Surface	Terminal W ramp
505						60" SD	8-26 W overrun
506						60" SD	8-26 W overrun
507						24" SD 18" SD	SW cor 17-35/8-26
508						48" SD	S side 8-26
509						48" SD	W end 12-30
510						54" SD	S of 8-26 W overrun
511						54" SD 24" SD	W 8-26 overrun, E of Univ

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
512						2-60" CLV	University crossing
513						surface	E of Univ, S of 8-26 overrun
514						arroyo	W of Univ
515						3-36" CLV	E of I-25
516							I-25
517						3-36" CLV	W of I-25
518						SDC	
Σ	0.3602	149.3	256.3	15.88	28.510	Open Chan.	to South Diversion Channel

G. **35SE Basin.** This 0.24 square mile basin includes a small area just east of Runway 17-35. The existing drainage system drains the area only after considerable ponding occurs along Taxiway E and an extension will be constructed to provide additional inlet capacity. This extension (28 cfs capacity) was connected to the existing system. The additional runoff volume generated by the 100 year, 6 hour design storm will be contained in the existing island. The necessary improvements in this basin were made with the Taxiway E construction project.

Did I check?

improvements?

TABLE III-6
Basin 35SE

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
601						36" SD	SE cor 17-35/8-26
602						24" SD	SW cor 17-35/8-26
603						18" SD	W of 602
604						42" SD	17-35/3-21/TW E
605						24" SD	W of 17-35
606						24" SD	E of 3-21
607						24" SD	SE end 12-30
608						42" SD	12-30
609						24" SD	S of 12-30, E of 17-35
610						48" SD	E of 17-35
611						48" SD	Escarpment

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
Σ	0.2525	68.40	82.87	8.807	17.101	48"SD	to Tijeras Arroyo

H. Basin GA.

This 0.7446 square mile basin encompasses the general aviation area located south and west of the intersection of Runways 17-35 and 3-21. These drainage basins also include areas around Clark Carr, Spirit Drive, the UNM Golf Course and pass through I-25 near the old railroad track crossing.

Flows are conveyed mostly by storm sewers which discharge into channels and then through culverts beneath I-25. There are some problems with the crossing structures on University Avenue due to low inlet capacities of these structures.

TABLE III-7
Basin GA

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
701						24" SD	R/W 12-35 NW of 17-35
702						24" SD	R/W 3-21 SW of 17-35
703						24" SD	R/W 12-35 NW of 17-35
704						18" SD	R/W 12-35 NW of 17-35
705						24" SD	Between R/W 12-30 & GA Area
706						36" SD	Between R/W 12-30 & GA Area
707						24" SD	Between T/W C & GA Area
708						24" SD	Between R/W 3-21 & T/W C
709						24" SD	Between R/W 3-21 & T/W C
710						42" SD	GA Area North
711						36" SD	GA Area SE
712						24" SD	GA Area West
713						42" SD	GA Area South
714						Surface	GA Area SW
715						42" SD	GA Area NW
716						48" SD	GA Area West

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
717						24" SD	GA Area NW
718						42" SD	GA Area NW
719						36" SD	W. Side Spirit Drive, S. Side Clark C
720						60" SD	E. Side Univ., N. of Clark Carr
721						60" SD	E. Side Univ., N. Side Clark Carr
722						24" SD	Between Freight Apron & Spirit Drive
723						24" SD	Spirit Drive
723.1						Surface	W. Side Spirit Drive
724						48" SD	Between Spirit Drive & Univ.
725						24" SD	W. Side Univ., NE Side Golf Course
726						8'X6' CLV	UNM Golf Course
726.1						60"	W. Side Univ., E. Side Golf Course
727						3-36" CLV	Between I-25 & RR Tracks
728						arroyo	Between I-25 & SDC
Σ	0.7446	207.0	449.84	23.007	46.031	Open Chan.	to South Diversion Channel

I. **Basin SW35.** This is a 0.123 square mile basin surrounding the southern end of runway 17-35 from the intersection with Runway 3-21 to the south runway end. No capacity problems exist in this area, however, the system outfall on the escarpment has experienced erosion problems. This area is scheduled for major renovations as the AMP has planned this area to be the new freight operations area. See Section IV for proposed Improvements.

TABLE III-8
Basin SW35

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
801						24" SD	W. Side R/W 17-35, S. of R/W 3-21
802						24" SD	W. Side R/W 17-35, S. end
803						24" SD	S. Runway overrun
804						18" SD	W. of R/W 17-35
805						Surface	W. of R/W 17-35

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
806						Surface	S. Runway overrun
Σ	0.1130	29.86	72.89	2.986	6.635	48" SD	to Tijeras Arroyo

J. **Basin M3.** This 0.172 square mile basin encompasses the middle portions of Runway 3-21 just south of the general aviation area. The outfall is a storm sewer discharging off the escarpment to the Tijeras arroyo. This system was constructed in 1990 with the Runway 3-21 renovation project and experiences no capacity problems but does exhibit some erosion at the discharge point.

TABLE III-9

Basin M3

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
901						24" SD	E. Side R/W 3-21
902						54" SD	R/W 3-21 by Freight Apron
903						48" SD	T/W F
904						24" SD	Between T/W F & Freight Apron
905						24" SD	N. End Freight Apron
906						48" SD	N. of Freight Apron
907						72" SD	S. of R/W 3-21
Σ	0.1719	54.02	94.11	5.880	11.549	72" SD	to Tijeras Arroyo

K. **Basin US.** This basin is made up of areas between University Boulevard and the South Diversion Channel in the UNM Golf Course area. There are no storm sewers in this system. Most of the area is undeveloped and also includes the UNM Golf Course. All runoff is surface transported to culverts which convey the flows beneath roadways through culverts. The upper reaches of the watershed are limited by University Boulevard. University diverts all upstream flows east of University to the north or south so that there is no further upstream offsite flow contributions. This is important to the integrity of the golf course as such drainage courses may have adverse impact on the golf course facilities. The golf course conveys flows by surface transport to several culverts beneath I-25 where flows are discharged to loosely defined earth channels on undeveloped property. These flows then eventually reach the South Diversion Channel. There are no identified problems within this basin relating to airport development.

TABLE III-10
Basin US

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
1001						60" CLV	University to I-25
1002							University, west
1003						4'X6' CLV 54" CLV	University to I-25
1004						SDC	I-25 to SDC
1005						SDC	I-25 to SDC
Σ						Open Chan.	to South Diversion Channel

L. **Basin W3.** This 0.13 square mile basin includes the southern most end of Runway 3-21 including most of the existing freight apron. The upper reaches of this basin includes the concrete freight apron. This apron contains several drop inlets and a 36" storm sewer. A second storm sewer exists between Runway 3-21 and the parallel taxiway and these two systems connect near the south end of the runway where a 48" storm sewer conveys flows beneath the runway to the southeast, eventually discharging the flows to the Tijeras Arroyo. No problems exist with capacity but erosion problems do occur at the discharge point on the escarpment where flows are directed to the Tijeras Arroyo.

TABLE III-11
Basin W3

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
1101						36" SD	Freight Apron
1102						24" SD	Freight Apron to TW F
1103						48" SD	Runway 3-21
1104						48" SD	SE of Runway
1105						48" SD	R/W 3-21 to T/W F
1106						36" SD	S of Freight apron
1107						surface	S of Freight apron
1108						24" SD	R/W 3 overrun
Σ	0.1305	73.04	131.31	4.419	8.815	66" SD	to Tijeras Arroyo

M. **Basin UE.** This 0.36680 square mile basin includes areas between the west airport boundary and the South Diversion Channel in the Ethicon Plant area. No airport area flows reach these watersheds and almost all drainage is conveyed through culverts. The higher reaches of the watershed include those areas adjacent to the University/Spirit Drive intersection. Flows are directed to this intersection where they are collected by several drop inlets and a short segment of storm sewer. These flows are then discharged through a 48" culvert beneath University into a natural channel. Several more drainage areas between the airport boundary and University are currently undeveloped and flows are conveyed beneath University through a series of culvert pipes. The middle portion of the basin includes the Ethicon plant area. Flows are then conveyed under I-25 through culvert pipes discharging to natural channels. Finally all flows discharge to the South Diversion Channel just north of Rio Bravo.

The character of this area will change significantly in the very near future. A grading and drainage plan for this area, known as the UNM Business Park has been submitted for approval to develop the areas adjacent to the Spirit Drive/University Intersection.

TABLE III-12
Basin UE

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
1201						DI	Spirit Drive to University
1202						DI	AIA Boundary to Spirit Drive
1203						48" CLV	AIA Boundary to University
1204						42" CLV	AIA Boundary to University
1205						54" CLV	University to I-25
1206						SDC	I-25 to Broadway
1207						42" CLV 36" CLV 48" CLV	University to I-25
Σ	0.3668	71.83	189.7	5.773	15.048	Open Chan.	to South Diversion Channel

N. **Basin RB.** This basin includes areas immediately adjacent to Rio Bravo Boulevard from the south side of Ethicon, along Broadway to the South Diversion Channel. This area was included in the computer models for this AIA DMP, however, this is not an area of great concern as no airport property flows reach this basin. There are no known problems in this basin.

TABLE III-13
Basin RB

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
1301						Surface	I-25/Rio Bravo Intx.
1302						Surface	Broadway @ RR Tracks
1303						Surface	Broadway @ SDC
Σ						Rio Bravo	to South Diversion Channel

O. **Middle Basin.** This 1.454 square mile basin is the largest basin in this design project and includes approximately 60 percent of the Runway 8-26 area to the east of Runway 17-35 and the area north to Gibson Boulevard. Developed areas within KAFB make up roughly 80 percent of the basin. The existing drainage situation consists of several collection systems within KAFB and along Runway 8-26 that drain to a 60" (84 cfs capacity) and parallel 2-24" outfalls (22 cfs capacity). Under the 10 year 6 hour design storm, these outfalls have insufficient capacity to handle the runoff from the various collection system. The outfalls discharge to the area south of the control tower through a system of channels and a 66" culvert at Kirtland Road. Below Kirtland Road, the existing arroyo carries the runoff to the Tijeras Arroyo. Including the local runoff from the areas south of the airport, the existing peak discharge reaching Kirtland Road is 117 cfs and 60.4 AF of runoff volume.

The master planned system includes new collection branches to provide drainage capacity along the reconstructed taxiway E, a parallel 72" outfall will collect the flows from the various collection systems and carry the combined flow to the detention facility. The proposed detention facility consists of an excavated reservoir on KAFB just north of Kirtland Road with the road serving as the embankment for the structure. Under the master plan flows, the facility is designed to contain the 10 year design storm with a principal spillway discharge of 86 cfs and a 100 year design storm runoff with a principal spillway discharge of 118 cfs with no flow through the emergency spillway. The emergency spillway consist of a soil cement lined low-flow channel sized to carry the 100 year 6 hour inflow of 540 cfs. One hundred percent of the runoff produced by the 100 year 6 hour rainfall will drain from the reservoir in 48 hours. Since considerable capacity is being provided for KAFB runoff that cannot be collected by the existing systems, the master planned system will carry 329 cfs and is expected to release well below the 10 year master plan level. The proposed system to be constructed in conjunction with Taxiway E will carry 186 cfs to the detention facility that will have a discharge rate of 81 cfs and 64.4 acre-feet of maximum storage. This master planned system is increasing the collection capacity within the airport and extending branches to a couple of areas that have had historical drainage problems. Also, some areas that have freely drained to the small arroyos south of the airport will be within the expanded Taxiway E and this runoff is being diverted to the detention facility. Under the 100 year, 6 hour design storm, the AIA property will generate 4.3 ac-ft of additional volume that will be contained in the islands.

All required improvements in this basin were made with the Taxiway E and A construction projects.

TABLE III-14
Middle Basin

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
1400						48" SD	E of 17-35
1401						48" SD	17-35/TW A
1402						66" SD	17-35/TW E
1403						30" SD	TW E/3-21
1404						66" SD	Detention Pond
1405							eliminated
1406							eliminated
1407							eliminated
1408							eliminated
1409							eliminated
1410						42sd	FAA ATC/Base Ops
1411						36sd	West Base Ops Apron
1412						42sd	West Base Ops Apron
1413						54sd	Island
1414						36sd	Perimeter Rd.
1415						48sd	Island
1420						54sd	Ross Apron/Hangar
1421						42sd	Apron C
1422						54sd	Center Base Ops Apron
1423						66sd	Island
1424A						72&66sd	Island
1424B						66sd	Island
1425						30sd	Island
1426						30sd	Island
1430						24sd	Island
1431						42sd	Island

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
1432						30sd	Arm/Dearm Apron
1433						30sd	Island
1440						60sd	KAFB Office Area
1441						72sd	East Base Ops Apron
1442A						72sd	Perimeter Rd.
1442B						42sd	Perimeter Rd.
1443						42sd	Expire Ramp
1444						42sd	Island
1445						36sd	Island
1446						48sd	Island
1447						78sd	Island
1448						72sd	Island
1450						2-72sd	Fire Train/Old FAA Tower
1451						surface	S. of Hot Pads
Σ						Deten. Pond	to Tijeras Arroyo

P. **East End and Air Guard Basins.** These basins cover 0.78 and 0.21 square miles respectively and are discussed together because of the drainage situation within KAFB. Over 80 percent of these two basins are within the base with two existing systems available that outfall south of the airport. All of the arroyos draining these two basins are entirely within KAFB property from the airport to their confluence with the Tijeras Arroyo. One small collection system is in the west end of the East End basin and requires much overland/channel flow before the outfall is reached. The existing 42" pipe and 24" pipe under the airport (with a combined 82 cfs capacity) is inadequate to handle the flows reaching it from the collection system. Even under small storms, considerable ponding occurs within KAFB. This system outfalls into an old landfill site where extensive erosion has occurred. KAFB has expressed concerns about increased erosion and expects to address the problems at the landfill with a future construction project. The existing flows from the basin under the 10 year design storm is 95 cfs. The other system at the Air Guard site is in the eastern end of the two basins and has 30" and 48" pipes that terminate at the north side of Taxiway A for future connections. This system (with 164 cfs capacity) has an existing 10 year discharge of 33 cfs. Currently, no erosion problems have been identified in the arroyo downstream of the outlet structure.

The proposed system in the East End basin includes a collection branch along the extension of Taxiway E, replacing the existing 42" outfall with a 48" outfall at a steeper slope, and a new 66"

outfall that will handle future collection systems within KAFB. The culvert under Kirtland Road at the outfall of the existing 42" pipe will also be replaced. The proposed 66" outfall will be capped at each end and will not carry any flow at this time. KAFB is proposing to construct improvements south of the airport to accept the flows from this master planned facility. The proposed system under this design project will carry 102 cfs under the 10 year design storm with the additional 100 year volume contained in the island ponding. Where the proposed collection branch serving Taxiway E will be intercepted by the future 66" line, an orifice plate has been installed to temporarily restrict flows to the existing level (95 cfs) at the existing outfall.

The new system in the Air Guard basin added a small collection branch added to serve the extension of Taxiway E. This branch adds 14 cfs (for a total of 47 cfs) to the system that has a capacity of 164 cfs. Based on the current conditions, we do not anticipate that this additional flow will adversely impact the downstream situation. Further, this branch intercepts a portion of the runoff that currently drains to the system that outfalls into the landfill.

TABLE III-15
East End and Air Guard Basin

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
1501						surface	Along Kirtland Drive
1502						pond	VA Hospital
1503						42" SD	
1504						48" SD	Vacant
1505						42" SD	S. Expire Ramp
1506							Between T/W A & Ramp
1507						42" SD	Between R/W 8-26 & T/W A
1508						48" SD	Between R/W 8-26 & T/W E
1509						24" SD	Between R/W 8-26 & T/W E
1510						30" SD	Ramp
1511						54" SD	N. side T/W A
1512						24" SD	Between R/W 8-26 & T/W A
1513							N. side T/W A
1514						48" SD	N. side T/W A
1515						66" SD	Between T/W A & R/W 8-26

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
1516						24" SD	Between R/W 8-26 & T/W E
Σ1	0.1705	81.67	97.46	8.785	15.223	48" CLV	to landfill
Σ2	0.6135	193.6	317.04	24.317	45.213	66" CLV	to RR Spur
1600						54" SD	Air Guard
1601						54" SD	E end 8-26
Σ	.2050	128.1	187.14	11.963	19.888	60"SD	to E. of 6000 area

Q. **Far East Basin.** This 0.24 square mile basin is almost entirely within KAFB with the northeast portion of the basin consisting of developed, technical areas and the open, graded approach areas for Runway 8-26. The drainage system consists of a series of roadside channels and culverts that pass near the southeast corner of the airport. The existing crossing onto AIA property consists of 3-24" culverts that have an approximate capacity of 45 cfs with the excess runoff spilling and ponding to the east of the runway. Including the airport runoff, the system discharges 45 cfs into the arroyo south of the airport.

Since the extension of Taxiway E crosses the existing drainage facilities, the new system consists of providing a 48" culvert to pass this runoff under the taxiway, then to its historic arroyo and a proposed inlet to drain the far east end of the taxiway. Under the 10 year design storm, the airport drainage has a design discharge of 9 cfs with the majority of the runoff occurring prior to the peak discharge from KAFB. Based on the master plan, the culvert extension was designed to handle the entire peak discharge if KAFB upgrades their system in the future with this flow being 81 cfs. For the 100 year design storm, excess ponding capacity within the taxiway island is sufficient to detain the excess runoff beyond the 10 year storm. The arroyo that drains this basin is also entirely within KAFB property.,

TABLE III-16

Far East Basin

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
1700						30" SD 48" SD	NE of R/W 8-26 end
1701						54" SD	SE of R/W 8-26 end
Σ	0.2420	46.40	96.82	10.03	18.628	54" SD	to KAFB

R. **Other Areas.** There are numerous other drainage areas which are technically inside the airport property and drain offsite. These areas include those steep slopes along the side of Tijeras Arroyo between the airport and the railroad at the toe of this slope. These areas are do not affect airport activities, and in most cases, are not impacted by airport activities. A summary of these drainage areas is shown in the following table.

TABLE III-17
Other Areas

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
1801							
1802							
1803							
1804							
1805							
1901							
1902							
1903							
1904							
1905							

S. **Summary of Existing Area Problems.** The following is a summary of the problems encountered in the existing system.

1. **Basin G17**. The storm sewer collecting runoff from the areas around the extreme north end of Runway 17-35 discharges into the storm sewer in Gibson Boulevard. Approximately half way between Yale and Girard and just south of Gibson this sewer makes a 90 degree bend to enter the Gibson system. At this severe bend, water leaves the manhole and floods the surface.

2. **Basin N17**. There are two principal problems in this basin. First, there is a restrictor plate installed in the storm sewer conveying around the terminal building. Second, there is some stormwater exiting the storm sewer just north of the lower parking lot and adjacent to the commercial development areas along Alamo Drive. The restrictor plate was installed to correct the flooding problem at this location.

3. **Basin WE**. The 2-60" culverts under University have a capacity of 100 cfs whereas the existing flows are 370 cfs.

4. **Basin WE.** Subbasin 504, between the B and C Concourses on the terminal apron ponds significantly during typical storms.

5. **Basin GA.** Crossing structures on University Boulevard have inadequate inlet capacities.

6. **Basins SE35, SW35, W3, and SE3.** AMAFCA and City have expressed concerns over erosion off the south side of the airport in the steep areas contributing to sedimentation problems downstream.

SECTION IV

PROPOSED CONDITIONS

A. **General.** The proposed development at AIA considered in this study is based on recommendations contained in the Albuquerque International Airport Master Plan, December 1993, prepared by Coffman Associates, Inc. In addition, several improvements to the storm drainage system are analyzed in this study to relieve several problems found with the existing system. The improvements recommended to relieve the existing problems are identified in-depth in Section III, Existing Conditions.

B. **Proposed Improvements - Master Plan.** Several major changes to Albuquerque International Airport are recommended in the 1993 master plan update that will significantly affect the storm drainage systems over the planning period. The proposed improvements are illustrated on Plate 3 in Appendix G. In summary the major improvements are listed below:

1. Upgrade of Runway 3-21 to air carrier status including construction of new connector taxiways and upgrade of existing taxiways.

2. Closure of Runway 17-35 and expansion of the terminal to the east of the existing parking structure.

3. Construction of new Air Cargo Facility south east of Runway 3-21.

4. Development of airport support areas generally on the west side of the airport property and east of University Blvd.

In addition to development on AIA property, it is expected that UNM will continue to develop the property west and south of Spirit Drive. Drainage impacts from this development will be seen in the GA Basin discharge and in the crossing at University Blvd. and Spirit Drive. UNM has addressed the impacts from development that affects the University Blvd/Spirit Drive crossing in the UNM Business Park Drainage Report, February 1993. Impact from additional flow and volume discharging through GA Basin will be addressed in this study.

C. **Basins East of Runway 17-35.** Far East End, Air Guard, Eend, and Middle Basins are considered to be at full development with the existing conditions. There is no expansion planned by the Airport in these areas within City property. KAFB and the NMANG may develop the areas to the north of the airfield contained in these basins but the existing storm systems, upgraded recently under the Taxiway E and A projects, have been designed and constructed to limit discharges to existing values.

The discharge from the Middle Basin is controlled at 81cfs and 64.4 AF for the 100-year, 6-hour storm. These values assume the KAFB will improve their collection system in the future, tieing onto the AIA system at connection points constructed in the Taxiway A Reconstruction project.

The existing EEND Basin is considered fully developed under this study. The portion within Airport property was expanded under the Taxiway E and Taxiway A projects. Under the master plan, two connector taxiways are planned in the south central portion of the basin. The addition of these taxiways will not significantly impact the recently constructed system. The discharges from the EEND Basin fall within KAFB property. Improvements are planned by KAFB for both of these discharge points due to their impact on an abandoned landfill. The improvements are not considered in this study since they fall downstream of the Airport and are within KAFB jurisdiction.

The Far East End Basin lies predominantly within KAFB property off the east end of the Airport. Improvements made during the Taxiway E project were sized to handle developed conditions on the airfield. If future development occurs on Air Force property, KAFB should be responsible for any increase in discharge or volume to the Airport system.

C. **G-17 Basin.** The existing conditions in Basin G17 is are assumed to be fully developed under this study. Subbasins 101 and 102 lie within KAFB and are predominantly vacant. Capacity of the storm drainage system serving these areas will not allow additional flows to be discharged. Diversion to the storm sewer system in Gibson Blvd. is not an option according to City of Albuquerque Public Works because of the existing stress on the system from current conditions. Diversion of subbasins 101, 102, 103 and 104 to N17 Basin at subbasin 206 is recommended after improvements are made within N17 Basin. The diversion will help relieve capacity problems in subbasin 108 at the 90-degree bend upstream of the basin discharge to Gibson Blvd.

Downstream of subbasin 104, the existing storm sewer system serves a light industrial and business park. It does not appear that additional significant development can occur in these subbasins. The existing drainage system is adequate for the 10-year event with the exception of the 90 degree turn in subbasin 108. Improvement to this bend will relieve flooding problems on the surrounding private property. Appendix H contains a conceptual scope for the improvements in this area.

D. **N17 - KC Basins.** Significant changes are proposed for N-17 Basin in the AIA master plan. Most significant is the proposed terminal expansion in the upper reaches of N-17 basin, specifically in subbasins 203, 204, 207, 208, 209, 210, 211, 212, 213, and 220. Currently, this area contains most of the northern portion of Runway 17-35 and Taxiway C. If Runway 17-35 is closed and the terminal is expanded into this area, it is assumed that the majority of the area will be impervious.

Under the proposed conditions outlined in the master plan, the new aircraft parking apron which will surround the new terminal area will cover subbasins 203, 204, 207, 208 and 209. Subbasin 209 is a consolidation of subbasins 209, 210, 212, 213 and a portion of 211. An increase in the storm drain system will be required to carry the increased flows from the apron and terminal as outlined below in Table IV-1.

To initiate the analysis of the lower reaches of the Kirtland Channel, general flow characteristics of the channel were computed. These are as follows:

Q	=	950 cfs			
Yn	=	2.58'	Yc	=	4.92'
n	=	0.016			
B	=	10'			
H/V	=	1.74			
Vn	=	25.44 fps	Vc	=	10.41 fps
S	=	0.0334 ft/ft	Sc	=	0.00282 ft/ft
TWn	=	18.98'			
An	=	37.38sf			

Obviously, flow in the channel is well within the supercritical flow regime and velocities are quite high. Using the DPM criteria, superelevation of the water surface was checked as follows:

$$Se = \frac{1.3*V^2(b+2zD)}{g*r} = \frac{1.3*25.44^2(10+2*1.74*2.58)}{32.2*800} = 0.62'$$

Based on these calculations, the problems experienced with the overtopping the channel side should not be occurring as the maximum outside radius depth of flow is only 3.20 feet. The channel has a lined depth of five feet. Additional freeboard is needed to maintain a two foot minimum but the overtopping of flows should not be occurring strictly from the superelevation.

Another source of the problem could involve the box culvert capacity. Using the DPM criteria, the pier losses were calculated and the results show that this box does, indeed, have proper capacity. Constructing a HEC-2 (Water surface profiles analysis) was not done as it was felt that HEC-2 would not accurately model the complex variables occuring at this location.

It is somewhat apparent that the problem is being caused by the reverse curves in the channel alignment. Since the flow in the channel is supercritical, the box itself cannot be causing a backwater affect unless the box has insufficient capacity. Calculations show that the box does have capacity. Therefore, the only affect that the box has on the flow is some disturbance at the center pier, an affect which cannot be recognized further upstream. If the flow in the channel were subcritical, then such an affect would be felt some distance upstream. While some effort has been made to increase freeboard along the channel, the success of these improvements is limited due to the presence of an additional channel discharge just upstream of the box culvert. This side channel cannot be blocked by increasing freeboard on the Kirtland Channel.

The problems experienced in this area are complex. Solutions cannot be achieved by computer analysis, but rather by a more logical approach. The Kirtland Channel should be reconstructed in a new location, further south of the existing alignment. This reconstruction should occur between the most upstream horizontal curve and the South Diversion Channel. This area is currently undeveloped and some flexibility would exist in both horizontal and vertical alignment. This reconstruction would allow the two horizontal curves to be lengthened causing less superelevation or eliminated completely. Furthermore, the side discharge from the residential areas to the north could be realigned to enter the Kirtland Channel at a more desirable location, without the potential of Kirtland flows backing up into this side channel. The box culvert could also be eliminated with this channel realignment. This is dependent upon the future planning of

the undeveloped areas surrounding the Mulberry crossing. If Mulberry will not be extended south in the future, no channel crossing will be needed.

TABLE IV-1
Improvements to N17 Basin

SUBBASIN NO.	PIPE DIAMETER	FLOW, 10 and 100 year, cfs	COMMENTS
204-207	30-inch	19.6 (10), 32 (100)	North N17 to center N17.
203-208	36-inch	24 (10), 37 (100)	Flow from 203 to 208
208-207	48-inch	49.3 (10), 76.2 (100)	South N17 to center N17
207-209	66-inch	164 (10), 255 (100)	Flow from 207 to 209
216-215	30-inch	13.8 (10), 20.3 (100)	Currently vacant parcel
220	60-inch	272.3 (10), 430.7 (100)	Flow from east terminal area
209-215-220	66-inch	272.3 (10), 430.7 (100)	Flow from new apron and terminal
214.2	42-inch	112 (10)	Flow from exist. terminal area, parking, rental lots, Craddock.
214.2-Alamo	72-inch	403 (10) 558 (100)	Flow from exist. terminal area, parking, rental lots, and new loop road curbside to new terminal.

The proposed terminal expansion will be drained through subbasin 207 collecting flows from the northern and southern portion of N17 Basin with new 30-inch and 48-inch drains, respectfully. Subbasin 209, consolidated, will accept the flow from subbasin 207 and discharge through a new 60-inch drain. This drain will increase in size to a 66-inch drain running west on the south side of the Craddock Business Park. After flow is collected from subbasin 214.2, incorporating discharges from the existing terminal area, parking area and rental car lots, a 72-inch drain will carry the 10-year flow to Alamo St. The existing 60-inch drain in Alamo St. will discharge across Yale Blvd. into the upstream reaches of the Kirtland Channel.

The current storm sewer system serving the existing terminal, parking, loop road, and rental car lot subbasins are restricted by the capacity of the 36-inch drain across subbasin 214.2 An orifice plate was installed at the discharge point of subbasin 220 when the existing terminal was expanded to control the discharge to approximately 30 cfs. Removal of this plate is recommended with the increase in capacity of the downstream system in subbasin 214.2 where the 36-inch drain will be replaced with a new 42-inch drain through subbasin 214.2. A new 72-

inch drain from northwest corner of subbasin 214.2 to Alamo St. is recommended to replace the existing 60-inch drain.

Subbasins 227 and 228, which serve the south end of Yale Blvd., will continue to discharge into the Kirtland Channel system through the 60-inch Yale crossing near Alamo St. and Yale Blvd. The conditions in the Yale system will be relieved with the construction of Sunport Blvd. which diverts a significant amount of the storm water from the north and west side of the existing terminal. This diversion is discussed in and Section IV-E.

E. **SP Basin.** SP Basin contains Sunport Boulevard, George Road, a portion of University Blvd., and various commercial tracts of land between the airport and the South Diversion Channel. Sunport Blvd. and the relocation of George Road are currently under design with construction scheduled to start Fall, 1994.

Incorporated into the Sunport Blvd. project is a system which will collect storm water runoff from the new roadways from the terminal building to Interstate 25 and from surrounding property. The storm drainage system within the Sunport right of way was designed using the 100-year, 6-hour event. Sunport Blvd. will be an elevated roadway from the Interstate to a point approximately 700-feet east of University Blvd. The drainage system will be installed at an elevation which will enable a significant amount of the surrounding properties to discharge into it. Discharge from the new Sunport System will be through existing crossing points at Interstate 25.

A major diversion is proposed in the Sunport project at its eastern terminus with Yale Blvd. The existing system which transports storm flows from the west terminal area, subbasin 222 on Plate 1, will be severed by the new Sunport system. The diversion will amount to approximately 77 cfs and 3.2 a-f under present conditions. Storm runoff from the old airport terminal building and the USPS Air Mail Facility will be collected by the new system installed during the relocation of George Road.

A detention basin is proposed south of Sunport Blvd. and west of University Blvd. for the attenuation of peak flows discharged from subbasins 400, 401, 401.1, 401.2, 402, 401.1 and 402.3. The basin is required to protect the existing 3-36" RCP culverts under Interstate 25 west of subbasin 404. Without the detention basin, an additional crossing would be required at the Interstate. Discharge from the basin through a riser with an orifice plate will be controlled at approximately 85 cfs. Total pond storage volume to the emergency spillway crest is approximately 7.6 a-f. This facility was designed using the 100-year, 24-hour event per the City of Albuquerque and New Mexico State Highway and Transportation Department.

The Master Plan identifies the parcels of land adjacent to the relocated George Road as future airport support areas. The design of the drain systems in the Sunport project have taken into account the development of these properties. In addition, property within the SP Basin was considered fully developed for the design of the Sunport storm drainage system an as such, this study does not incorporate any additional development scenarios for analysis. Reference is made to the Sunport Boulevard Drainage Report, February, 1994, Molzen-Corbin & Associates, for additional and specific information regarding the design of the Sunport storm drainage system.

F. **WE Basin.** The WE Basin incorporates the area south and west of the existing terminal area with boundaries of Runway 12-30 to the south, Taxiway C to the east, the existing terminal building to the north and its discharge point to the South Diversion Channel to the west.

Changes to the facilities in this basin that are proposed under the Master Plan include the demolition of the old Air Freight Building located within subbasin 505. Extension of the existing storm drain system is recommended to follow the demolition of the building to relieve ponding occuring on the terminal apron between the B and C concourses. The extension of the drain (a 36-inch enlarged to a 42-inch connecting to the existing 60-inch) was planned during the Taxiway E Improvements project. A stub was installed at the 60-inch tee constructed near the northwest corner of Runway 8.

Downstream of the airfield, WE Basin discharges into a rip-rap lined channel and detention pond east of University Blvd. In order to maintain the future flows to near existing conditions, it is recommended that a restrictor plate be installed upstream. A 48-inch equivalent orifice installed at the 60" x 60" x 48" junction near the southwest corner of Runway 8 would help maintain existing conditions.

There is expected to be development west of University Blvd. in the Sunport Business Park, specifically in subbasins 514 and 515. Discussions between the developer and the City indicate that the natural arroyo that carries the basin discharge may be relocated to the south property boundary, approximately on the southern border of subbasins 514 and 515. The channel will run west until it nears Interstate 25 where it will be turned north to discharge through the Interstate at the 6'x 8' concrete box culvert. The future conditions modelling does not indicate the relocation of the arroyo and does not indicate that there will be any capacity problems at the Interstate crossing.

The Table IV-2 indicates the addition of the storm drain into subbasin 504 from the 60-inch drain. There are not any other improvements recommended as far as new drain systems in WE basin.

TABLE IV-2
Improvements to WE Basin

SUBBASIN NO.	PIPE DIAMETER	FLOW, 10 and 100-year, cfs	COMMENTS
504-505A	36-inch	39.2 (10) 43.4 (100)	Pipe to collect discharge from subbasin 504.
505A-60" tee	42-inch	57.6 (10) 59.0 (100)	Pipe collects NW corner of 505

G. **GA Basin.** The Master Plan identifies several parcels within the GA Basin which are recommended for development. With these parcels developed, the GA Basin should be

considered essentially fully developed. Subbasins 720, 721, and 724 contain the parcels which have been identified as airport support areas and will be typical of a light industrial or business park.

The capacity problem at the University crossing (60" and 36" CMP) with the 100-year event will be resolved with the proposed diversion of portions of the GA Basin to the M3 Basin discussed later in this section. Modelling indicates that the pond on the east side of University Blvd. will exceed the available capacity. During the 100-year event, a peak flow, 100-year, will reach approximately 320 cfs with a volume of 32.5 a-f resulting in over topping of University Blvd. With the recommended diversion upstream, the 100-year future flow is 299 cfs with a volume of approximately 28 a-f. This results in approximately two feet lowering of the maximum water surface.

There are no other improvements proposed for GA Basin in this study. As development is continued as recommended in the Master Plan, discharge to the University Arroyo should be metered to maintain the existing conditions. Construction of a detention pond near the discharges of the three branches of GA Basin with an outflow controlled to existing conditions would also help maintain the crossing under University Blvd.

H. **M3 Basin (SE35, SW35, W3, SE3).** Under the future conditions, SE35, SW35, W3, SE3 and M3 Basins have been consolidated to form one basin with a detention pond at the discharge point off of the airfield. The intent of the consolidation is to provide a common controlled discharge point off the southeast side of upgraded Runway 3-21, new Air Cargo Freight Facility, expansion on the existing Air Freight Ramp, and to relieve GA Basin by diverting flow from the north and east sides of the General Aviation Area. Consolidation of five discharge points at natural arroyos into one discharge from a detention pond with a controlled outlet will also allow more efficient storm water monitoring if required under AIA's EPA NPDES storm water permit.

The changes proposed in the Master Plan for this part of the airport will fully develop the basins. Additional capacity is required in the storm drains as well as additional trunk lines required to consolidate the flow. It anticipated that the majority of the storm drains will be constructed as part of the Runway 3-21 Upgrade project. An exception may be the new trunk line that will collect discharge from the proposed Air Cargo Facility. Installation of this line could occur when the new apron is constructed delaying the diversion of existing SE35 and approximately half of SW35 flows to the detention pond. Existing drains have the capacity for the additional flows which will result with construction of the northeastern reach of the southern parallel taxiway for Runway 3-21 and the extension of Taxiway D, south of Taxiway E.

Table IV-3 contains the new drains that would serve a consolidated M3 Basin as shown on Plate 3. All of the collector lines are not included in Table IV-XX. In most cases, these lines conceptually are 24 and 30-inch drains. The middle branch depends primarily on existing lines with the exception of new collector lines in subbasins 902 and 901.1

TABLE IV-3
Improvements to M3 Basin

SUBBASIN NO.	PIPE DIAMETER	FLOWS, 10 and 100 year, cfs	COMMENTS
609-608	60-inch	56.3 (10) 82.9 (100)	Drain for SE35
608-607	66-inch	113.4 (10) 171.6 (100)	Across new taxiway. Carries flow to new Air Cargo Apron
607-616	72-inch	442 (10) 676 (100)	GA diversion, SE35 diversion, most of 616
616-Pond	84-inch	442 (10) 700 (100)	East branch discharge to pond
1104-1103	30-inch	19.4 (10)	Collector in 1104 and 1103
1103-pond	60-inch	144 (10) 179 (100)	Discharge from west branch to pond

There are three branches in M3 Basin. SE35 and SW35 along with diversion from a portion of the north side of GA Basin make up the east branch. The middle branch contains the diversion from the southeastern portion of GA Basin as well as M3 Basin and a portion of W3 Basin. The south branch contains SE3 and a portion of W3.

The location of the detention pond is conceptually selected for its central location and because it is currently a major discharge point for existing flows from M3 and W3 Basins. The detention pond is sized to accept 100-year, 6-hour flows and volumes. The volume of the pond should accept approximately 75 a-f of storm water at a peak flow of approximately 1000 cfs. Discharge from the detention pond will be controlled at approximately 95.2 cfs with a volume of approximately 83 a-f. This discharge is equivalent to the 10-year flow for the existing outfalls combined. Consolidation of five discharge points in the existing basins will also allow more efficient storm water discharge monitoring.

I. **UE, SW3 Basin.** The UE and SW3 Basins within airport property lie off of the airfield on steep terrain. Future development is restricted in these basins due to the terrain and proximity to the protected zones for Runway 3-21 after it has been upgraded to air carrier status. UE Basin does contain developable property which is owned by UNM and identified as subbasins 1201, 1202 and 1203 on Plate 1. Subbasins are addressed as Basin C1 in Drainage Report and Conceptual Grading Plan for UNM Business Park, Jan. 1993, Bohannon-Huston Inc. This report indicates that the existing 48-inch crossing under University Blvd. has sufficient capacity to handle developed flows from this basin.

Subbasin 1201 is identified as Basins A-4, A-3 and C-2 in the Bohannon-Huston report. Flows from these A-3 and A-4, the most northern portion of subbasin 1201, will be carried by a storm drain to University Blvd. then to a detention basin located to the northwest of the existing 48-

inch crossing. The drain system will designed to accommodate the 100-year runoff. Discharge from the pond will be controlled to a level less than existing. Pond capacity is proposed at 4 a-f.

J. **Middle, Eend, Air Guard, Far East End Basins.** These basins, which lie to the east of Runway 17-35 and discharge to the south through KAFB property, were analyzed under the Taxiway E and A improvement projects in order that systems which would handle fully developed flows from KAFB and the airfield were constructed while Runway 8-26, Taxiway A and Taxiway E were closed. This study does not address additional development or future conditions for these basins due to the capacity of the upgraded systems constructed. Section III, Existing Conditions, contains as-constructed information on these basins. The existing systems are considered adequate to accommodate foreseeable future conditions.

SECTION V

COST ESTIMATES

The attached conceptual cost estimates have been developed for the improvements outlined in Section III and IV of this study. It is understood that priorities and scheduling of projects will play an important role in the construction of the improvements. Construction of upgrades to Runway 3-21 in the near future could incorporate many of the improvements recommended for the M3 Basin. Furthermore, the Master Plan CIP program has outlined a schedule for the other capital projects recommended for Albuquerque International Airport including the expansion of the terminal and apron with the closure of Runway 17-35. This document should be viewed in tandem with the Master Plan CIP to schedule construction of the drainage system improvements.

THIS DOCUMENT IS BEING RE-RECORDED AS A SEPARATE DOCUMENT

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DRAINAGE EASEMENT DECLARATION

This Drainage Easement Declaration (this "Declaration") is executed as of this 20th day of October, 2000 by SCHWARTZMAN, INC., a corporation of the State of New Mexico, ("Schwartzman"):

1. Recitals. Schwartzman makes the following recitals with respect to this Declaration:

a. Schwartzman owns real property lying within Section 5, Township 9 North, Range 3 East N.M.P.M. ("Schwartzman Industrial Park"), more particularly described in Exhibit A attached hereto ("Tract 4").

b. Schwartzman wishes to grant a drainage easement ("Drainage Easement"), particularly described in Exhibit A, over and across Tract 4 to benefit upstream property owners, including the City of Albuquerque, New Mexico, a municipal corporation, the owner of property described in Exhibit B attached hereto ("Tract 2").

c. Schwartzman desires to define its rights and obligations with respect to the Drainage Easement in this Declaration.

d. Schwartzman agrees that this Declaration will bind and burden its successors and assigns in ownership of Tract 4 and the Drainage Easement will touch and run with Tract 4.

2. Use of Easement. Schwartzman agrees that the Drainage Easement shall be used to convey storm water drainage from Tract 2 and other upstream property.

3. Term. The term of the Drainage Easement will begin on the date that this Declaration is recorded in the records of the Bernalillo County, New Mexico County Clerk's office, and will continue until termination of the Drainage Easement pursuant to paragraph 9 below.

4. Maintenance of Drainage Easement. The City of Albuquerque and its assigns or designees will be responsible for maintaining the Drainage Easement.

5. Limitations on Easement and Enforcement. The Drainage Easement and the enforcement of this Declaration are subject to the following limitations:

a. The Drainage Easement and the use of the Drainage Easement will comply with all applicable laws, ordinances, regulations, orders or rulings of any federal, state, county or municipal government, court or public authority, having jurisdiction over (I) Tract 4, and (ii) Tract 2 and the Drainage Easement.

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b. This Declaration does not limit and will not be construed as limiting the rights of an owner of either Tract 4, Tract 2, or the Drainage Easement to grant third partles further non-exclusive or exclusive easements encumbering either Tract 4, Tract 2, or the Drainage Easement; provided, however, that such further easements shall not interfere with the use of the Drainage Easement.

6. No Dedication. Nothing provided in this Declaration or in the Drainage Easement will be deemed a gift or dedication of the Drainage Easement or any portion of the Drainage Easement to, or will confer any rights on, the public in general or the City of Albuquerque, New Mexico except as specifically provided in this Declaration.

7. Arbitration. Any controversy or claim arising out of or relating to this Declaration or the breach hereof, will be settled by arbitration in accordance with the Commercial Arbitration Rules of the American Arbitration Association ("Arbitration Rules"). The arbitration will be conducted in Bernalllio County, New Mexico before a single arbitrator appointed by the American Arbitration Association In accordance with the Arbitration Rules.

8. Declaration Complete. This Declaration embodies and constitutes the entire declaration with respect to the Drainage Easement, and all prior agreements, understandings, representations and statements by Schwartzman with respect to the Drainage Easement, oral or written, are merged into this Declaration.

9. Termination. The Drainage Easement and all rights and responsibilities conferred by this Declaration will automatically terminate (i) upon the approval by the City of Albuquerque of a site drainage plan for Tract 2 which does not use the Drainage Easement to service Tract 2 and the completion of all improvements contemplated by such drainage plan, or (ii) upon the recording by the Owner of Tract 2 of any document relinquishing its rights to the Drainage Easement as granted by this Declaration.

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10. Notice. Any notice, demand, approval or disapproval, consent or submission for approval or consent permitted or required under this Declaration (collectively, any "Notice") will be in writing, and any such Notlce will be sent by registered or certified mail, return receipt requested, postage prepaid, addressed as follows:

TO: Schwartzman, Inc.
c/o Gary Schwartzman
1513 California NE
Albuquerque, NM 87110

AND TO: City of Albuquerque
Aviation Department
Albuquerque International Sunport
P.O. Box 9948
2200 Sunport Blvd. SE 3rd Floor
Albuquerque, NM 87119-1048

or the same may be delivered by messenger at the same addresses.

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11. Applicable Law. The Drainage Easement Declaration will be governed by and construed under and in accordance with the laws of the State of New Mexico.

12. Severability. If any provision of this Declaration, for any reason, is held to be in violation of any applicable law, then the invalidity of such specific provisions herein will not be held to invalidate any other provision herein and the remainder of this Declaration will remain in full force and effect.

SCHWARTZMAN, INC.

By: _____

Its: President

Date: 10/20/2000

APPROVED AS TO FORM ONLY:

CITY OF ALBUQUERQUE

By: _____ for
Jay J. Czár, Director of Aviation

Date: 11-3-00

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ACKNOWLEDGMENTS

STATE OF NEW MEXICO)
) ss.
COUNTY OF BERNALILLO)

The foregoing instrument was acknowledged before me this 26th day of October, 2000, by Alfred Vollum, President for Schwartzman, Inc., a corporation of the State of New Mexico, on behalf of the corporation.

Notary Public

My commission expires:

10/11/2004

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SKETCH AND DESCRIPTION SHOWING

STORM DRAINAGE EASEMENT

TRACT 4, SCHWARTZMAN INDUSTRIAL PARK
LOCATED WITHIN SECTION 5, TOWNSHIP 9 NORTH, RANGE 3 WEST
PORTIONS TRACT 1-A, M.R.G.C.D. MAP 50
AND TRACT 24-A-1, M.R.G.C.D. MAP 45
BERNALILLO COUNTY, NEW MEXICO
JANUARY, 2000

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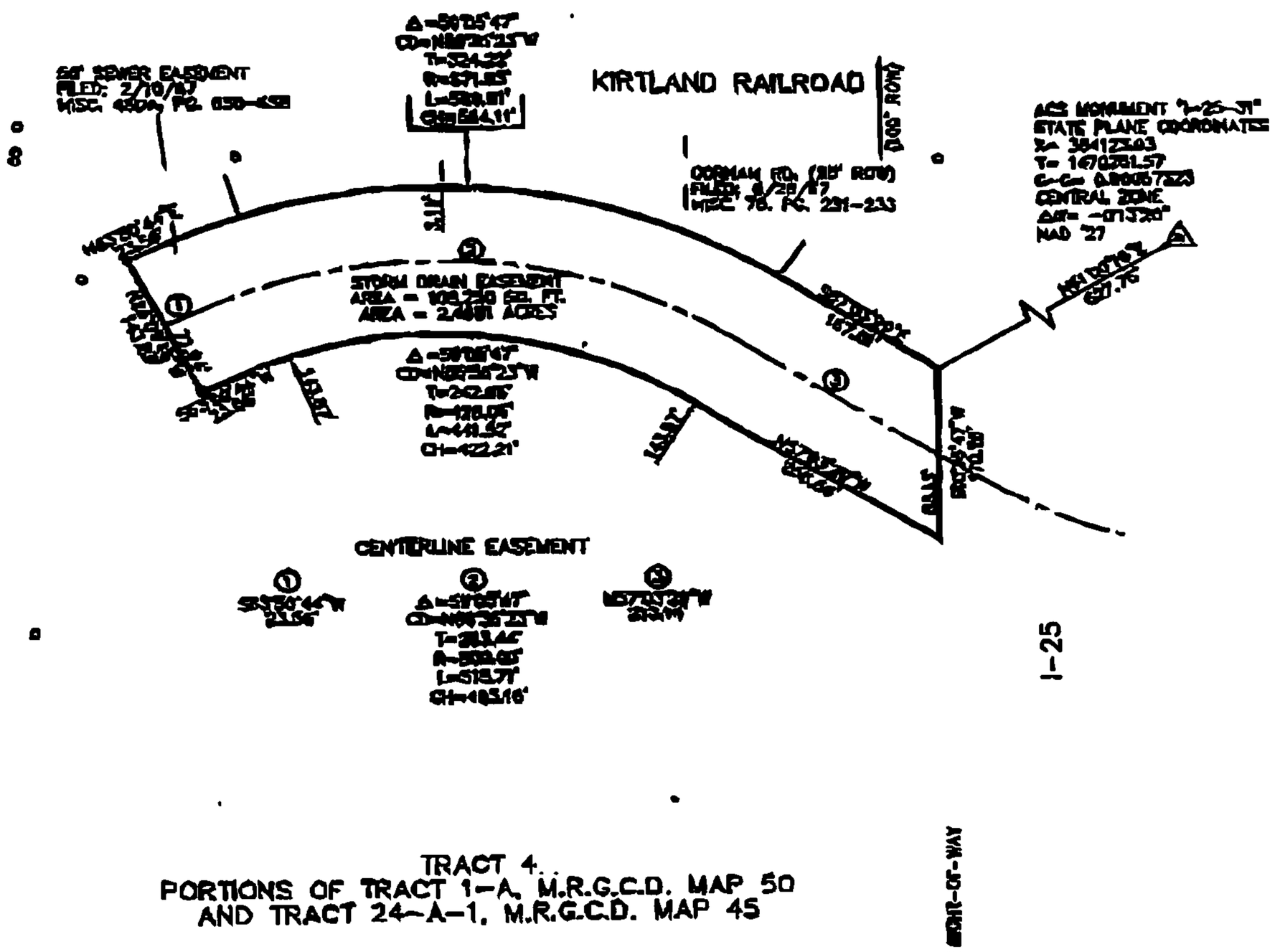

TRACT 4
PORTIONS OF TRACT 1-A, M.R.G.C.D. MAP 50
AND TRACT 24-A-1, M.R.G.C.D. MAP 45

○ FOUND #5 REBAR W/PLASTIC CAP "TIERRA P.S. 3516"

URS Greiner Woodward Clyde

5971 JEFFERSON BLVD. N.E. ALBUQUERQUE, NEW MEXICO 87109
(505) 345-3999 FAX: (505) 345-8395

EXHIBIT "A"
(page 1 of 2)

SKETCH AND DESCRIPTION SHOWING
STORM DRAINAGE EASEMENT
LOCATED WITHIN SECTION 5 TOWNSHIP 9 NORTH, RANGE 3 EAST, PORTIONS OF TRACT 1-A, M.R.G.C.D. MAP 50 AND TRACT 24-A-1, M.R.G.C.D. MAP 45 ALBUQUERQUE, NEW MEXICO JANUARY, 2000

A PARCEL OF LAND SITUATED WITHIN PORTION TRACT 1-A, M.R.G.C.D. MAP 50 AND TRACT 24-A-1, M.R.G.C.D. MAP 45, IN SECTION 5, TOWNSHIP 9 NORTH, RANGE 3 EAST, NEW MEXICO PRINCIPAL MERIDIAN WITHIN THE CITY OF ALBUQUERQUE, BERNALILLO COUNTY, NEW MEXICO. BEING THE SAME AS SHOWN AND DESIGNED ON THE PLAT FILED FOR RECORD IN THE COUNTY CLERK OF BERNALILLO COUNTY, NEW MEXICO ON DECEMBER 16, 1992 BK. 92-30, PGS 696-712 AND BEING MORE PARTICULARY DESCRIBED AS FOLLOWS:

BEGINNING AT THE NORTHEAST CORNER OF SAID TRACT, A POINT ON THE WESTERLY RIGHT-OF-WAY LINE OF INTERSTATE 25, FROM WHICH THE CITY OF ALBUQUERQUE SURVEY CONTROL MONUMENT "1-25-31" BEARS N61°00'16"E, 627.76 FEET (NAD '27); THENCE,

S0°35'47"W, 170.26 FEET, ON THE WESTERLY RIGHT-OF-WAY INTERSTATE 25; THENCE,

N57°03'29"W, 258.56 FEET, TO A POINT ON THE SOUTH LINE OF THE EASEMENT HEREIN DESCRIBED; THENCE,

441.52 FEET ALONG A CURVE TO THE LEFT WHOSE RADIUS IS 428.06 FEET AND WHOSE CHORD BEARS N86°36'23"W, 422.21 FEET THROUGH A CENTRAL ANGLE OF 59°05'47"; THENCE,

S63°50'44"W, 23.56 FEET, TO THE SOUTHWEST CORNER OF THE EASEMENT HEREIN DESCRIBED, ALSO BEING THE EASTERLY RIGHT-OF-WAY OF THE A.M.A.F.C.A. SOUTH DIVERSION CHANNEL; THENCE,

N26°09'16"W, 143.87 FEET, TO THE NORTHWEST CORNER OF THE EASEMENT HEREIN DESCRIBED; THENCE,

N63°50'44"E, 23.58 FEET, TO A POINT ON THE NORTHERN LINE OF THE EASEMENT HEREIN DESCRIBED; THENCE,

589.91 FEET ALONG A CURVE TO THE RIGHT WHOSE RADIUS IS 571.93 FEET AND WHOSE CHORD BEARS N86°36'23"W, 564.11 FEET THROUGH A CENTRAL ANGLE OF 59°05'47"; THENCE,

S57°03'29"E, 167.61 FEET, ON THE NORTHERN LINE OF THE EASEMENT HEREIN DESCRIBED, TO THE POINT OF BEGINNING AND CONTAINING 108,250.24 SQUARE FEET (2.4851 ARCES).

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5442792
Page: 6 of 7
12/05/2000 03:07P
Bk-A13 Pg-515
Judy D. Woodward Bern. Co. DEC R 19.00

I, WILLIAM D. HOUSTON, A PROFESSIONAL SURVEYOR REGISTERED IN THE STATE OF NEW MEXICO HEREBY CERIFY THAT THIS SKETCH AND DESCRIPTION WAS PREPARED FROM FIELD WORK PERFORMED BY ME OR UNDER MY DIRECT SUPERVISION, MEETS THE MINIMUM STANDARDS FOR SURVEYS IN NEW MEXICO AND IS TRUE TO THE BEST OF MY KNOWLEDGE AND BELIEF.

William D. Houston, PS #6685 Date

2000120290
5442923
12/04/2000 03:17P
Judy D. Woodward Bern. Co. TERM R 37.00 Bk-A12 Pg-9843

EXHIBIT "A"
(page 2 of 2)

DRAINAGE EASEMENT AGREEMENT

This Drainage Easement Agreement (this "Easement Agreement") is executed as of this 5th day of January, 2000 by THE REGENTS OF THE UNIVERSITY OF NEW MEXICO, a body corporate of the State of New Mexico, organized pursuant to §§ 21-71-1, et seq. N.M.S.A. 1978 ("UNM) and the City of Albuquerque, a municipal corporation (the "City"):

1. Recitals. UNM and the City make the following recitals with respect to this Easement Agreement:

a. UNM owns real property lying within Section 4, Township 9 North, Range 3 East N.M.P.M. ("UNM South Golf Course"), more particularly described in part in Exhibit A attached hereto and incorporated herein ("Tract 1"). UNM, and its successors and assigns in ownership of Tract 1, will hereinafter be referred to as the "Owner of Tract 1".

b. UNM wishes to grant a non-exclusive drainage easement ("Drainage Easement"), more particulary described in Exhibit A attached hereto and incorporated herein, over and across Tract 1 to benefit certain property belonging to the City, an upstream property owner , which property is more particularly described in Exhibit B, attached hereto and incorporated herein ("Tract 2").

c. UNM and the City desire to define their rights and obligations with respect to the Drainage Easement .

d. UNM and the City agree that this Easement Agreement will bind and burden their successors and assigns in ownership of Tract 1 and Tract 2 and the Drainage Easement will touch and run with Tract 1.

2. Grant of Drainage Easement. UNM grants to the City the non-exclusive Drainage Easement described in Exhibit A subject to the terms and conditions set forth in this Easement Agreement.

3. Use of Drainage Easement. UNM and the City agree that the Drainage Easement maybe used, in part, to convey storm water drainage from Tract 2 with the understanding that historical storm drainage flows from Tract 2 on and over the Drainage Easement shall not be increased. It is understood that UNM is and shall be entitled to discharge storm waters from Tract 1 into the Drainage Easement and onto and over any improvements constructed and maintained by the City or others within the Drainage Easement.

4. Term. The term of the Easement Agreement will begin on the date that this Easement Agreement is recorded in Bernalillo County, New Mexico, and will continue until termination of the Easement Agreement pursuant to paragraph 10 below.

5. Maintenance and Improvements to Drainage Easement. The City and its assigns or designees will be responsible for improving and maintaining the Drainage Easement. The

Mary Herrera Bern. Co. AGRE R 21.00
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Page: 1 of 8
01/22/2001 11:35A
Bk-A14 Pg-4839

City agrees to consult with and obtain the written approval of management of the UNM South Golf Course prior to undertaking improvements within the Drainage Easement. The City shall restore the Drainage Easement to its original condition if the Drainage Easement is terminated or abandoned unless UNM requests, in writing, that the improvements within the Drainage Easement be allowed to remain in place in which event the improvements shall remain in place.

6. Limitations on Easement and Enforcement. The Drainage Easement and the enforcement of this Easement Agreement are subject to the following limitations:

a. The Drainage Easement and the use of the Drainage Easement will comply with all applicable laws, ordinances, regulations, orders or rulings of any federal, state, county or municipal government, court or public authority, having jurisdiction over (i) Tract 1, (ii) Tract 2 and the Drainage Easement, or (iii) any improvements or activities on Tract 1, Tract 2, or the Drainage Easement.

b. This Declaration does not limit and will not be construed as limiting the rights of the owner of Tract 1 to grant third parties further non-exclusive easements encumbering Tract 1, provided, however, that such further easements shall not interfere with the use of the Drainage Easement.

7. Indemnification. The City of Albuquerque agrees to indemnify and hold UNM harmless from and against all lawsuits, actions, claims, demands, penalties, fines, liabilities, settlements, damages, costs, and expenses brought against UNM arising out of or related to the existence or use of the Drainage Easement.

To the extent, if at all, Section 56-7-1, NMSA 1978, is applicable to this paragraph, this indemnification shall not extend to or be construed to require the City of Albuquerque to defend, indemnify and hold harmless UNM, its officers, and employees from and against all liability, claims, damages, losses or expenses, including attorneys fees, arising out of bodily injury to persons or damage to property caused by, resulting from, or arising out of (1) the preparation or approval of maps, drawings, opinions, reports, surveys, change orders, designs or specifications by UNM or the agents or employees or officers of UNM or (2) the giving or failure to give directions or instructions by UNM or agents or employees or officers of UNM where such giving or failure to give directions or instructions is the primary cause of bodily injury to persons or damage to property.

8. No Dedication. Nothing provided in this Easement Agreement or in the Drainage Easement will be deemed a gift or dedication of the Drainage Easement or any portion of the Drainage Easement to, or will confer any rights on, the public in general or the City , except as specifically provided in this Easement Agreement.

9. Arbitration. Any controversy or claim arising out of or relating to this Easement Agreement or the breach hereof, will be settled by arbitration in accordance with the Commercial

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Page: 2 of 8
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Mary Herrera Bern. Co. AGRE R 21.00

Arbitration Rules of the American Arbitration Association ("Arbitration Rules"). The arbitration will be conducted in Bernalillo County, New Mexico before a single arbitrator appointed by the American Arbitration Association in accordance with the Arbitration Rules.

10. Easement Agreement Complete. This Easement Agreement embodies and constitutes the entire declaration with respect to the Drainage Easement, and all prior agreements, understandings, representations and statements by UNM and the City with respect to the Drainage Easement, oral or written, are merged into this Easement Agreement.

11. Termination. The Drainage Easement and all rights and responsibilities conferred by this Easement Agreement will automatically terminate (i) upon the approval by the City of a site drainage plan for Tract 2 which does not use the Drainage Easement to service Tract 2 and the completion of all improvements contemplated by such drainage plan, or (ii) upon the recording by the Owner of Tract 2 of any document relinquishing its rights to the Drainage Easement as granted by this Easement Agreement, or (iii) or upon abandonment of the Drainage Easement by the City .

12. Notice. Any notice, demand, approval or disapproval, consent or submission for approval or consent permitted or required under this Easement Agreement (collectively, any "Notice") will be in writing, and any such Notice will be sent by registered or certified mail, return receipt requested, postage prepaid, addressed as follows:

TO: University of New Mexico
Real Estate Department
University of New Mexico
252 Scholes Hall
Albuquerque, NM 87131

AND TO: City of Albuquerque, Aviation Dept.
Albuquerque International Sunport
P.O. Box 9948
2200 Sunport Blvd. SE 3rd Floor
Albuquerque, NM 87119-1048

or the same may be delivered by messenger at the same addresses.

13. Applicable Law. This Easement Agreement will be governed by and construed under and in accordance with the laws of the State of New Mexico.

14. Severability. If any provision of this Easement Agreement, for any reason, is held to be in violation of any applicable law, then the invalidity of such specific provisions herein will not be held to invalidate any other provision herein and the remainder of this Easement Agreement will remain in full force and effect.

15. Date of Easement Agreement. The date of this Easement Agreement shall be the date of the last signature affixed hereto.

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Mary Herrera Bern. Co. AGRE R 21.00

THE REGENTS OF THE UNIVERSITY OF NEW MEXICO
a body corporate of the State of New Mexico

By: _____ 1/12/01
Date

Its: JULIE C. WEAKS
VICE PRESIDENT
FOR BUSINESS & FINANCE

APPROVED ACCEPTED AND AGREED: CITY OF ALBUQUERQUE
a municipal corporation of the State of New Mexico

By: _____ 1/19/01
Date

Lawrence Rael
Chief Administrative Officer

Jay J. Czar, Director of Aviation

APPROVED AS TO FORM ONLY:

By: _____
Assistant City Attorney

By: _____
University Counsel for UNM

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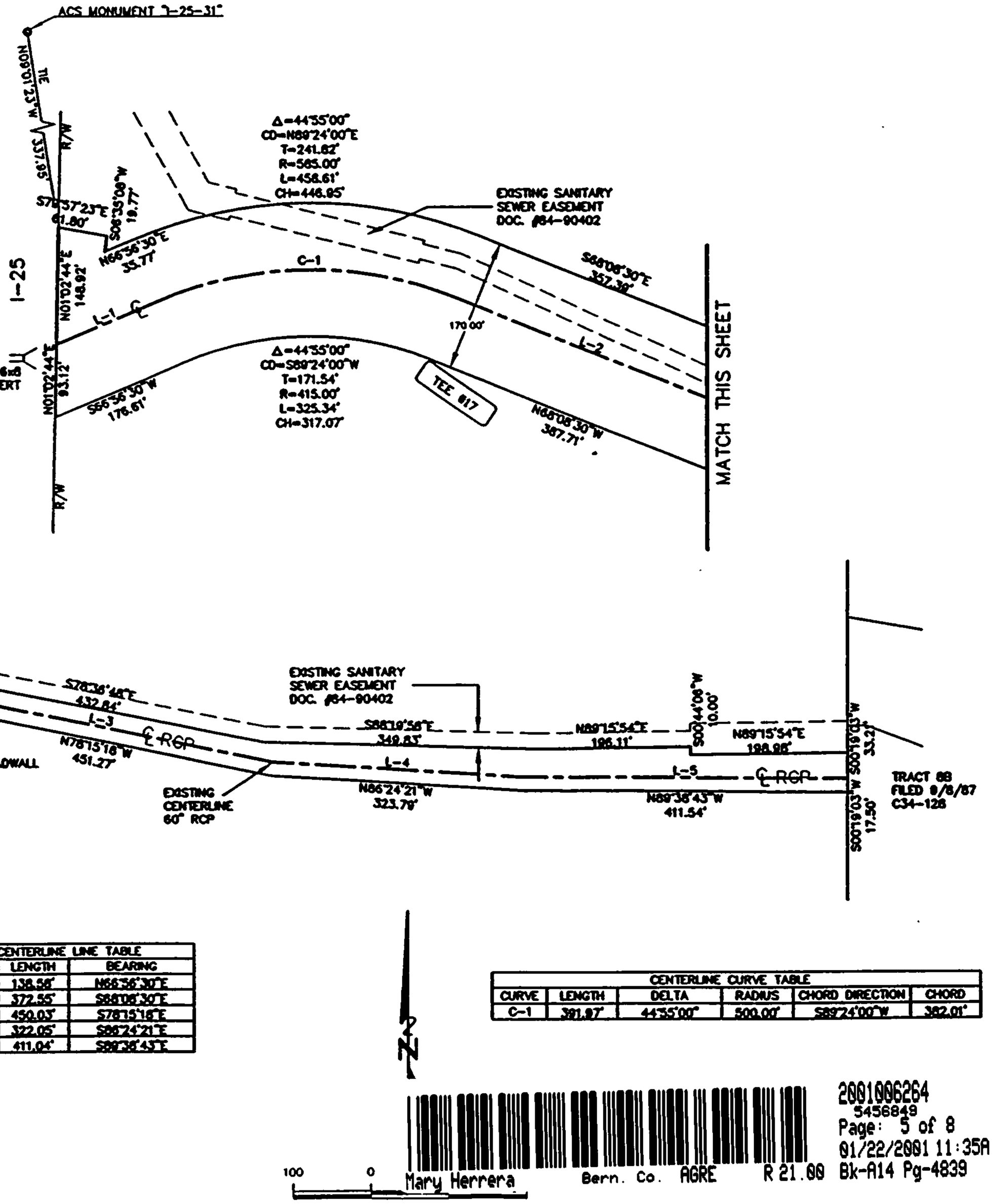
Mary Herrera Bern. Co. AGRE R 21.00

S RCH AND DESCRIPTION SHC NG

STORM DRAINAGE EASEMENT

LOCATED WITHIN SECTION 4, TOWNSHIP 9 NORTH, RANGE 3 WEST IN THE GOLF COURSE OF THE UNIVERSITY OF NEW MEXICO BERNALILLO COUNTY, NEW MEXICO AUGUST, 2000



CENTERLINE LINE TABLE		
LINE	LENGTH	BEARING
L-1	138.56'	N66°56'30"E
L-2	372.55'	S88°08'30"E
L-3	450.03'	S78°15'18"E
L-4	322.05'	S86°24'21"E
L-5	411.04'	S89°38'43"E

CENTERLINE CURVE TABLE					
CURVE	LENGTH	DELTA	RADIUS	CHORD DIRECTION	CHORD
C-1	391.87'	44°35'00"	500.00'	S89°24'00"W	382.01'

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Mary Herrera Bern. Co. AGRE R 21.00

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EXHIBIT "A"
(page 1 of 3)

URS Greiner Woodward Clyde
5971 JEFFERSON BLVD. N.E. ALBUQUERQUE, NEW MEXICO 87109
(505) 345-3999 FAX: (505) 345-8393

P:\CADD\SURV\DWG\GOLF COURSE EASE\GOLFCRS.DWG 01/03/00 10:29:41 AM MST

SKETCH AND DESCRIPTION SHOWING

STORM DRAINAGE EASEMENT

LOCATED WITHIN SECTION 4 TOWNSHIP 9 NORTH, RANGE 3 EAST
IN THE GOLF COURSE OF THE UNIVERSITY OF NEW MEXICO
ALBUQUERQUE, NEW MEXICO
AUGUST, 2000

A PARCEL OF LAND SITUATED WITHIN PORTION TRACT "1-A, M.R.G.C.D. MAP 50 AND TRACT"24-A-1, M.R.G.C.D. MAP 45, IN SECTION 5, TOWNSHIP 9 NORTH, RANGE 3 EAST, NEW MEXICO PRINCIPAL MERIDIAN WITHIN THE CITY OF ALBUQUERQUE, BERNALILLO COUNTY, NEW MEXICO. BEING THE SAME AS SHOWN AND DESIGNED ON THE PLAT FILED FOR RECORD IN THE COUNTY CLERK OF BERNALILLO COUNTY, NEW MEXICO ON DECEMBER 16, 1992 BK. 92-30, PGS 696-712 AND BEING MORE PARTICULARY DESCRIBED AS FOLLOWS:

BEGINNING AT THE NORTHWEST CORNER OF SAID TRACT, A POINT ON THE EASTERLY RIGHT-OF-WAY LINE OF INTERSTATE 25, FROM WHICH THE CITY OF ALBUQUERQUE SURVEY CONTROL MONUMENT "1-25-31" BEARS N61°00'16"E, 627.76 FEET (NAD '27); THENCE,

S79°57'23"E, 61.80 FEET, TO A POINT ON THE NORTH LINE OF THE EASEMENT HEREIN DESCRIBED: THENCE,

S6°35'08"W, 19.77 FEET,, TO A POINT ON THE NORTH LINE OF THE EASEMENT HEREIN DESCRIBED; THENCE,

N66°56'30"E, 35.77 FEET; THENCE,

458.61 FEET ALONG A CURVE TO THE LEFT WHOSE RADIUS IS 585.00 FEET AND WHOSE CHORD BEARS N89°24'00"E, 446.95 FEET THROUGH A CENTRAL ANGLE OF 44°55'00"; THENCE,

S68°08'30"E, 357.39 FEET; THENCE,

S11°44'42"W, 66.49 FEET,; THENCE,

S78°36'48"E, 432.84 FEET, ALONG THE SOUTHERN LINE OF A SANITARY SEWER EASEMENT (FILED 11/28/84, MISC. 177A; PG. 681-683); THENCE,

S88°19'58"E, 349.83 FEET, ALONG THE SOUTHERN LINE OF A SANITARY SEWER EASEMENT; THENCE,

N89°15'54"E, 196.11 FEET, ALONG THE SOUTHERN LINE OF A SANITARY SEWER EASEMENT; THENCE,

S0°44'06"W, 10.00 FEET, ALONG THE SOUTHERN LINE OF A SANITARY SEWER EASEMENT; THENCE,

N89°15'54"E, 198.98 FEET, TO THE NORTHEAST CORNER OF THE EASEMENT HEREIN DESCRIBED AND ON THE WESTERLY PROPERTY LINE OF TRACT 8B OF THE AIRPORT TECHNICAL CENTER (FILED 9/8/87, C34-128); THENCE,

S0°19'03"W, 33.21 FEET, TO THE CENTERLINE OF A 60" RCP; THENCE,

S0°19'03"W, 17.50 FEET, TO THE SOUTHEAST CORNER OF THE EASEMENT HEREIN DESCRIBED; THENCE,

N89°38'43"W, 411.54 FEET, TO THE SOUTHEAST CORNER OF THE EASEMENT HEREIN DESCRIBED; THENCE,

N86°24'21"W, 323.79 FEET, TO THE SOUTHEAST CORNER OF THE EASEMENT HEREIN DESCRIBED; THENCE,

N78°15'18"W, 451.27 FEET, TO THE SOUTHEAST CORNER OF THE EASEMENT HEREIN DESCRIBED; THENCE,

S11°44'42"W, 68.84 FEET, TO THE SOUTHEAST CORNER OF THE EASEMENT HEREIN DESCRIBED; THENCE,

N68°08'30"W, 387.71 FEET, TO THE SOUTHEAST CORNER OF THE EASEMENT HEREIN DESCRIBED; THENCE,

325.34 FEET ALONG A CURVE TO THE RIGHT WHOSE RADIUS IS 415.00 FEET AND WHOSE CHORD BEARS S89°24'00"W, 317.07 FEET THROUGH A CENTRAL ANGLE OF 44°55'00"; THENCE,

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Mary Herrera Bern. Co. AGRE R 21.00

S66°56'30"W, 176.61 FEET, TO THE SOUTHWEST CORNER OF THE EASEMENT HEREIN DESCRIBED, ALSO BEING THE EASTERLY RIGHT-OF-WAY OF INTERSTATE 25; THENCE,

N1°02'44"E, 93.12 FEET, TO THE CENTERLINE OF THE EASEMENT HEREIN DESCRIBED; THENCE,

N1°02'44"E, 148.92 FEET, ON THE WESTERN LINE OF THE EASEMENT HEREIN DESCRIBED, TO THE POINT OF BEGINNING AND CONTAINING 210,074.91 SQUARE FEET (4.8227 ARCES).

I, SANTIAGO ROMERO Jr., A PROFESSIONAL SURVEYOR REGISTERED IN THE STATE OF NEW MEXICO HEREBY CERIFY THAT THIS SKETCH AND DESCRIPTION WAS PREPARED FROM FIELD WORK PERFORMED BY ME OR UNDER MY DIRECT SUPERVISION, MEETS THE MINIMUM STANDARDS FOR SURVEYS IN NEW MEXICO AND IS TRUE TO THE BEST OF MY KNOWLEDGE AND BELIEF.

Santiago Romero Jr., NMPLS 7924 — Date: AUG. 1, 2000

SANTIAGO ROMERO, JR. NEW MEXICO 7924 PROFESSIONAL SURVEYOR

Mary Herrera Bern. Co. AGRE R 21.00

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A certain parcel of land situate within the Northeast Quarter of Section 4, Township 9 North, Range 3 East, N.M.P.M., Bernalillo County, New Mexico and being more particularly described as follows:

Beginning at the southeast corner of said parcel, being the same point as the Northeast corner of Tract 1, AIRPORT TECHNICAL CENTER, filed July 1, 1998 in Bk. 98C, Pg. 178, whence A.C.S. Station 1-N16 bears first S 00°01'34" E 445.10 feet, then S 48°36'21" E, 1452.770 feet distant; thence, from said point of beginning

S 89°16'26" W, 489.32 feet: thence

S 00°01'34" E, 445.10 feet; thence,

S 89°16'26" W, 488.03 feet to the Southwest corner of said tract, a point on the East right-of-way of University Boulevard S.E.; thence,

N 00°07'04" E, 1909.51 feet along said right-of-way to the Northwest corner of said tract: thence, leaving said right-of-way

S 89°39'19" E, 972.50 feet to the Northeast corner of said tract; thence,

S00°01'34" E, 1446.17 feet to the point of beginning.

Mary Herrera Bern. Co. AGRE R 21.00
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1-16/D024

City of Albuquerque

P.O. BOX 1293 ALBUQUERQUE, NEW MEXICO 87103

February 6, 1990

RECEIVED
FEB 16 1990
HYDROLOGY SECTION

PROJECT ACCEPTANCE LETTER

Michael S. Rich Contractors, Inc.
P.O. Box 90727
Albuquerque, NM 87199

RE: VEHICLE STORAGE BUILDING, PROJECT NO. 3562

Dear Mr. Rich:

The above referenced project has been completed according to the plans and specifications.

The project consisted of the general construction and related sitework of 12,796 square feet pre-engineered metal building, concrete foundations, concrete masonary, mechanical, electrical, and asphalt paving work. This portion of the Project has been accepted as a whole and the contractual correction period for this portion of the Project began **October 11, 1989.**

The project also consisted of the construction of a steel beacon tower approximately fifty feet (50') high located south of the Cutter Hanger. This portion of the Project has been accepted as a whole and the contractual correction period began on **December 11, 1989.**

The correction period on both portions of this project is for one (1) year.

Sincerely,

Brian L. Speicher, P.E.
Chief Construction Engineer
Design/Construction Division
Engineering Group
Public Works Department

CONCURRENCE:

Tyler Mason, Architect/Engineer
Burns/Peters Group, P.A.

BLS:tp
INP138480

FILE COPY

City of Albuquerque

P.O. BOX 1293 ALBUQUERQUE, NEW MEXICO 87103

KEN SCHULTZ
MAYOR

CLARENCE V. LITHGOW
CHIEF
ADMINISTRATIVE OFFICER

DAN WEAKS
DEPUTY CAO
PUBLIC SERVICES

FRED E. MONDRAGON
DEPUTY CAO
DEVELOPMENT & ENTERPRISE SERVICES

February 28, 1989

Philip Clark, P.E.
BPLW Engineers/Architects
2400 Louisiana Boulevard, NE Suite 400
Albuquerque, New Mexico 87110

RE: AMENDED GRADING & DRAINAGE PLAN FOR THE ALBUQUERQUE MAINTENANCE FACILITY ADDITION, SUBMITTED FEBRUARY 27, 1989, FOR BUILDING PERMIT APPROVAL ~~(N-15/D3)~~ M-16/D024

Dear Mr. Clark:

Your submittal, referred to above, with a revision #3 date of 2-27-89, is approved for Building Permit sign-off by the Hydrology Section. Please include the approved sheet C2 in the construction sets routed for sign-off.

If you have any questions, please call me at 768-2650.

Cordially,

Stuart Reeder

G. Stuart Reeder, P.E.
C.E./Hydrology Section

GSR
(WP+868)

FILE COPY

City of Albuquerque

P.O. BOX 1293 ALBUQUERQUE, NEW MEXICO 87103

KEN SCHULTZ
MAYOR

October 4, 1988

Phillip Clark
BPLW Engineers/Architects
2400 Louisiana Boulevard, NE Suite 400
Albuquerque, New Mexico 87110

RE: GRADING & DRAINAGE PLAN SUBMITTAL OF VEHICLE STORAGE BUILDING ALBUQUERQUE INTERNATIONAL AIRPORT, RECEIVED SEPTEMBER 7, 1988 FOR BUILDING PERMIT APPROVAL (~~N-15/D3~~) M-16/D024

Dear Mr. Clark:

The above referenced submittal, dated September 7, 1988 is approved for Building Permit sign-off by Hydrology. Include this approved plan with the construction sets routed for sign-off.

If you have any questions, please call me at 768-2650.

Cordially,

Roger A. Green, P.E.
C.E./Hydrology Section

RAG/bsj
(WP+868)

1909.11

DESIGN ANALYSIS REPORT

ALBUQUERQUE INTERNATIONAL AIRPORT GENERAL AVIATION AREA, PART 4

PREPARED FOR
CITY OF ALBUQUERQUE, NEW MEXICO

APRIL 1983

FOR REVIEW ONLY

LEEDSHILL-HERKENHOFF, INC. COPPER SQUARE P.O. BOX 1217 ALBUQUERQUE, NEW MEXICO 87103

LEEDSHILL · HERKENHOFF, INC.
Copper Square
P. O. Box 1217
Albuquerque, New Mexico 87103
(505) 247-0294

632-1909.11-83

April 12, 1983

Mr. R. L. Pratte, A.A.E.
Director of Aviation
City of Albuquerque
P.O. Box 1293
Albuquerque, New Mexico 87103

RE: SOUTH GENERAL AVIATION AREA 'B', PART 4

This report contains the data developed during the design analysis phase of this project and is for your review. Alternates, cost data and engineering data are presented in accordance with our contract. The recommendations are contained in Chapter VI along with a possible schedule.

We will schedule a review meeting as soon as the City has had a chance to look over this data. If there are any questions, please do not hesitate to call me.

CLINT DODGE, P.E.

bz

Enc.

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CHAPTER I

PROJECT DEFINITION

A. General:

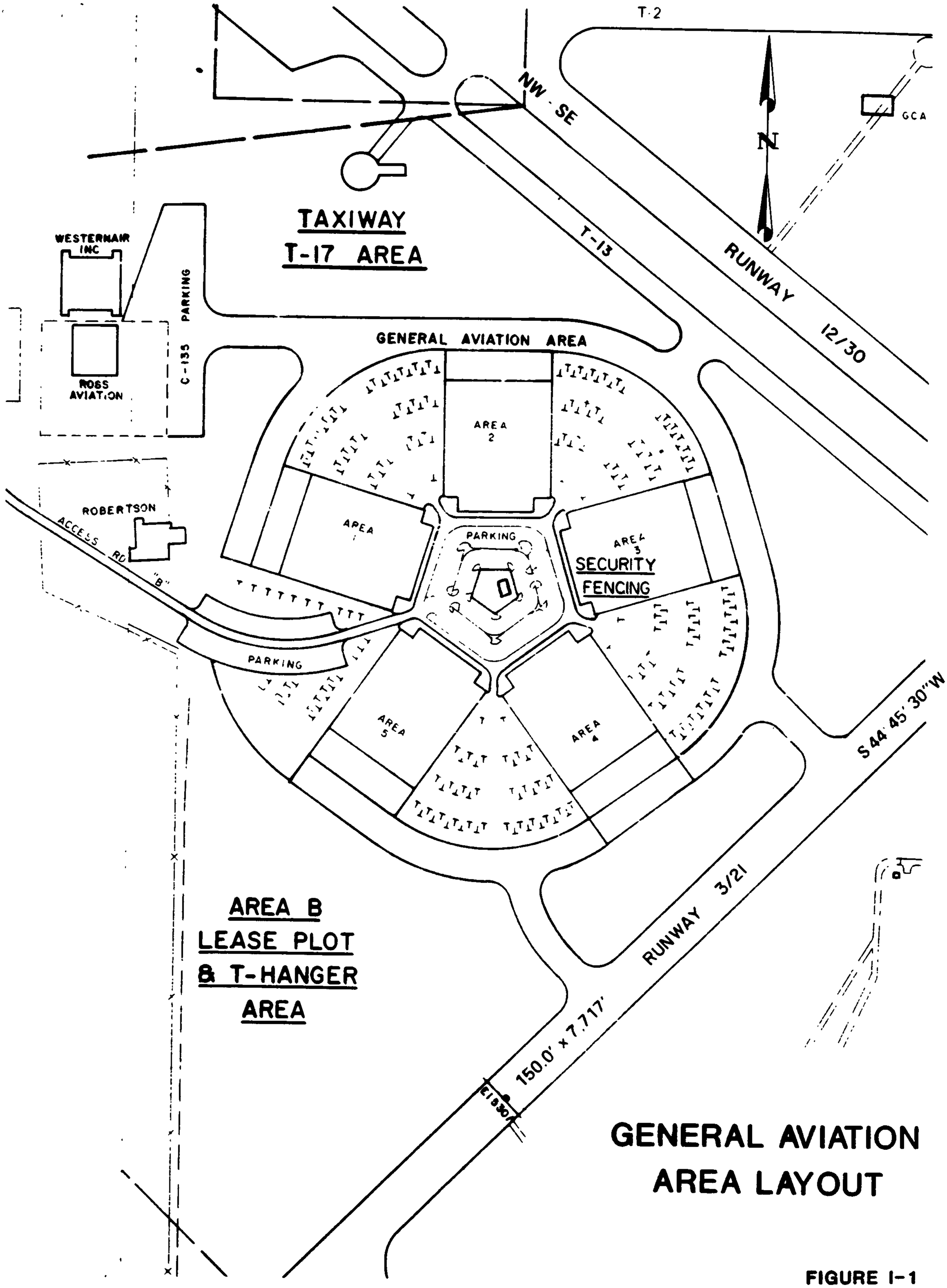
This report is intended to document the design factors considered in the development of this project. The report consists of three general major sections; the first being a project description and consideration of alternates, the selected alternates are more fully analyzed in the second section and supportive calculations, tests, etc. are contained in the appendix. The report is intended to be detailed enough to provide the City with sufficient data so that final design of construction plans can proceed immediately following report approval.

B. Project Description:

The project location and adjacent existing facilities are shown on Figure I-1. The project consists of the following elements:

1. Connecting unlit taxiway from T-13 to the northern edge of the South General Aviation Area. This taxiway is intended to serve two major functions; provide General Aviation (GA) aircraft direct access to T-2 (parallel to runway 8/26) and to provide access by occasional DC-9 type aircraft to the Western Air facility for maintenance work. The location as established by the City in conjunction with Air Traffic Control Tower (ATCT) preferences is from the T-2/T-13 intersection south to the GA area. This taxiway will be identified as T-17.

2. Taxilanes and ramp area for 'T' hangars and lease plots south of the GA area west of runway 3/21. The only lessor presently interested in aviation plots in this area is Robertson Aircraft. Consequently, a reconfiguration of the 100,000 SF lease plot originally described in the lease bid to optimize access, utilities, future expansion and costs is mutually agreeable. The alternates which are identified include the consideration that runway



GENERAL AVIATION AREA LAYOUT

FIGURE I-1

3/21 may be lengthened and upgraded for air carrier use. The entire area south of the GA area, west of the runway 3/21 building restriction line and east of the present fence line is available for potential use, as directed by the City. The tiedown area originally proposed for a portion of this area will be partially superceded by the improvements of this project. The 'T' hangars to be re-located here are those presently situated west of the FAA buildings.

3. Access and utilities are to be extended to this 'T' hangar and lease plot area. An access road as well as sanitary sewer, gas and electrical will be extended from the western edge of the South GA area. Water will be extended from the distribution main in the road along the western property line. Storm drainage will be provided by the existing storm drain system insofar as is feasible.

4. Security fencing will be provided for access control in the G.A. area as part of the relocation of the gate now presently located on the western access road.

C. Design Criteria:

1. Taxiway T-17

a)	Design Aircraft	DC-9
b)	Gross Weight	90,000 #
c)	Frequency	500 operations/year ±
d)	Taxi Speed	10 mph or less
e)	Soil CBR	8 @ 85%, 28 @ 90%, 42 @ 95%
f)	Lighting	None (Retro reflective)

2. G.A. Taxilanes/Ramp

a)	Design Aircraft	G.A. Utility
b)	Gross Weight	12,500 lb.
c)	'T' Hangar Sizes	140' x 50' ± 310' x 50' ±
d)	Soil CBR	1 @ 85%, 20 @ 90%, 23 @ 95%
e)	Lease Area	100,000 SF
f)	Anticipated Lease Hangar	120' x 160'
g)	Lighting	None
h)	Runway Clearance	Air Carrier

3. Access/Utilities

a)	Access Road Category	COA Collector
b)	Traffic Lanes	2
c)	Water Fire Flow Requirement	2000 gpm (8" min.)

4. Security Fencing

a)	Height/Type	5' Chain Link
b)	Anticlimb barbed wire	Yes

CHAPTER II

LAYOUT ALTERNATIVES

The layouts are based on typical dimensional criteria for air carrier runways and for GA T-Hangar areas. Based on recommended FAA standards unless otherwise noted, these criteria are:

Separation-Runway ℄ to Taxiway ℄ (Adequate for all AC fleet)	400 ft.
Building Restriction Line from Runway ℄	750 ft.
Taxiway ℄ to taxilane ℄	325 ft. typ.
T-17 Width (DC-9 class A/C)	50 ft.
T-17 Shoulder width (20' recommended by FAA)	10 ft. min.
T-17 ℄ Radius	100 ft.
T-Hangar separation	70 ft.
G.A. Taxilane (40 ft. recommended by FAA additional width since next to lease boundary)	50 ft. typ.

A. Leased Plot/'T' Hangar Area

The lease plot layout was originally laid out to honor the set aside area for tiedowns south of the main GA area. At the direction of the City, modifications to the original plot boundaries to optimize land use were investigated. Several different layouts were developed using the design criteria. These layouts are presented in schematic form on Figures III-1 through III-5. Elements which were considered in the evaluation included cost, traffic flow, compatability with possible westward expansion, availability of future lease plots, hangar orientation (to minimize winter snow/ice problems), and security.

A brief description of each of the layouts follows:

Plan A: Plan A attempts to maximize the accessibility and leave as much leasable property open as feasible. The facilities are oriented parallel to runway 3/21 which allows compatable future development. Utility extensions are minimized.

Plan B: Plan B is a variation of Plan A with 'T' hangars oriented perpendicular to the runway instead of parallel. There are no significant advantages over Plan A with the disadvantage of winter snow and ice accumulation on the north side of the hangars.

Plan C: Plan C has the lease plot area oriented with the long lease frontage facing the G.A. apron area but situated to avoid the storm drain. Disadvantages include the poor use of potential leasable areas adjacent to the G.A. area and drainage considerations in the infield area.

Plan D: Plan D is similar to Plan C except that the lease plot is located further south in the same locale as the original lease plot layout. The lease plot shown is configured to allow optimum siting of the proposed hangar.

Plan E: Plan E utilizes the original lease configuration. A true north-south orientation for the 'T' hangars is shown (optimum). However, this orientation does not lend itself to future westward expansion which will most probably should be oriented parallel to the runway.

A summary of the parameters for each alternate plan and of the estimated cost is presented on the following table. The recommended alternate is Plan "A" which minimizes cost and maximizes accessability. Plan "A" also represents the best layout from considerations of future airport development.

B. Taxiway T-17

Taxiway T-17 alternates are limited in scope due to the desired connecting point onto T-2/T-13. Three alternates were defined as illustrated on Figures II-6, 7 and 8.

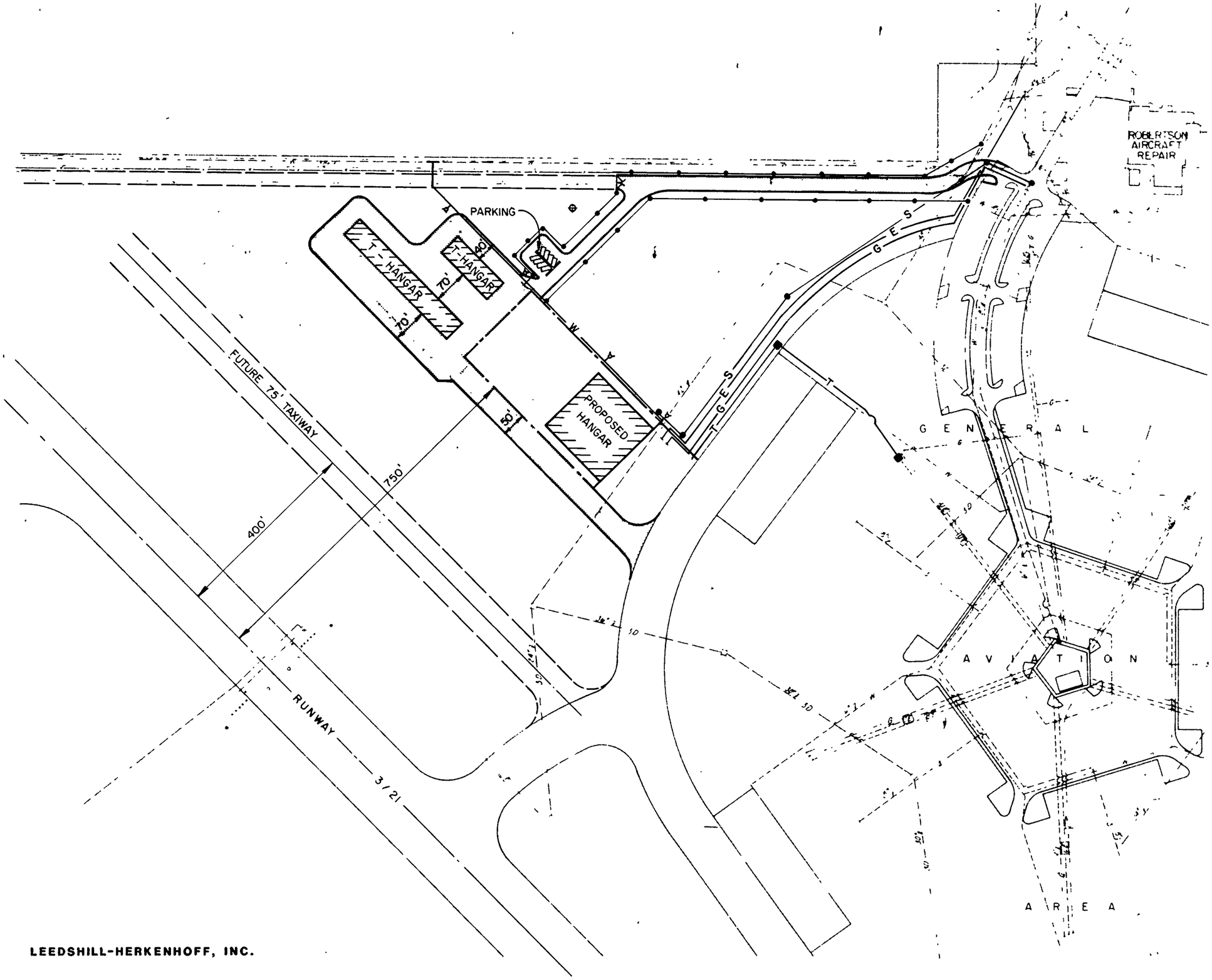
"Plan A" provides the best exit from T-13 onto T-17 for traffic from both directions since it is at right angles. The connection onto the GA ramp taxilane provides good access for all three traffic directions (i.e., east, south and west).

Plan "B" is the minimum cost alternate since it is the shortest distance. However, the connection onto T-13 for traffic east on T-13 is not as suitable, but would be satisfactory. The connection to the G.A. ramp is oriented directly towards a leased hangar site and is not as satisfactory from this viewpoint as Plan "A".

Plan "C" provides ready and direct access to the Western Air/Ross area but does not provide good G.A. traffic flows, a primary purpose of T-17.

A fourth alternate, which is not illustrated is a taxiway connecting onto the south side of the holding bay and thence running south to connect onto the taxilane area in front of Western Air. Ground control (ATCT) recommended the connection at the T-2 intersection to facilitate ground movements and increase the use of T-2.

Plan "A" is recommended as best meeting the design intent of this taxiway. The taxiway will not be lit, but will be provided with retroflective edge markers. Ducts for taxiway lights and future use will be provided.



ALBUQUERQUE INTERNATIONAL AIRPORT SOUTH

GENERAL AVIATION AREA, PART 4

AREA B SCHEMATIC LAYOUT PLAN "A"

FIGURE II-1

LEEDSHILL-HERKENHOFF, INC.

N

100 50 0 100 200

SCALE IN FEET

LEGEND

- 32' ROAD PAVEMENT
- APRON PAVEMENT
- S — SANITARY SEWER LINE (M.H.)
- G — GAS LINE
- E — ELECTRICAL LINE
- CHAIN LINK FENCE
- PROPOSED LEASE BUILDING
- F.H. W — WATER LINE

ROBERTSON AIRCRAFT REPAIR

PARKING

T-HANGAR 70'

T-HANGAR 70'

T-HANGAR 50'

50'

PROPOSED HANGAR

FUTURE 75' TAXIWAY

750'

300'

400'

RUNWAY 3/21

GENERAL AVIATION AREA

ALBUQUERQUE INTERNATIONAL AIRPORT SOUTH

GENERAL AVIATION AREA, PART 4

AREA B SCHEMATIC LAYOUT PLAN "B"

FIGURE II-2

LEEDSHILL-HERKENHOFF, INC.

N

100 50 0 100 200

SCALE IN FEET

LEGEND

- 32' ROAD PAVEMENT
- APRON PAVEMENT
- S — SANITARY SEWER LINE (M.H.)
- G — GAS LINE
- E — ELECTRICAL LINE
- CHAIN LINK FENCE
- F.H W — WATER LINE
- PROPOSED LEASE BUILDING

ROBERTSON AIRCRAFT REPAIR

PARKING

T-HANGAR

T-HANGAR

PROPOSED HANGAR

FUTURE 75' TAXIWAY

750.00'

400.00'

RUNWAY 4-21

G E N E R A L

A V I A T I O N

A R E A

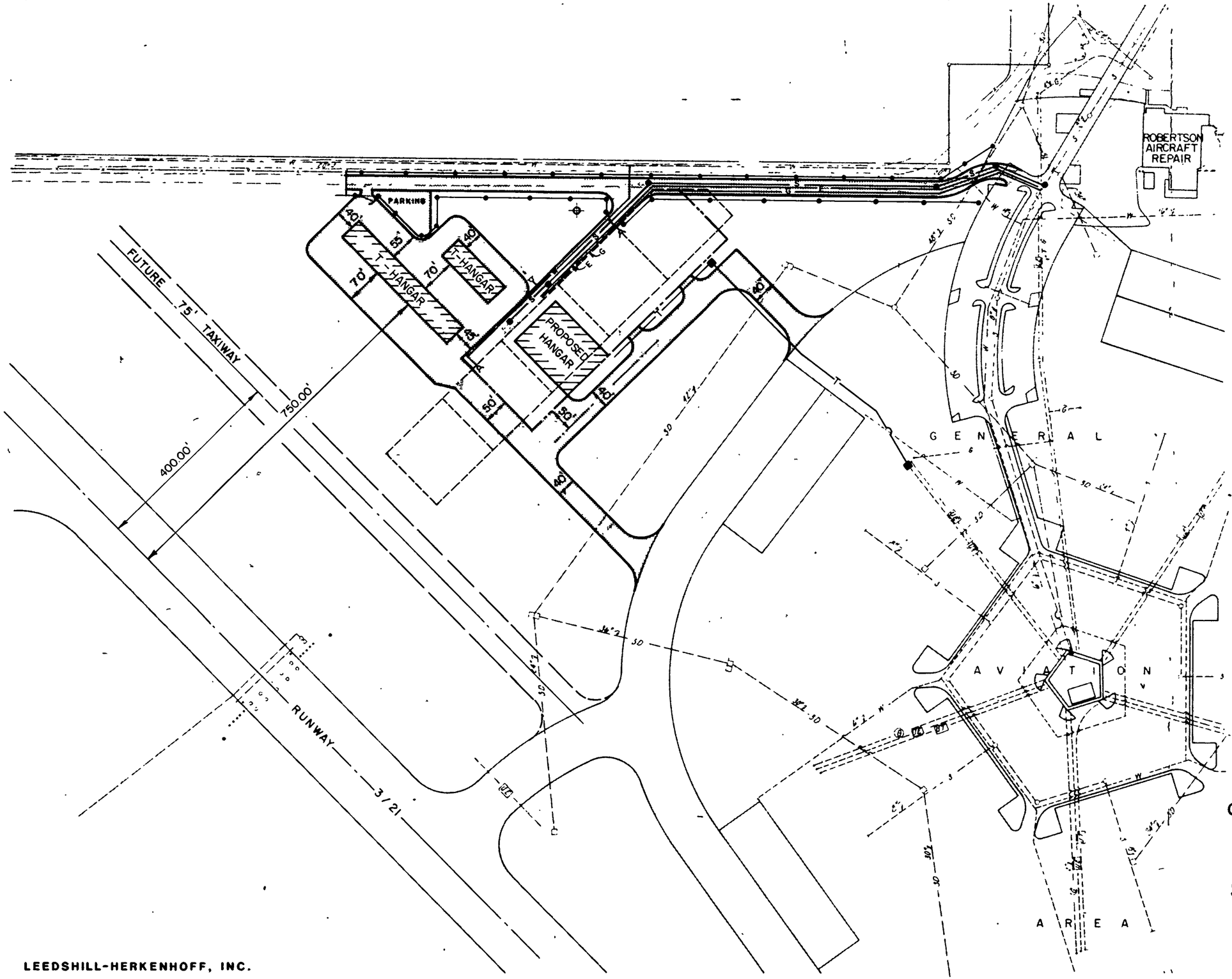
ALBUQUERQUE INTERNATIONAL AIRPORT SOUTH

GENERAL AVIATION AREA, PART 4

AREA B SCHEMATIC LAYOUT PLAN "C"

FIGURE II-3

LEEDSHILL-HERKENHOFF, INC.



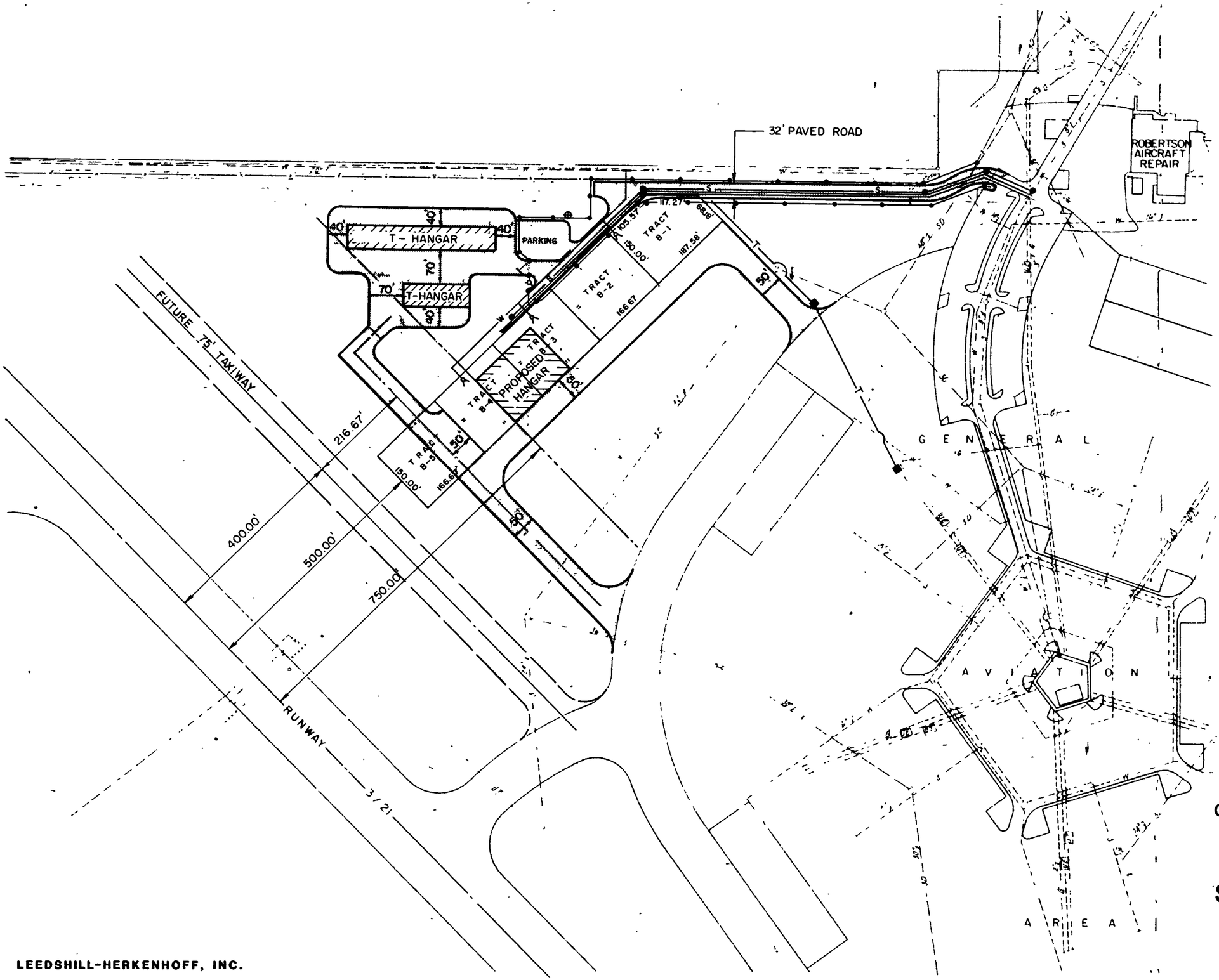
ALBUQUERQUE INTERNATIONAL AIRPORT SOUTH

GENERAL AVIATION AREA, PART 4

AREA B SCHEMATIC LAYOUT PLAN "D"

FIGURE II-4

LEEDSHILL-HERKENHOFF, INC.



N

100 50 0 100 200

SCALE IN FEET

LEGEND

- 32' ROAD PAVEMENT
- APRON PAVEMENT
- S — SANITARY SEWER LINE (M.H.)
- G — GAS LINE
- E — ELECTRICAL LINE
- CHAIN LINK FENCE
- W — WATER LINE (F.H.)

ALBUQUERQUE INTERNATIONAL AIRPORT SOUTH

GENERAL AVIATION AREA, PART 4

AREA B SCHEMATIC LAYOUT PLAN "E"

LEEDSHILL-HERKENHOFF, INC.

FIGURE II-5

ROSS AVIATION HANGAR

WESTERNAIR INC HANGAR

ROBERTSON AIRCRAFT REPAIR

GENERAL AVIATION AREA

100'R

18"

SD

UE

CONCRETE PAD TO BE REMOVED

HOLDING LANE

50'

100'R

100'R

100'R

℄ CURVE DATA

Δ = 40°00'00"
R = 996.45'
T = 362.68
D = 5°45'

TAXIWAY T-13

RUNWAY 12/30

TAXIWAY T-2

RUNWAY 8/26

N

100 50 0 100 200

SCALE IN FEET

LEGEND

- TAXIWAY PAVEMENT
- UE — UNDERGROUND ELECTRICAL DUCT
- SD — STORM DRAIN

LEEDSHILL-HERKENHOFF, INC.

ALBUQUERQUE INTERNATIONAL AIRPORT SOUTH

GENERAL AVIATION AREA, PART 4

TAXIWAY T-17 SCHEMATIC LAYOUT PLAN "A"

FIGURE II-6

ROSS AVIATION HANGAR

WESTERNAIR INC. HANGAR

ROBERTSON AIRCRAFT REPAIR

GENERAL

AVIATION

AREA

100'R

18" SD

50'

150'R

100'R

100'R

HOLDING LANE

TAXIWAY T-13

TAXIWAY T-2

RUNWAY 12/30

RUNWAY 8/26

LEEDSHILL-HERKENHOFF, INC.

N

100 50 0 100 200

SCALE IN FEET

LEGEND

TAXIWAY PAVEMENT

UE — UNDERGROUND ELECTRICAL DUCT

SD — STORM DRAIN

ALBUQUERQUE INTERNATIONAL AIRPORT SOUTH

GENERAL AVIATION AREA, PART 4

TAXIWAY T-17 SCHEMATIC LAYOUT PLAN "B"

FIGURE II-7

ROSS AVIATION HANGAR

WESTERNAIR INC. HANGAR

ROBERTSON AIRCRAFT REPAIR

100'R

150'R

50'

100'R

HOLDING LANE

CONCRETE PAD TO BE REMOVED

100' R

GENERAL AVIATION AREA

TAXIWAY T-13

RUNWAY 12/30

RUNWAY 8/26

TAXIWAY T-2

LEEDSHILL-HERKENHOFF, INC.

N

100 50 0 100 200

SCALE IN FEET

LEGEND

TAXIWAY PAVEMENT

UE — UNDERGROUND ELECTRICAL DUCT

ALBUQUERQUE INTERNATIONAL AIRPORT SOUTH

GENERAL AVIATION AREA, PART 4

TAXIWAY T-17 SCHEMATIC LAYOUT PLAN "C"

FIGURE II-8

CHAPTER III
UTILITY EXTENSIONS

A. Water:

There is an existing 16 inch water line in the perimeter loop road in the G.A. area. It is proposed that a connection be made to this line to supply service to the T hangars and lease plot. Flow requirements are based upon some domestic water use (drinking water, restrooms, washdowns, etc.) and provisions for fire flow. Based upon City of Albuquerque criteria, a minimum size 8 inch line must be provided for this light commercial use area. Preliminary fire flow requirements are based upon total square foot area of buildings. The total square foot area of a potential lease plot hangar is estimated to be 20,000 S.F. The formula used in determining the total fire flow is:

$$F=18C(A)^{0.5}$$

where F = required fire flow in gallons per minute

C = coefficient for building construction, assumed to be 0-8 in this case

A = total floor area

In this case, the fire flow is 2,000 gallons per minute. To supply this, two hydrants each capable of handling 1,000 gallons per minute are proposed for the lease plot area. Two hydrants will also be provided adjacent to the T-hangar area.

B. Natural Gas:

A natural gas line is located in the access road to the South G.A. area, with extension into the Fixed Base Operator areas. A natural gas service will be required in the lease plot area. The Gas Company of New Mexico will install the line. This should be done in conjunction with installation of other utilities so that these utilities are in place prior to paving.

C. Sanitary Sewer:

There is an 8 inch sanitary sewer system in the access road and Fixed Base Operator area. The proposed extension into the lease plot area is via an 8 inch line to this area with 4 inch service lines as required. This sewer line extension will tie into an existing manhole located in the access road. This high strength V.C.P. sewer line will be extended into the lease plot area in this contract with individual service lines the responsibility of the leasors. The minimum slope of this line will be 0.0060 feet/foot. Manholes will be installed at 350 feet maximum spacings for access and cleaning purposes. The manhole located in front of the present Robertson hangar is 21.55 feet deep which allows a maximum average slope for the sewer of 1.9%. Drop manholes will be utilized to minimize the depth of the sewer and to prevent supercritical flow.

D. Electrical:

The proposed electrical extension to the lease plot area will ultimately be the responsibility of the lease plot individuals. The work proposed under this contract would be to construct concrete encased duck work, traffic bearing pull boxes and concrete pads for pad-mounted transformers. The primary power duct work would begin at the existing power supply with three phase power and consist of two-four inch P.V.C. concrete encased ducts. The ducts would enter traffic bearing pull boxes at regular intervals. The pull boxes would be built in accordance with Public Service Company of New Mexico specifications. The four inch duct work would lead to a concrete pad located in the utility easement west of the lease plot area. The concrete pad would be built per PNM specifications to accommodate one 120/208 V 3Ø transformer and one 120/240 V 1Ø transformer. From the transformer pads the two 4 inch ducts would continue to a pull box located near the T hangars. From this pull box, two 2 inch concrete encased PVC conduits would go to each T hangar. These conduits would be provided with steel elbows and risers to five feet above finish grade and capped. This duct work and pull box system would

allow for power to be supplied to the lease plot area as required by the leasors. It would then be their responsibility to obtain the necessary electrical service as well as metering devices.

The taxiway T-17 connection to T-13 will cross the taxiway lighting cables. Two 4 inch concrete encased ducts will be installed at the return and new 5KV L-824 cable will be installed. The circuit supplying power to the taxiway lgihts will be connected at the adjacent lights and through cables will be spliced with 5KV connectors. One duct will be kept as a spare.

Two 4 inch concrete encased ducts will also be supplied at the T-17 connection to the G.A. ramp for future use.

E. Telephone:

Duct work and pull boxes will also be provided for underground telephone service to the lease plot areas. Extension and installation of services will again be the responsibility of the lessors. The telephone trunk cable will be extended from the terminal area into the "utilities building" in the center of the FBO area. Service for the lease plot will be extended from the existing telephone duct in the FBO area. A pull box will be located at this point to allow service to the FBO hangar as well as service to the lease area. A 4'x 2' watertight pull box will be required to protect the splice. Other pull boxes will be 3'x 3' as required for turns at a maximum of 350'.

All utilities will be installed in a utility "corridor" that roughly parallels the Fixed Base Operator ramp area, then turns and extends southwest adjacent to the lease plot. This configuration allows for future expansion to the west of this area.

CHAPTER IV

PAVEMENT DESIGN

The preliminary pavement design is based upon soils data gathered at the sites in March 1983 and the FAA Advisory Circular entitled "Airport Pavement Design and Evaluation" (AC 150/5320-6C). A copy of the soils report is contained in the Appendix.

A. Taxiway T-17:

The design loading for taxiway T-17 is for a dual gear DC-9 with a gross vehicle weight of 90,000 pounds and an estimated 500 operations per year. Pavement design for aircraft is based upon the California Bearing Ration (CBR) value of the existing soil. Tests performed also included determination of Atterburg limits, in-place densities and gradation.

The soil in the area of the proposed taxiway is a silty sand, non-cohesive and well drained. The liquid limit and plasticity index are both zero. In place densities ranged from 101.7 pounds per cubic foot at a depth of 7 feet to 118.8 pounds per cubic foot at a depth of 1 foot below some existing pavement. The optimum density as determined by a Modified Proctor was 124.4 pounds per cubic foot.

California Bearing Ratio Tests were conducted and yielded the following results:

at 85% relative compaction, CBR = 8
at 90% relative compaction, CBR = 28
at 95% relative compaction, CBR = 42

Based upon these CBR values, a preliminary pavement analysis was done assuming a dual wheel geared aircraft. Using the criteria in the referenced circular with a minimum CBR of 20, it was determined that the total pavement thickness required was 9.5 inches with a minimum bituminous surface course of

4 inches and a minimum asphalt treated base course of 4 inches. This method of analysis assumes removal and recompaction of the native material to a depth of 54".

To analyze this design in another manner, a CBR value of 8 for the native soil was used. In this case the critical assumption used is that the native material has an in place density of 85% of the maximum. From the field investigation this does not seem an unreasonable assumption. Using this CBR value, a total thickness of 18.5 inches is required. The thickness of the pavement required is then 9.5 inches and with a compacted subgrade with a CBR value of 20+, no subbase is required. Using this information, the pavement section for proposed Taxiway T-17 consists of 4 inches of asphalt concrete surface course, (1800 pounds stability) tack coat if required, 6 inches of asphalt treated base course (1800 pounds stability), 6 inches of prepared subgrade (100% compaction) and 6 inches of prepared subgrade (95% compaction). This pavement section exceeds the minimum standards. Because of the high CBR values of the compacted subgrade, costs of removal and compaction of excessive subgrade excavation can be avoided using the subgrade preparation specified. The asphalt treated base course will be used in lieu of cement treated base course to minimize reflective cracking.

B. Taxilane and Aprons in Lease Plot Areas:

The design loading for this paved area is for aircraft having a gross vehicle weight of 12,500 pounds or less. Again, this pavement design is based upon the CBR value of the subgrade material. The same tests performed for the taxiway were performed in the lease plot area.

The soil in the lease plot area was slightly coarser than that in the taxiway area. The liquid limit was found to be 23, while the plasticity index was found to be zero. The in-place densities ranged from 95 to 105.9 pounds per cubic foot. The optimum density was determined by Modified Proctor to be 123.0 pounds per cubic foot and the optimum moisture was 9.6%.

The CBR values were found to be:

at 85% relative compaction, CBR = 1
at 90% relative compaction, CBR = 20
at 95% relative compaction, CBR = 23

Using the design curves in the advisory circular, and the minimum CBC value of 20, the minimum allowable thickness is 2 inches of surface course pavement.

Base course is normally required under the minimum surface. However, the minimum surface can be reduced if bound base is used. In this instance, it is proposed that the City of Albuquerque standard design for residential pavement be used. This pavement section consists of 1 inch of asphalt concrete surface course (with 1500 pound stability), 2 inches asphalt treated base course (with 1500 pounds stability), and 6 inches of prepared subgrade, compacted to 95% maximum density. The reasons for using this pavement section are twofold, there is additional strength for occasional heavy loads (such as trucks) and there is additional grade control in two pavement mats. This section will also meet FAA requirements.

CHAPTER V

DRAINAGE

The drainage analysis of this report is limited to the investigation of the effect of this project on the existing drainage system. The outfall of the improvements installed as part of the G.A. area is into the historic drainage path which flows westward, crosses University Blvd., UNM Golf Course, I-25 and is intercepted by the AMAFCA South Diversion Channel. A report of this system was prepared by Tom Mann and Associates titled "Albuquerque Golf Course Drainage Study (A/E 32-14) dated July 30, 1982 (referred to as AIA/UNM). A subsequent companion study for UNM by Denny-Gross and Associates is in preparation. The first report concentrates on the hydrologic and hydraulic changes brought about by the construction of the G.A. area, including sediment transport. The second report concentrates on the means of conveying this flow across the golf course.

The present plan is to convey the runoff across the golf course in a closed conduit which has sufficient capacity to carry runoff from future developments in the escarpment area below the airport. It is assumed that this plan will be put into effect.

There is an existing strom drain system in both areas of this project. This system consists of double and triple "D" inlets and 36" to 48" reinforced concrete pipe and is shown on thee layout plans. This system was constructed in 1979 as part of the Fixed Base Operations area. The collected water discharges to the arroyo on the west.

Discussion with city hydrology staff indicates a preference to pond the runoff generated from development of the lease plot area and new taxiway at the existing inlets. This would allow a gradual "bleed off" of the collected water. The inlets could be partially closed off with plates if desired to slow down the discharge rate. Current City criteria states that all ponding areas must be drained completely within 24 hours.

Hydrologic investigations were accomplished following the Development Process Manual. Outlined procedures were used to estimate rainfall volume and intensities as well as runoff volumes and peak rates.

The design storm for this project is the 6 hour 100 year storm. The volume of rainfall for this storm at the airport is 2.33 inches.

Soil in this area was classified as wink fine sandy loam having 0-5 percent slopes by the Soil Conservation Service's "Soil Survey of Bernalillo County and Parts of Sandoval and Valencia Counties, New Mexico". This soil was then classified as soil type B. Coefficients of runoff were then determined from land use descriptions as:

Pasture or range land, fair condition C = 0.34

Paved parking lots, roofs C = 0.98

Rainfall intensities were determined using the time of concentration and total 6 hour rainfall volumes. The times of concentrations were determined using the formula:

$$Tc = 0.0078 \frac{(L)^{0.77}}{(S)^{0.33}}$$

where Tc = time of concentration, in minutes
L = length of flow path of runoff, in feet
S = slope of flow path, ft/ft

The minimum Tc allowed for design by the DPM is 10 minutes. Actual times were calculated and averaged under 10 minutes, so a time of concentration of 10 minutes is used. These calculations are given in the Appendix.

Rainfall intensity as determined from the DPM was 5.01 in./hr. This value was used to determine the peak runoff by the rational formula, Q = CIA.

Taxiway Area: In the taxiway area, contributing runoff to the existing triple D inlet is approximately 18 acres. There exists a concrete trim pad and abandoned paved taxi of 1 acre. The proposed taxiway would consist of approximately 1.4 acres of paved area. The existing pad and strip will be removed. Using a C value of 0.39 for the developed area, a peak flow rate of 35.2 cfs was calculated. Flow will be collected on both sides of the taxiway, via the existing inlet and a new, single "B" inlet which will be constructed directly across from and tie into the existing inlet with an 18" RCP line. A rough grading plan of the proposed construction is shown in Figure V-1. Because of the existing pavement in the area, which will be removed, the peak runoff from new construction is expected to increase by approximately 1.4 cfs over what now exists. According to the Albuquerque International Airport/University Golf Course Drainage Study (AE 82-14), this leg of the storm drainage system can accommodate the 100 year runoff without significant ponding on the inlets. Therefore, the effect of the proposed improvement will be to increase the peak flow rate for the 100 year event from the entire northern storm drain system from the estimated 122.9 cfs from the current condition to a peak rate of 124.3 cfs, an increase of 1.1%. Total volume increases from the reported 6.699 A-F to 6.819 A-F.

Area B: Area B presently drains both into the inlet system and into a large shallow closed basin. The existing contours on the Rough Grading Plan (Figure V-2) show this existing divide. The area which does drain to the existing inlets was originally graded as part of the G.A. ramp area in preparation for possible expansion of the tiedown area. This divide roughly corresponds to the drainage divide shown in the AIA/UNM Study (Figure 7 of the study).

One concept for handling the runoff following development is to honor this drainage divide, drain the T-hangar area as it presently flows, and continue to allow ponding in this approximately 6 acre area between runway 3/21 and the improvements. According to the City Drainage Ordinance, use of permanent ponding in this manner requires waiver by the City Engineer. We

have been informed by the hydrology section staff of the City Engineer's office that such a waiver is very unlikely and their preference is to drain the entire area of development into the strom drainage system.

Consequently, the proposed drainage system which is analyzed in the following paragraphs utilizes the existing inlets. This system adds about 7 acres of drainage area to the storm drain system (identified as area 205 in the AIA/UNM study). The developed drainage in Area B will be divided into two contributing areas which flow to the existing inlets.

The west area currently drains approximately 6.3 acres. As developed approximately 1 acre will be paved access road. The undeveloped coefficient of runoff of 0.34 increases to 0.44 and the theoretical peak discharge for a 100 year event increases from 10.7 to 13.9 cfs. Runoff volume increases from 0.416 A-F to 0.538 A.F.

The east inlet presently drains approximately 8.7 acres. In the developed state, this inlet will drain approximately 15.7 acres of which 5.4 acres will be impervious. The theorectical peak rate of runoff for the 100 year storm at the inlet increases from 14.8 cfs to 44 cfs with a coefficeint of runoff change from 0.34 to 0.56. Runoff volume increases from .574 A-F to 1.707 A-F.

The Albuquerque International Airport/University Golf Course Drainage Study (A/E 82-14) investigated the storm drain capacities of this leg of the system. The report states that the storm drain system is inadequate to convey peak discharge from the 25 year 6 hour storm, as well as less frequent storms, causing water to pond on the inlets in both the G.A. area and the inlets effected by this project. Therefore, the peak runoff rate for these less frequent storms from the storm drain system will not be increased by this project. Therefore, no throttling devices on the inlets to limit peak discharges are proposed. It should be noted that if either inlet were to be completely blocked (by debris or otherwise) the storage pool from the 100 year event does not innundate any paved areas.

ROSS AVIATION HANGAR

WESTERNAIR INC. HANGAR

ROBERTSON AIRCRAFT REPAIR

CONCRETE PAD TO BE REMOVED

HOLDING LANE

DA = 18 Ac.
Q_{P100} = 35.2 cfs

TAXIWAY T-13

TAXIWAY T-2

RUNWAY 12/30

RUNWAY 8/26

GENERAL AVIATION AREA

LEEDSHILL-HERKENHOFF, INC.

100 50 0 100 200

SCALE IN FEET

LEGEND

07 EXISTING CONTOURS

07 FINISH CONTOURS

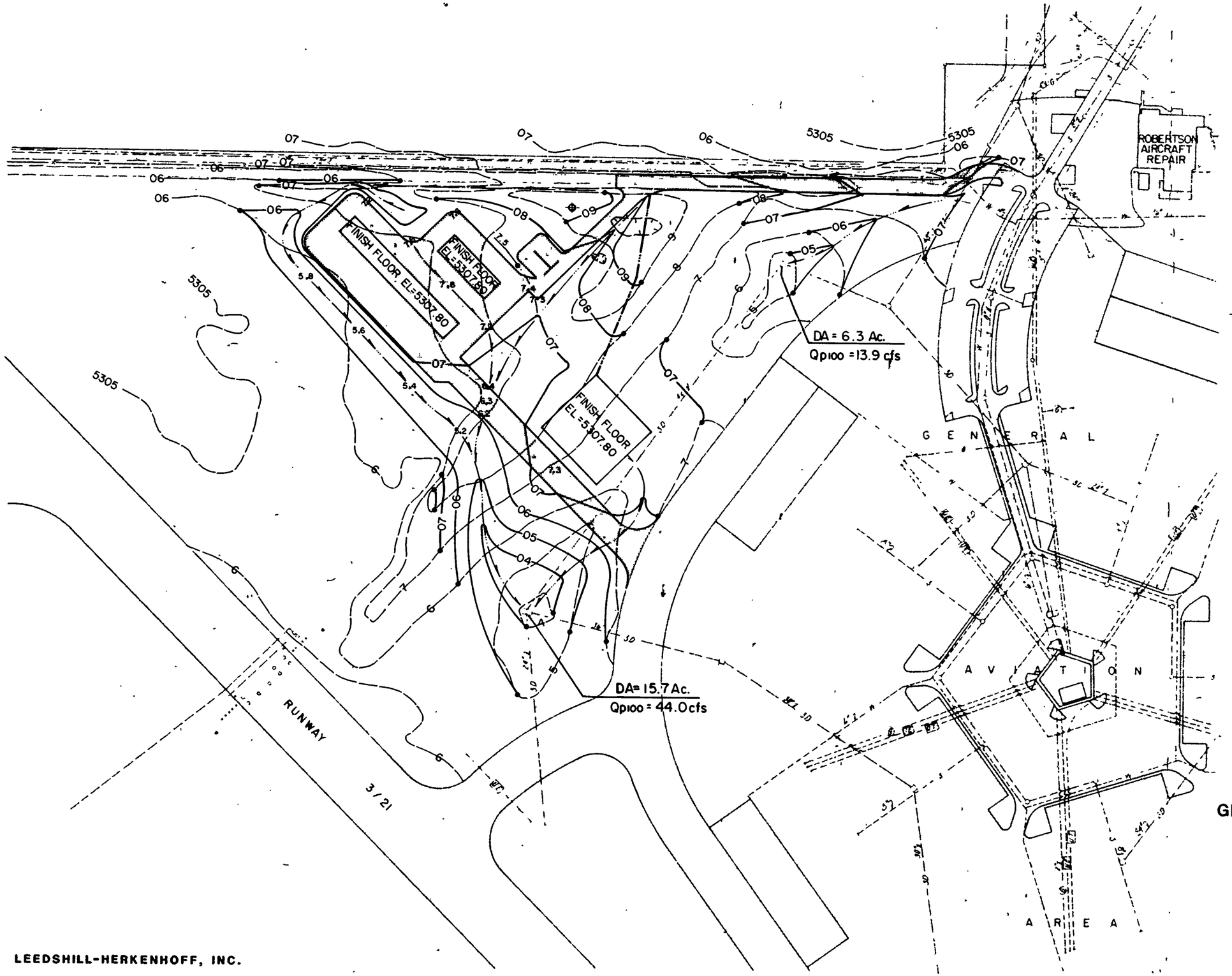
TAXIWAY PAVEMENT AREA

ALBUQUERQUE INTERNATIONAL AIRPORT SOUTH

GENERAL AVIATION AREA, PART 4

TAXIWAY T-17 ROUGH GRADING PLAN

FIGURE V-1



N

100 50 0 100 200

SCALE IN FEET

LEGEND

- 7 — EXISTING CONTOURS
- 07 — FINISH CONTOURS
- 6.3 — SPOT ELEVATION
- FLOW LINE
- DRAINAGE DIVIDE
- PAVED AREA

ALBUQUERQUE INTERNATIONAL AIRPORT SOUTH

GENERAL AVIATION AREA, PART 4

AREA B ROUGH GRADING PLAN

LEEDSHILL-HERKENHOFF, INC.

FIGURE V-2

CHAPTER VI

SUMMARY

The recommendations for each element of the project are contained in the section dealing with the design variables. In summary, the recommended project and preliminary construction cost estimates are as follows:

Taxiway T-17: Plan A connecting the T-2/T-13 intersection to the northwest edge of the FBO-GA ramp taxi lane is the preferred alternate. It offers the best traffic flow of all the alignments.

Area B: Plan A is the recommended alternate. It involves re-aligning the lease plot area to maximize utilization of the available land and optimize access.

Security Fencing: Security fencing will be provided to control automobile access for those FBO areas which are not scheduled for immediate development (all areas except Cutter).

The contruction cost estimates for the project are:

T-17	$244,000
Area B	346,000
Security Fence	40,000
Total Construction	$630,000

(See Appendix for detailed takeoff.)

The proposed schedule for this project is:

Approval of Design Analysis	April 27, 1983
Submittal of Final Review Plans	June 1, 1983
Approval of Final Plans/Advertise	June 16, 1983
Open Bids	June 30, 1983
Award and Notice to Proceed	July 7, 1983
Complete Construction	Oct. 1, 1983

This is a very ambitious schedule particularly for the review and award sequences and will require a concerted effort by everyone involved.

APPENDIX

CALCULATIONS	A-1 through A-17
SOILS REPORT	B-1 through B-5

A1

GORDON HERKENHOFF & ASSOCIATES, INC.
302 Eighth Street, N.W.
Albuquerque, New Mexico 87102
(505) 247-8284
ENGINEERING COMPUTATIONS

NAME OF PROJECT			SHEET NUMBER
ALBQ. SOUTH GA AREA			1 OF 5
COMPUTED BY	CHECKED BY	JOB NUMBER	DATE
JAE		1909.11	11 MAR 83

CONCEPT · COST ESTIMATES FOR DESIGN ALTERNATIVES

PLAN A: RECOMMENDED ALTERNATIVE

1) RELOCATION OF TEE HANGARS	BY OTHERS
2) PAVED ACCESS TO TEE HANGARS & ROBERTSON'S 16000 SY @ $12/SY	$ 192,000
3) ADDITIONAL PARKING 560 SY @ $10/SY	5,600
4) EXTENSION OF SANITARY SEWER 1100 LF OF 8" PIPE @ $12/LF 4 MANHOLES @ $1250/EA	13,200 5,000
5) EXTENSION OF ELECTRICITY 1100 LF OF POWER LINES (UNDERGROUND) MISC. SECURITY LIGHTING	20,000
6) EXTENSION OF NATURAL GAS 1100 LF OF 2" LINE @ $3/LF	3,300
7) EXTENSION OF WATER LINE 650 LF OF 6" LINE @ $17°/LF	11,050
8) PAVED ACCESS ROAD 3300 SY @ $10/SY	33,000
9) FENCING 4000 LF @ $10/LF	40,000
10) CURB AND GUTTER 1600 LF @ $8/LF	12,800
SUBTOTAL	$335,950
CONTINGENCIES (20%)	67,190
TOTAL	$403,140

NOTE: SEE SHEETS A-18 & A-19 FOR "PRELIMINARY COST ESTIMATE"

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NAME OF PROJECT			SHEET NUMBER 2 OF 5
COMPUTED BY	CHECKED BY	JOB NUMBER	DATE

PLAN B

1) RELOCATION OF TEE HANGARS	BY OTHERS
2) PAVED ACCESS TO TEE HANGARS & ROBERTSONS 16,500 SY @ $12/SY	$ 198,000
3) ADDITIONAL PARKING 560 SY @ $10/SY	5,600
4) EXTENSION OF SANITARY SEWER 1100 LF OF 8" PIPE @ $12/LF	13,200
4 MANHOLES @ $1250/EA	5,000
5) EXTENSION OF ELECTRICITY 1100 LF OF POWER LINES (UNDERGROUND) MISC SECURITY LIGHTING	20,000
6) EXTENSION OF NATURAL GAS 1100 LF OF 2" GAS LINE @ $3/LF	3,300
7) EXTENSION OF 6" WATER LINE 700 LF OF 6" WL @ $17/LF	11,900
8) PAVED ACCESS ROAD 3030 SY @ $10/SY	30,300
9) FENCING 4000 LF @ $10/LF	40,000
10) CURB & GUTTER 1650 LF @ $8/LF	13,200
SUB-TOTAL	$ 340,500
CONTINGENCIES	68,100
TOTAL	$ 408,600

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NAME OF PROJECT			SHEET NUMBER 3 OF 5
COMPUTED BY	CHECKED BY	JOB NUMBER	DATE

PLAN C:

1) RELOCATION OF TEE HANGARS	BY OTHERS
2) PAVED ACCESS TO TEE HANGARS & ROBERTSON'S 17,000 SY @ $12/SY	$ 204,000
3) ADDITIONAL PARKING 560 SY @ $10/SY	5,600
4) EXTENSION OF SANITARY SEWER 960 LF OF 8" LINE @ $12/LF 4 MANHOLES @ $1250/EA	11,520 5,000
5) EXTENSION OF ELECTRICITY 800 LF OF POWER LINES (UNDERGROUND) MISC. SECURITY LIGHTING	17,000
6) EXTENSION OF NATURAL GAS 800 LF OF 2" LINE @ $3/LF	2,400
7) EXTENSION OF WATER LINE 800 LF OF 6" LINE @ $17/LF	13,600
8) PAVED ACCESS ROAD 3910 SY @ $10/SY	39,100
9) FENCING 3000 LF @ $10/LF	30,000
10) CURB & GUTTER 1700 LF @ $8/LF	13,600
SUB TOTAL	$341,820
CONTINGENCIES (20%)	68,364
TOTAL	$ 410,184

A4

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ENGINEERING COMPUTATIONS

NAME OF PROJECT			SHEET NUMBER 4 OF 5
COMPUTED BY	CHECKED BY	JOB NUMBER	DATE

PLAN D:

1) RELOCATION OF TEE HANGARS	BY OTHERS	
2) PAVED ACCESS TO TEE HANGARS & ROBERTSON'S 15,500 SY @ $12/SY		$186,000
3) ADDITIONAL PARKING 560 SY @ $10/SY		5,600
4) EXTENSION OF SANITARY SEWER 1050 LF @ $12/LF		12,600
MANHOLES @ $1250/EA		5,000
5) EXTENSION OF ELECTRICITY 1000 LF OF POWER LINES (UNDERGROUND) MISC. SECURITY LIGHTING		19,000
6) EXTENSION OF NATURAL GAS 1000 LF OF 2" LINE @ $3/LF		3,000
7) EXTENSION OF WATER LINE 450 LF OF 6" LINE @ $17/LF		7,650
8) PAVED ACCESS ROAD 4300 SY @ $10/SY		43,000
9) FENCING 4500 LF @ $10/LF		45,000
10) CURB & GUTTER 1550 LF @ $8/LF		12,400
SUB-TOTAL		$339,250
CONTINGENCIES		67,850
TOTAL		$407,100

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ENGINEERING COMPUTATIONS

NAME OF PROJECT			SHEET NUMBER 5 OF 5
COMPUTED BY	CHECKED BY	JOB NUMBER	DATE

PLAN E:

1) RELOCATION OF TEE HANGARS	BY OTHERS
2) PAVED ALLESS TO TEE HANGARS & ROBERTSON'S 26,000 SY @ $12/SY	$ 312,000
3) ADDITIONAL PARKING 560 SY @ $ 10/SY	5,600
4) EXTENSION OF SANITARY SEWER 1250 LF OF 8" LINE @ $12/LF	15,000
5 MANHOLES @ $1250/EA	6,250
5) EXTENSION OF ELECTRICITY 1200 LF OF POWER LINES (UNDERGROUND) MISC. SECURITY LIGHTING	21,000
6) EXTENSION OF NATURAL GAS 1200 LF OF 2" LINE @ $3/LF	3,600
7) EXTENSION OF WATER LINE 500 LF OF 6" LINE @ $17/LF	8,500
8) PAVED ACCESS ROAD 2850 SY @ $10/SY	28,500
9) FENCING 4000 LF @ $10/LF	40,000
10) CURB & GUTTER 2000 LF @ $8/LF	16,000
SUBTOTAL	$ 456,450
CONTINGENCIES	91,290
TOTAL	$ 547,740

A6

GORDON HERKENHOFF & ASSOCIATES, INC. 302 Eighth Street, N.W. Albuquerque, New Mexico 87102 (505) 247-0294 ENGINEERING COMPUTATIONS	NAME OF PROJECT			SHEET NUMBER 1 OF 1
	COMPUTED BY	CHECKED BY	JOB NUMBER	DATE

CONCEPT COST ESTIMATE OF TAXIWAY ALTERNATIVES

PLAN A :

1) KEEL SECTION:
   4500 SY @ $30/SY — $135,000

2) SHOULDER SECTION
   2,550 SY @ $13/SY — 33,150

3) DUCTS
   190 LF @ $10/LF — 1,900

SUBTOTAL $170,050
CONTINGENCIES (20%) 34,010
TOTAL $204,060

PLAN B
1) KEEL SECTION
   2700 SY @ $30/SY — $81,000

2) SHOULDER SECTION
   2500 SY @ $13/SY — 32,500

3) DUCTS
   150 LF @ $10/LF — 1,500

SUB TOTAL $115,000
CONTINGENCIES (20%) 23,000
TOTAL $138,000

PLAN C
1) KEEL SECTION
   4400 SY @ $30/SY — $132,000

2) SHOULDER SECTION
   3700 SY @ $12/SY — 44,400

3) DUCTS
   220 LF @ $10/LF — 2,200

SUBTOTAL $178,600
CONTINGENCIES (20%) 35,720
TOTAL $214,320

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ENGINEERING COMPUTATIONS

NAME OF PROJECT			SHEET NUMBER
ALBQ AIRPORT			OF
COMPUTED BY	CHECKED BY	JOB NUMBER	DATE
JAE			17 MAR 83

WATER LINE EXTENSION TO SOUTH GA AREA

GENERAL NOTES:

1) THIS IS NOT A MASTER PLAN LINE
2) 8" MIN. SIZE, MAY BE LARGER DUE TO FIRE FLOW
3) MUST BE LOCATED WITHIN PAVED ROW, 25' WIDTH MIN.
4) MUST BE MIN. OF 10' FROM SAN. SEWER
5) MAX. 1200' BETWEEN IN LINE VALVES (GATE VALVES)
6) AIR RELIEF VALVES NECESSARY ON VALVES > 8"
7) INDIVIDUAL LEASORS WILL BE RESPONSIBLE FOR THEIR METERED SERVICE
8) ASSUME THAT THIS IS A HEAVY COMMERCIAL LINE, SO FIRE HYDRANT SPACING IS 300' MAX. BETWEEN HYDRANTS AND BETWEEN HYDRANT & BUILDING.
9) FIRE FLOW REQUIREMENTS BASED UPON

$$F = 18C(A)^{0.5}$$

WHERE F = REQ'D FIRE FLOW IN G.P.M
A = TOTAL FLOOR AREA
C = ASSUMED 1.0 FOR ORDINARY CONSTRUCTION

A = 20 000 SF. (120x160 assumed)

F = 2000 gpm

FROM DPM P. 116, REQUIRE 4 HYDRANTS MIN. THIS AREA

10) ALL REQUIRED FIRE HYDRANTS SHALL SUPPLY A MIN. OF 1000 gpm AT MIN. 20 P.S.I RESIDUAL PRESSURE FROM 4" OUTLET.
11) 8" LINE IS ALLOWABLE ($V \sim 12.8$ fps)

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ENGINEERING COMPUTATIONS

NAME OF PROJECT A1A			SHEET NUMBER OF
COMPUTED BY CD	CHECKED BY	JOB NUMBER	DATE

Check Sewer Velocities

$n = .013$ 8" pipe

$V = \frac{1.5}{n} R^{2/3} \sqrt{S} = \frac{1.5}{n} (D/4)^{2/3} \sqrt{S} = 34.9 \sqrt{S}$

$V < \text{critical} < \sqrt{gy}$ = 4.6 fps

$\sqrt{S} = .132$ $S_{max} = .0174$

available slope $S_{avail} \approx .019$

therefore, will require drop manholes

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ENGINEERING COMPUTATIONS

NAME OF PROJECT			SHEET NUMBER
SOUTH GA AREA			1 OF 5
COMPUTED BY JAE	CHECKED BY GD	JOB NUMBER	DATE 28 MAR 83

PAVEMENT DESIGN - TAXIWAY T-17 AREA

SOIL INFORMATION: COMPOSITE SAMPLES

GRADATION

SIZE	PERCENT PASSING
1"	—
3/4"	100%
1/2"	99.5
3/8"	99.0
no. 4	98.0
no. 10	95.8
no. 40	84.8
no. 80	56.8
no. 200	15.3

LIQUID LIMIT = 0
P.I. = 0

OPTIMUM DENSITY 124.4 pcf
MOISTURE 9.7%

CBR VALUES @ 85% RELATIVE COMPACTION CBR = 8
90% 28
95% 42

IN PLACE DENSITY AND MOISTURE

TEST HOLE #1	@ 1'	118.8 pcf 11.2%		@ 6'-7'	101.1 pcf 18.3%
" " #2	@ 1'	107.6 pcf 9.8%		@ 8'	114.6 pcf 11.2%

USE AC 150/5320-6C FOR PRELIMINARY DESIGN

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ENGINEERING COMPUTATIONS

NAME OF PROJECT			SHEET NUMBER 2 OF 5
COMPUTED BY	CHECKED BY	JOB NUMBER	DATE

DESIGN FOR DUAL WHEEL AIRCRAFT, GROSS WEIGHT 90000 lb

SUBGRADE COMPACTION REQUIREMENTS (p. 41)

USE CURVE FOR NON-COHESIVE SOIL

PERCENT COMPACTION	DEPTH REQ'D	IN PLACE ACTUAL #1	#2
100%	16"	95.5%	86.5%
95%	29.5"		
90%	42"		
85%	54"		
	66"	81.3%	
	96"		92.1%

∴ WILL NEED TO REMOVE AND REPLACE MATERIAL TO ATTAIN REQUIRED COMPACTION.

DEPTHS INDICATED ARE BELOW FINISHED SUBGRADE

CHECK TOTAL PAVEMENT THICKNESS REQUIREMENTS (p. 45)
USE MIN. ANN DEPARTURES = 1200
REQUIRED THICKNESS 4" CRITICAL AREAS
3" NON-CRITICAL AREAS } SURFACE COAT

CBR VALUE	RELATIVE COMPACTION	THICKNESS REQ'D
8	85%	18.5"
28	90%	7"
42	95%	4.6"

TOTAL MINIMUM PAVEMENT THICKNESS SHALL NOT BE LESS THAN THAT REQUIRED FOR CBR = 20, IN THIS CASE 9.5"
MINIMUM SURFACE COURSE = 4"
MINIMUM CTB COURSE = 6"
MINIMUM ATB COURSE = 4"

FAA REGULATIONS REQUIRE BOUND BASE COURSE FOR AIRCRAFT HAVING WEIGHTS > 100,000 lb, SINCE OUR DESIGN WEIGHT = 90000 lb, WHICH IS IN REASONABLE PROXIMITY, USE BOUND BASE COURSE.

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ENGINEERING COMPUTATIONS

NAME OF PROJECT			SHEET NUMBER 3 OF 5
COMPUTED BY	CHECKED BY	JOB NUMBER	DATE

LOOK AT USING MIN. CBR VALUE RATHER THAN MASSIVE REMOVAL AND REPLACEMENT OF SUBGRADE MATERIAL. CRITICAL ASSUMPTION IS THAT NATIVE MATERIAL IS 85% MAX. DENSITY (CBR=8)

18.5" THICKNESS (TOTAL) REQUIRED

THICKNESS OF SUBBASE REQUIRED (FOR CBR=20) 9.5"

18.5" - 9.5" = 9"

NEED 9" OF SUBBASE

THICKNESS OF BASE COURSE REQ'D (FROM P. 5=) 6" MIN.

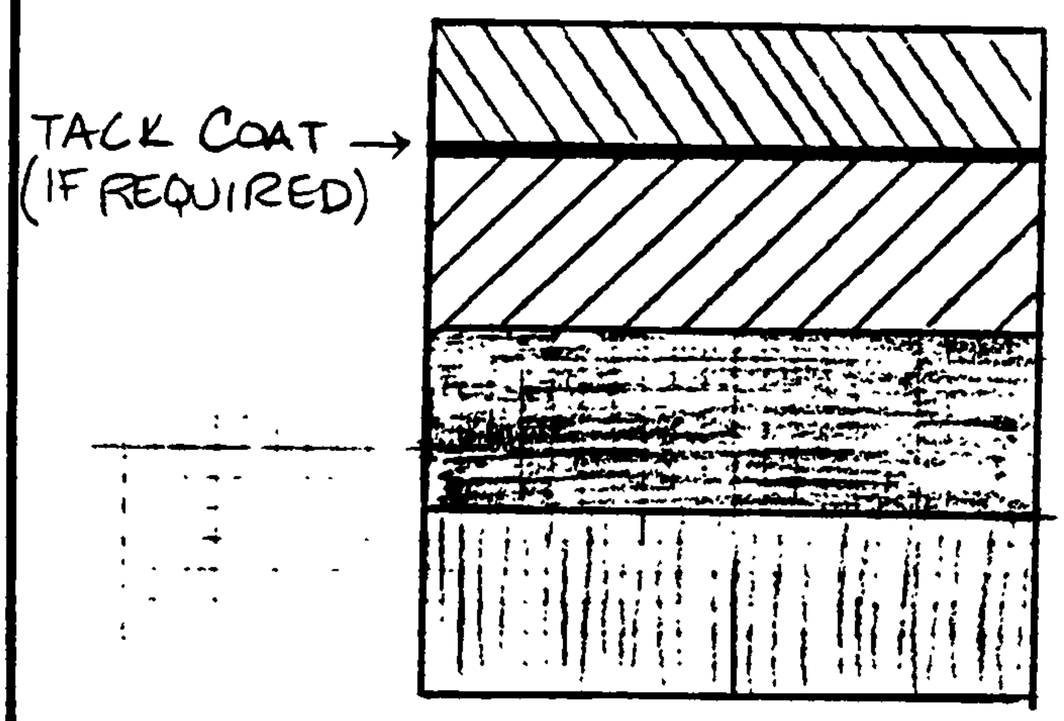
PAVEMENT SECTION WOULD THEN BE

SURFACE COURSE	4"
A.T. BASE COURSE	6"
COMPACTED SUBGRADE	9"

COMMENTS:

1) USE ATB RATHER THAN CTB TO MINIMIZE REFLECTIVE CRACKING
2) SUBGRADE COMPACTED TO 90% HAS CBR = 28. RECOMMEND USING THIS RATHER THAN SUBBASE, HOWEVER REQUIRE 100% COMPACTION ON UPPER 6"

RECOMMENDED PAVING SECTION IS:



4" ASPHALT SURFACE COURSE P-401 (STABILITY 1800)

6" ASPHALT TREATED BASE COURSE P-201 (STABILITY 1800)

6" PREPARED SUBGRADE (100% COMPACTION)

6" PREPARED SUBGRADE (95% COMPACTION)

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ENGINEERING COMPUTATIONS

NAME OF PROJECT			SHEET NUMBER 4 OF 5
COMPUTED BY	CHECKED BY	JOB NUMBER	DATE

PAVEMENT DESIGN - SOUTH GA AREA

SOIL... INFORMATION - COMPOSITE SAMPLES

GRADATION

SIZE	PERCENT PASSING
1"	100%
3/4"	99.3%
1/2"	99.2
3/8"	98.9
NO. 4	98.1
NO. 10	96.2
NO. 40	84.3
NO. 80	50.3
NO. 200	22.3

LIQUID LIMIT = 23
P.I = 0

OPTIMUM DENSITY 123.0 pcf
MOISTURE 9.6%

CBR VALUES @ 85% RELATIVE COMPACTION CBR = 1
@ 90% " " = 20
95% = 23

IN PLACE DENSITY AND MOISTURE

TEST HOLE #3 @ 1' 97.6 pcf (79%) 9.7% @ 5' 95.0 pcf (77%) 7.0%

#4 @ 1' 96.9 pcf (79%) 7.8% @ 8' 105.9 pcf (86%) 6.0%

A13'

GORDON HERKENHOFF & ASSOCIATES, INC. 302 Eighth Street, N.W. Albuquerque, New Mexico 87102 (505) 247-0294 ENGINEERING COMPUTATIONS	NAME OF PROJECT			SHEET NUMBER 5 OF 5
	COMPUTED BY	CHECKED BY	JOB NUMBER	DATE

DESIGN FOR LIGHT AIRCRAFT, GROSS WEIGHT 12,500 lb.

P.125

CBR VALUE	RELATIVE COMPACTION	THICKNESS REQ'D
1	N/A	N/A
20 23	} MIN. THICKNESS 2" SURFACE COURSE 3" BASE COURSE	

∴ CAN PROBABLY USE COMPACTED SUBGRADE AS BASE COURSE

PAVEMENT SECTION WOULD LOOK LIKE:

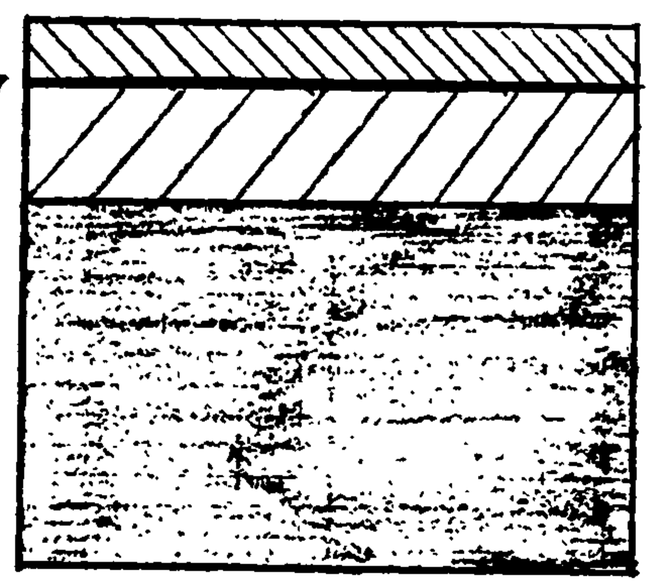
SURFACE COURSE 2"

6" COMPACTED SUBGRADE (95% MAX) AS BASE.

COMMENTS:

5320-6C IS AMBIGUOUS REGARDING REQUIREMENT FOR BASE COURSE. IT IS POSSIBLY ADVANTAGEOUS TO USE THE CITY OF ALBQ. STANDARD FOR A RESIDENTIAL PAVING SECTION AND ASPHALT MIX DESIGN. THIS WOULD SATISFY FAA REQUIREMENTS AND GIVE A LITTLE MORE GRADE CONTROL AS WELL AS ADDITIONAL STRENGTH OVER MINIMUM OF 2"

RECOMMENDED PAVING SECTION IS:



NOTE: @1.5" AGGREGATE BASE = 1" ATB, WE SATISFY THE 3" BASE COURSE REQUIREMENT FROM FAA.

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ENGINEERING COMPUTATIONS

NAME OF PROJECT			SHEET NUMBER
ALBQ AIRPORT			1 OF
COMPUTED BY: JAE	CHECKED BY: CD	JOB NUMBER:	DATE:

DRAINAGE CALCULATIONS:

SOIL TYPE "B" FROM SOIL SURVEY OF BERNALILLO COUNTY BY SCS.

COEFFICIENTS OF RUNOFF (PLATE 22.2 C-2)
UNDEVELOPED LAND = 0.34
PAVED AREAS, BUILDINGS = 0.98

PEAK 6 HR 100 YR STORM = 2.33" (PLATE 22.2 D-1)

DRAINAGE IN AREA A - TAXIWAY T-17

WANT TO DETERMINE DIFFERENCE IN FLOW VALUES FROM WHAT IS NOW EXISTING COMPARED TO DEVELOPED AREA.

TOTAL AREA CONTRIBUTING FLOW TO EXISTING INLET ~ 18 AC.
EXISTING PAVED AREA AND HELIPORT = 1 AC
AREA OF NEW TAXIWAY = 1.4 AC.

$$T_C = 0.0078 \frac{L^{0.77}}{S^{0.385}}$$

TIME OF CONCENTRATION (AVG) OF EXISTING CONDITION

$$T_{C_1} = 0.0078 \frac{(550)^{0.77}}{(0.011)^{0.385}} = 5.72 \text{ min}$$

$$T_{C_2} = 0.0078 \frac{(500)^{0.77}}{(0.01)^{.385}} = 5.5 \text{ min}$$

DPM CRITERIA SAYS MIN T_C = 10 MIN FOR DESIGN

TIME OF CONCENTRATION (AVG) OF DEVELOPED CONDITION

$$T_{C_1} = 0.0078 \frac{(600)^{0.77}}{(0.008)^{.385}} = 6.78 \text{ min}$$

$$T_{C_2} = 0.0078 \frac{(500)^{0.77}}{(0.01)^{.385}} = 5.5 \text{ min}$$

AGAIN USE T_C = 10 min.

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NAME OF PROJECT			SHEET NUMBER 2 OF
COMPUTED BY	CHECKED BY	JOB NUMBER	DATE

COMPARING I = 2.33" AGAINST T_c = 10 min IN PLATE 22.2 D-2 TO GET ACTUAL INTENSITY, MULTIPLY RAINFALL VOLUME (2.33) BY VALUE ON CURVE FOR T_c = 10 min (2.15)

I = 2.33(2.15) = 5.01" IN/HR

CALCULATE ACTUAL COEFFICIENTS OF RUNOFF

EXISTING C = $^{17}/_{18}$(.34) + $^{1}/_{18}$(.98) = .375

DEVELOPED C = $^{16.6}/_{18}$(.34) + $^{1.4}/_{18}$(.98) = .390

$Q_{DEVELOPED}$ = CIA
= .390(5.01")(18 AC) = 35.2 cfs

$Q_{UNDEVELOPED}$ = .375(5.01)18
= 33.8

ΔQ = 1.4 cfs

= CRA
$Volume_{DEVELOPED}$ = .375(2.33)(18) $^{1}/_{12}$ = 1.31 A-F

$Volume_{UNDEV}$ = .34(2.33)(18) $^{1}/_{12}$ = 1.19 A-F

ΔV = .12 AF

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NAME OF PROJECT			SHEET NUMBER 3 OF
COMPUTED BY	CHECKED BY	JOB NUMBER	DATE

DRAINAGE IN AREA B- LEASE PLOT AREA

WATER WILL DRAIN TO TWO EXISTING INLETS
WEST INLET CONTRIBUTING AREA = 6 3 AC
EAST INLET CONTRIBUTING AREA = 8.7AC (existing)

EXAMINE EAST INLET FIRST:
BUILDING AREAS = 41,700 sf ~ 1 AC
PAVED AREAS = 187,740 sf ~ 4.3 AC
NATURAL GROUND COVER = 1.7 AC. (additional)

LOOK AT RUNOFF NOW EXISTING ON UNDEVELOPED PROPERTY

$A = 8.7$ Ac

$C = 0.34$

$$T_{c_2} = 0.0078 \left(\frac{300}{(0.01)}\right)^{.77} = 371 \text{ min}$$

$$T_{c_3} = 0.0078 \frac{(500)^{.77}}{(0.0072)^{.385}} = 6.24 \text{ min}$$

MIN. $T_c = 10$ min

FROM PREVIOUS CALCULATIONS $I_{max} = 2.33(2.15) = 5.01"/HR$

$Q_{UNDEVELOPED} = CIA$
$= 0.34(5.01)(8.7)$
$= 14.8$ cfs

C OF DEVELOPED AREA
$C = 5.3/15.7(.98) + 10.4/15.7(.34) = .56$

$Q_{DEVELOPED} = CIA = 0.56(5.01)(15.7)$
$= 44$ cfs $\qquad \Delta Q = 29.2$ cfs

RUNOFF VOLUME = CRA/12 =

$V_{DEV} = 1.707$ A-F $\qquad V_{UNDEV} = .574$ A-F

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ENGINEERING COMPUTATIONS

NAME OF PROJECT			SHEET NUMBER 4 OF
COMPUTED BY	CHECKED BY	JOB NUMBER	DATE

EXAMINE WEST INLET

TOTAL DRAINAGE AREA = 6.3 AC
TOTAL PAVED AREA = 1 AC

CALCULATE AVG T_c

$$T_{c_1} = 0.0078 \frac{(350)^{.77}}{(0.0086)^{.385}} = 4.43 \text{ min}$$

$$T_{c_2} = 0.0078 \frac{(400)^{.77}}{(0.009)^{.385}} = 4.82 \text{ min}$$

$$T_{c_3} = 0.0078 \frac{(400)^{0.77}}{(0.005)^{.385}} = 6.05 \text{ min}$$

$T_{c_{AVG}} = 5.1$ min

Must use $T_c = 10$ min

CALCULATE $I_{max} = 2.33"(2.15) = 5.01"/1$ HR

$Q_{UNDEVELOPED} = 0.34(5.01")(6.3 \text{ AC})$
$= 10.7$ cfs

$C_{DEVELOPED} = 1/6.3(.98) + 5.3/6.3(.34)$
$= .44$

$Q_{DEVELOPED} = .44(5.01)(6.3)$
$= 13.9$ cfs

$\Delta Q = 3.2$ cfs

$V_{UND} = .034(2.33)(6.3)\,1/12 = .416$ A-F

$V_{DEV} = .44(2.33)(6.3)\,1/12 = .538$ AF

City of Albuquerque

P.O. BOX 1293 ALBUQUERQUE, NEW MEXICO 87103

LEGAL DEPARTMENT
(505) 768-4500

RECEIVED
FEB 03 1986
DH64)
HYDROLOGY SECTION

January 30, 1986

RECEIVED
JAN 31 1986
CITY ENGINEER

Alan Prickett
Director of Real Estate
University of New Mexico
Scholes Hall
Albuquerque, New Mexico 87131

Re: Amended Agreement - UNM South Golf Course Easements

Dear Mr. Prickett:

Enclosed please find the Amended Agreement regarding the UNM South Golf Course easements which has been revised to reflect the agreements we have reached. If I do not hear from you, I assume this version will be submitted to the Regents for approval on February 4, 1986.

Very truly yours,

Susan McKee

Susan McKee
Assistant City Attorney

SMcK/phm
Enclosure
cc: Dwayne Sheppard, City Engineer
F. Vedamanikam,
Water Utilities Division

AN EQUAL OPPORTUNITY EMPLOYER

AMENDED AGREEMENT
UNM SOUTH GOLF COURSE EASEMENTS

This Amended Agreement is entered into this 4th day of February, 1986, by and between the Regents of the University of New Mexico, a corporation of the State of New Mexico ("UNM") and the City of Albuquerque ("City") and amends that Agreement dated October 9, 1984 ("Original Agreement") between these parties.

WHEREAS, Airport Terminal Center, Ltd. ("ATC") owns the property east of the UNM South Golf Course ("Golf Course") and is in the process of developing same; and

WHEREAS, ATC has dedicated a public easement to the City for drainage through this property and is willing and able to construct a drainage pipe which will connect with the pipe to be constructed pursuant to the Original Agreement; and

WHEREAS, UNM and the City will benefit by such connection because it will negate the requirement for an inlet structure at the east side of the Golf Course thereby reducing the cost of the improvements which were the subject of the Original Agreement;

NOW THEREFORE in consideration of the above and for other good and valuable consideration, the parties agree to the following amendments to the Original Agreement:

1. The drainage system to be designed and constructed by UNM pursuant to the Original Agreement shall be modified to

eliminate the inlet structure at the east side of the Golf Course.

2. ATC will utilize a qualified Contractor on the above-referenced system to construct the pipe across ATC's property and connect ATC's pipe to UNM's and the City will enter into a Subdivision Improvements Agreement with ATC for same which shall provide that ATC is totally responsible for the cost of the connection to UNM's pipe, the construction of the pipe across its property, and the inlet structure at the east side of its property.

3. If ATC is unable to reach an agreement with a qualified contractor to assure that construction of its pipe and the connection to UNM's pipe will be completed by April 1, 1986, then UNM will not be bound by this agreement.

4. UNM agrees to grant to the City a public drainage easement across the South Golf Course in the final "as built" location of the UNM owned drainage system.

5. The City will assume maintenance responsibility for said drainage system, subject to the following requirements:

a. The maintenance schedule will be agreed upon between the City and the Director of UNM's Golf Course in order to minimize interruption of golf play.

b. Any damage to the South Golf Course due to the City's maintenance will be promptly repaired by the City at the City's expense.

c. The requirements referred to in paragraphs 5a and 5b shall also apply to City maintenance of that sanitary sewer easement across UNM's South Golf Course granted by UNM to the City on November 20, 1984, except that if any emergency arises the City shall only be required to notify the Golf Course Director of the need to make repairs and coordinate to the extent possible to avoid disruption of golf play.

6. Except as specifically amended herein, all other terms of the Original Agreement shall remain in full force and effect.

CITY OF ALBUQUERQUE

Approved By:

ATTEST:

_____
Chief Administrative Officer

_____
City Clerk

Approved as to form:

_____
Director, Aviation Department

_____
Assistant City Attorney

Reviewed by:

_____
Director, Water Resources Department

_____
City Attorney

_____
City Engineer

UNIVERSITY OF NEW MEXICO:

By:_____
Title:_____

_____
University Counsel

RECEIVED 1536
JUL 19 1983
CITY ENGINEER

CITY OF ALBUQUERQUE

ALBUQUERQUE, NEW MEXICO

INTER-OFFICE CORRESPONDENCE July 15, 1983 REF. NO._____

TO: C. Dwayne Sheppard, Acting City Engineer-MDD

FROM: R. L. Pratte, Director of Aviation

SUBJECT: ~~Executed Storm Drain Agreement~~ (Cutter Flying Service, Inc.)

Enclosed is one (1) executed copy of the ~~Storm Drain Agreement~~ for your reference and files.

RLP/mb
Enclosure

Copy for Bill Kearney, Please
OK FS

RSH DES
HRO INSP
CME HYDRO Fred
JNH SUR
SAD ADM
FILE
RESPONSE BY

Fred Aguirre is to be the Contact on this Inspection will Probably be contacted by Fred for Construction Inspection 7/19/83

STORM DRAIN AGREEMENT

WHEREAS, the CITY OF ALBUQUERQUE, a municipal corporation ("City"), is the owner of a parcel of land generally described as Sites 1 and 2 in Area A, South General Aviation Area, Albuquerque International Airport and more particularly described in Exhibit "A" to this Agreement; and

WHEREAS, CUTTER FLYING SERVICE, INC. ("Cutter") has leased Sites 1 and 2 by the Lease Agreement of November 14, 1980, as amended ("Lease"); and

WHEREAS, Cutter has plans to construct a hangar building ("Hangar Building") on Site 2; and

WHEREAS, the Hangar Building would have been located over an existing storm drain line ("Existing Storm Drain Line"); and

WHEREAS, the parties could not come to an agreement regarding liability in the event the Hangar Building was located over the Existing Storm Drain Line; and

WHEREAS, the parties have agreed that Cutter will remove and relocate the Existing Storm Drain Line, and the parties will share the cost of relocating the Existing Storm Drain Line;

NOW, THEREFORE, in consideration of the premises and mutual obligations in this Agreement, the parties agree as follows:

Section 1. Cutter will remove and relocate the Existing Storm Drain Line ("Project"), including, but not limited to, the

design and construction of the Project, within the period of time permitted under the Lease for the construction or installation of all improvements and facilities thereunder, subject to the approval of final construction plans by the City Engineer and Cutter's obtaining all permits required by law and subject to applicable provisions of the Lease, including, but not limited to, the provision regarding a Performance Bond. At the time of acceptance by the City, as defined in Section 2 of this Agreement, Cutter shall turn the relocated storm drain line ("Relocated Storm Drain Line") over to the City free and clear of all claims, encumbrances, and liens, and furnish a bond, in a form and with a surety satisfactory to the City, to guarantee the Relocated Storm Drain Line against defective materials and workmanship for a period of three (3) years following the date of acceptance by the City. Upon the completion of the Relocated Storm Drain Line, Cutter will notify the Aviation Department of the City in writing.

Section 2. Within approximately one (1) week of receipt by the Aviation Department of written notification of completion of the Relocated Storm Drain Line, a representative of the Engineering Division of the Municipal Development Department of the City will inspect the Relocated Storm Drain Line. Upon the completion of the inspection by the City of the completed Relocated Storm Drain Line and upon the condition that the Relocated Storm Drain Line has been completed and turned over in a manner satisfactory to the City, the City shall accept the

Relocated Storm Drain Line. Upon acceptance of the Relocated Storm Drain Line by the City, the City shall assume dominion and control and the sole responsibility for maintenance of the Relocated Storm Drain Line, except as provided for by the guarantee as set forth in Section 1 of this Agreement. Upon acceptance of the Relocated Storm Drain Line by the City and presentation of properly documented invoices for payment, the City will reimburse Cutter in the form of Lease rental credits of Five Thousand Two Hundred Ninety-One Dollars and Fifteen Cents ($5,291.15) per month until the lesser of the amount of (a) Thirty-Nine Thousand Five Hundred Dollars ($39,500.00), or (b) the total cost of the Project, including, but not limited to, all pre-construction expenditures associated with the Project, less the amount of Three Thousand Dollars ($3,000.00) has been credited to Cutter's Lease rental payments. The balance of the costs of the Project will be borne by Cutter.

Section 3. This Agreement shall not be assigned except with the written consent of the parties to this Agreement and the express written concurrence of any surety which issued the bonds required in this Agreement. If so assigned, this Agreement shall extend to and be binding upon the successors and assigns of the parties to this Agreement.

Section 4. Cutter is an independent contractor, not an employee, of the City. Cutter shall indemnify the City and procure and maintain insurance in the manner provided for in Article VII,

Sections 2 and 12, of the Lease for the period of time required by Cutter to complete construction of the Project.

IN WITNESS WHEREOF, the parties have executed this Agreement on this 14th day of July, 1983.

CUTTER FLYING SERVICE, INC.

By _____
Kenn Holzer
Vice President

CITY OF ALBUQUERQUE, NEW MEXICO

By _____
Frank A. Kleinhenz
Chief Administrative Officer

ATTEST:

_____
City Clerk

REVIEWED AND RECOMMENDED BY
THE AVIATION DEPARTMENT:

_____
R. L. Pratte, A.A.E.
Director

REVIEWED AND RECOMMENDED BY
THE ENGINEERING DEPARTMENT:

By _____
C. Dwayne Sheppard
Acting City Engineer

REVIEWED BY THE LEGAL DEPARTMENT:

By _____
Assistant City Attorney

SITE DESCRIPTION
(F.B.O. Site #1)

E4

A SITE SITUATED WITHIN THE GENERAL AVIATION AREA AND LOCATED WITHIN THE ALBUQUERQUE INTERNATIONAL AIRPORT, ALBUQUERQUE, NEW MEXICO and being situate totally within "Tract G" of "Municipal Addition No. 9", a subdivision of Albuquerque, New Mexico, a replat of Municipal No. 3, and of Blocks 18, 19, 20, 22, 23, & 24, and a portion of Blocks 12, 13, 14, 15, 16, and 17 of Knob Heights Addition and a plat of lands in Sections 27, 34, 35, and 36, Township 10 North, Range 3 East, and in Sections 2, 3, and 4, Township 9 North, Range 3 East, N.M.P.M., Bernalillo County, New Mexico.

A plat of said Municipal Addition No. 9 being filed in the Bernalillo County Clerk's Office in Volume D3 Folio 105 on February 5, 1965.

The site is more particularly described as follows:

COMMENCE at City of Albuquerque Control Station (1-N16) located 1050 feet northwest of edge of Runway #21-03 and 75 feet east of fence line running on east side of road. Station is a standard City of Albuquerque brass cap set in concrete and stamped "1-N16" from whence a 2" iron pipe with cap marking section corners 3, 4, 9, & 10 of Township 9 North, Range 3 East, N.M.P.M., Bernalillo County, New Mexico bears S03°28'49"W, for a distance of 1,707.39 feet.

THENCE: N08°09'08"E, 1123.27 feet to the Point of Beginning of the herein described site and which is also the southwesterly corner of a concrete slab.

THENCE: N17°46'47"E, 350.00 feet along the edge of concrete slab.

THENCE: Along a curve to the right having a radius of 537.76 feet and a deflection angle of 36°00'00". The said curve's long chord bearing being N35°46'47"E, for a distance of 332.35 feet. The described curve running along the edge of taxiway.

THENCE: S36°13'13"E, 494.85 feet.

THENCE: S53°46'47"W, 16.62 feet.

THENCE: S17°46'47"W, 61.80 feet.

THENCE: S72°13'13"E, 42.76 feet.

THENCE: S17°46'47"W, 250.00 feet.

THENCE: N72°13'13"W, 42.76 feet.

THENCE: S17°46'47"W, 56.85 feet.

THENCE: S71°46'47"W, 185.27 feet along the back of sidewalk.

THENCE: Along a curve to the right having a radius of 552.33 feet and a deflection angle of 4°30'15". The said curve's long chord bearing being S74°01'54"W for a distance of 43.41 feet. The described curve running along the back of sidewalk.

THENCE: Along a curve to the right having a radius of 15.00 feet and a deflection angle of 47°03'10". The said curve's long chord bearing being N36°44'48"W, for a distance of 11.98 feet. The described curve running along the back of curb of the auto parking.

THENCE: N13°13'13"W, 60.50 feet along the back of curb of the auto parking area.

THENCE: Along a curve to the right having a radius of 480.83 feet and a deflection angle of 44°27'17". The said curve's long chord bearing being N80°59'35"W, for a distance of 363.78 feet. The described curve running along the back of curb of the auto parking.

THENCE: N58°45'56"W, 50.00 feet.

THENCE: N36°17'10"E, 131.72 feet.

THENCE: S72°13'13"E, 100.00 feet to the Point of Beginning. Being 7.889 acres, more or less, in area.

EXHIBIT "A"

SITE DESCRIPTION
(F.B.O. Site #2)

85

A SITE SITUATED WITHIN THE GENERAL AVIATION AREA AND LOCATED WITHIN THE ALBUQUERQUE INTERNATIONAL AIRPORT, ALBUQUERQUE, NEW MEXICO and being situate totally within "Tract G" of "Municipal Addition No. 9", a subdivision of Albuquerque, New Mexico, a replat of Municipal No. 3, and of Blocks 18, 19, 20, 22, 23, & 24, and a portion of Blocks 12, 13, 14, 15, 16, and 17 of Knob Heights Addition and a plat of lands in Sections 27, 34, 35, and 36, Township 10 North, Range 3 East, and in Sections 2, 3, and 4, Township 9 North, Range 3 East, N.M.P.M., Bernalillo County, New Mexico.

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THENCE: N23°06'38"E, 1977.48 feet to the Point of Beginning of the herein described site and which is also the northwesterly corner of a concrete slab.

THENCE: N89°46'47"E, 350.00 feet along the edge of concrete slab.

THENCE: Along a curve to the right having a radius of 537.76 feet and a deflection angle of 36°00'00". The said curve's long chord bearing being S72°13'13"E, for a distance of 332.35 feet. The described curve running along the edge of taxiway.

THENCE: S35°46'47"W, 494.85 feet.

THENCE: N54°13'13"W, 16.62 feet.

THENCE: S89°46'47"W, 61.80 feet.

THENCE: S0°13'13"E, 42.76 feet.

THENCE: S89°46'47"W, 250.00 feet.

THENCE: N0°13'13"W, 42.76 feet.

THENCE: S89°46'47"W, 61.80 feet.

THENCE: S53°46'47"W, 16.62 feet.

THENCE: N36°13'13"W, 494.85 feet.

THENCE: Along a curve to the right having a radius of 537.76 feet and a deflection angle of 36°00'00". The said curve's long chord bearing being N71°46'47"E for a distance of 332.35 feet. The described curve running along the edge of taxiway to the Point of Beginning.

Being 8.352 acres, more or less, in area.

City of Albuquerque

P.O. BOX 1293 ALBUQUERQUE, NEW MEXICO 87103

October 19, 2000

Kent S. Freier
Molzen-Corbin & Associates
2701 Miles Road SE
Albuquerque, New Mexico 87106

RE: Airport Car Rental Expansion Letter dated October 17, 2000

Dear Mr. Freier:

I have reviewed the referenced letter and forward the following comments.

1. I have received the necessary correspondence on the landfill.

2. The drainage easement you faxed me appears to cover the pond and the access road. I was hoping the easement would address the maintenance responsibility.

3.The airport will be responsible for the construction and maintenance of the conservation measures above the pond.

4.The easements through the Schwartzmann and UNM property is included in the Car Rental Infrastructure List.

5. Since the storm drains were constructed at the same elevation in the pond and a low flow channel would have to be flat. Also, reconstruction of the storm drain would be very costly. Therefore, because of these reasons a low flow channel should not be constructed.

6. I have attached a portion of the DPM on pond design. The DPM indicates that gates and fencing is required.

7. You have explained my question.

8. I have received the Highway Department memos.

9. The maintenance matrix appears to reflect the discussions that were held during the DRC

meetings.

10. We are expecting as-builts from BPLW for the site grading and paving and each new building, and from Greiner as-builts for University Boulevard and the Chavez storm drain.

11. I agree on the comment for the floodplain.

Hydrology can sign a temporary Certificate of Occupancy for the main Car Rental Building. Hydrology can sign a Certificate of Occupancy for all the minor buildings after we receive the as-builts on the site and buildings. BPLW will need to coordinate with you on the as-builts.

Sincerely,

Carlos A. Montoya

c: Fred Aguirre, City Engineer
Dan Hogan, Division Manager

accumulations on adjacent property and in public facilities is the responsibility of the contractor. Failure to do so promptly may result in a "stop-work order" being issued and maintained until repair and clean-up is completed to City Engineer satisfaction. All exposed earth surfaces must be protected from wind and water erosion prior to final City acceptance of any project. The continued maintenance of these protective measures is the responsibility of the owner or his assigns.

2. Phased Construction:

Areas graded in conjunction with phased projects, but not left in their permanent condition must be protected during the interim from wind and water erosion.

F. Means of Erosion Control

There are numerous mechanical and vegetative methods for preventing soil erosion. The U.S. Environmental Protection Agency Publication EPA-R2-72-OIS Guidelines for Erosion and Sediment Control Planning and Implementation and the local U.S.D.A. Soil Conservation Service Office can provide numerous, inexpensive and effective erosion management techniques.

G. Flat Grading Scheme

A flat grading scheme will be allowed only in the Valley region (within the Conservancy District) of Albuquerque and under the following conditions:

1. The site must be flat or can be graded flat.
2. No existing off-site flows enter the site.
3. The maximum pad size may not be greater than 45% of the lot size or the sum of all the impervious area contributing to the lot (including the street and sidewalk) is not greater than 45%.
4. Pad elevation shall be a minimum of one (1) foot above the 100 year 10-day storm water surface elevation.
5. The flow between the front yard and backyard cannot be obstructed. The storm water must be allowed to equalize to the same level between the front yard and backyard.
6. A permanent perimeter wall or barrier around the development is required to contain the 100 year 24 hour storm developed runoff.
7. The high point of the street should be 4 inches above the 100 year 10-day storm water surface elevation.
8. There are no accessable storm drains near the proposed development.

H. Pond Design (City Maintained Facilities)

1. Access: Access into facility shall be opposite outlet if possible with a minimum width of 12 feet. Maximum access slope shall be 10:1 or flatter (6:1 if hard surfaced with soil cement or concrete treated base). Standard design tube or pipe gates shall be installed to restrict vehicle access. Gates shall be set back 50 feet from arterial or collector streets so equipment does not have to park in the street.
2. SPILLWAYS: Emergency spillways shall always be provided, be erosion resistant, and discharge to a public right-of way, drainage easement and/or historic flow path.
3. OUTLETS:

a. Facility outlets shall always be gravity flow whenever feasable and located in a corner or accessible edge of facility, opposite of facility access point if possible. Outlet pipe shall be a minimum of 12 inches in diameter with a slope such that when flowing at 1/4 full, velocity is 3 fps or greater.

b. The outlet should be surrounded by a stablized grade pad appropiately sized for maintenance.

4. POND BOTTOMS:

a. Facility bottoms shall be designed to convey nuisance flows from the inlet to a storm water pollution prevention feature (such as a pervious bottom area for infiltration) prior to discharging to the outlet. Ease of maintenance shall be a consideration in all dams/detention basins. A feature [such as a hardened pad (having minimum dimensions of 3' by 6" thick with a 1" invert), low flow channel, etc.] shall be considered to allow maintenance crews a non-saturated, hardened surface to perform maintenance and provide a grade check in the bottom of the basin. Each dam/detention basin should be evaluated with regard to such features as ease of maintenance, water quality, desirability of vegetation and habitat, effect on neighborhoods (odors, mosquitoes, vectors), stability/safety of the foundation and embankment, well wash water and possible recharge.

b. The minimum pond bottom slope is 0.5%. Pond designs with bottom slopes greater than 1.0% may not require a hardened pad subject to final design review at the Design Review Committee (DRC).

5. SIDE SLOPE AND BOTTOM TREATMENTS:

a. Vegetation will be accepted if seeded per the City of Albuquerque Standard Specifications for Public Works Construction.

b. Aggregate or riprap may be used as an erosion control mulch for 3:1 and steeper slopes.

c. A geotechnical investigation and report may be required at the discretion of the City Engineer.

6. MINIMUM POND SIZE

In order for a pond to be publicly maintained, it must be a minimum of two (2) acre-feet.

7. FENCING:

a. Detention ponds 18 inches or greater in depth will require fencing unless side slopes are 3:1 or flatter and the pond drains in 96 hours or less.

b. If fencing is required, the minimum height is 42 inches. All fencing shall conform with the City of Albuquerque Standard Specifications for Public Works Construction.

c. Fencing as required above and as deemed appropiate based upon the circumstances (size, configuration, nature of flows entering, location, etc.) by the City Engineer.

8. All detention ponds must be evacuated in twenty four (24) hours or less, unless discharge is limited by downstream constraints. In any event, all ponds should be evacuated within 96-hours. Ponds that take more than six (6) hours to drain will be designed for a design storm equal to or exceeding the evacuation time. No percolation credit for volume reduction will be given.

Bob Heil

REVIEW OF DRAINAGE

UNIVERSITY SOUTH GOLF COURSE

ALBUQUERQUE INTERNATIONAL AIRPORT

PREPARED FOR

UNIVERSITY OF NEW MEXICO

UNIVERSITY LANDSCAPE ARCHITECT

ALBUQUERQUE, NEW MEXICO

MARCH 1983

PREPARED BY

DENNEY-GROSS & ASSOCIATES, INC.

ALBUQUERQUE, NEW MEXICO

I, Louis W. Gross, hereby certify that the enclosed documents and drawings were prepared under my supervision and are true and correct to the best of my knowledge and belief.

Louis W. Gross

New Mexico Registered Professional
Engineer & Land Surveyor No. 4100

I. INTRODUCTION:

The scope of this study is as follows:

A. Review the published drainage report, "Albuquerque International Airport/University Golf Course Drainage Study".

B. Confer with University Staff to establish a history of events concerning storm drainage and to establish criteria for alternative solutions.

C. Perform a field inspection with golf course staff to make measurements, obtain photographs, and record staff imput.

D. Prepare alternative schemes for handling the storm water through the golf course. Make comparative cost estimates for each scheme.

E. Prepare an analysis of the contributing watershed to determine effects of recent and future development on the sediment transport and flows.

II. REVIEW:

We have reviewed the published drainage report, "Albuquerque International Airport/University Golf Course Drainage Study". The report addresses the hydrologic aspects of the study area using standard engineering procedures. We did not check watershed parameters or the actual computations.

The flows and volumes of storm waters for the various events and developmental stages from the report are summarized below for ease of view.

Year Storm	Tc (min)	T (min)	Q (cfs)	V (Ac.Ft.)	V_{D/V_E}	Runoff Inches	Total Rainfall	Runoff Factor
2E	0.60	1.80	12.3	1.443		.08	1.00	.08
2D	0.54	1.63	29.8	2.844	1.971	.13	1.00	.13
5E	0.54	1.62	32.7	3.268		.18	1.30	.14
5D	0.54	1.62	56.5	5.703	1.745	.27	1.30	.21
10E	0.54	1.62	48.5	4.883		.27	1.50	.18
10D	0.54	1.62	78.3	7.916	1.621	.37	1.50	.25
25E	0.52	1.56	76.6	7.572		.42	1.80	.23
25D	0.42	1.26	152.0	11.516	1.521	.54	1.80	.30
50E	0.42	1.26	156.4	11.170		.61	2.15	.28
50D	0.48	1.02	225.5	16.187	1.449	.76	2.15	.35
100E	0.38	1.14	197.8	12.838		.70	2.30	.30
100D	0.50	1.50	238.3	18.316	1.427	.86	2.30	.37

Area Existing = A_E = 0.342 sq.mi. = 218.8 Acres, Impervious Area = 0.072 sq.mi. = 46.1 Ac.

Area Developed = A_D = 0.398 sq.mi. = 254.7 Acres, Impervious Area = 0.133 sq.mi. = 85.1 Ac.

From this summarization it can be seen that the development has drastically increased the peak flows and volume of flows. This is more evident in the storms which occur more frequently such as the 2-year and the 5-year.

While standard procedures were used to compute the sediment transport, we have doubts that any procedure will produce valid results on this particular watershed. The construction of the pavement and storm sewer improvements have caused a rapid deterioration of the drainage courses below the piped outlets. The photos on Photo Sheets 1 through 5 illustrate the bottom deterioration, side sloughing and side rivlet erosion. The concentrated flows of clean water from the two storm drain systems have been released into the steep sloped arroyos. The soil comprising the arroyo

bottoms and sides is a course sand. The sand is highly erodable and subject to sloughing at side cuts. These conditions cause rapidly changing drainage courses and erosion patterns.

The history of events concerning storm damage on the Golf Course contained in the report check with information we have gathered.

III. OBSERVATIONS AND CONCLUSIONS:

1. Fairways 2 and 8 were constructed across the arroyo under study. No channels or conduits were installed at the time the Golf Course was constructed to convey storm water. The dike that was constructed in 1966 provided some protection to Fairway 2 until 1980. The General Aviation Facility construction was completed in the fall of 1979. The two largest total depth rainfalls occurred in 1980 and 1981.

2. Fairway 8 is being damaged mainly by sedimentation. This is mainly due to the dikes that were downstream of Fairway 8 slowing down and backing up of the silt laden water. If the dikes had not been there, erosion would have occurred across Fairway 8. Fairway 2 is being damaged mostly by erosion.

3. The type of material which has settled on Fairway 8 is a course sand. We did not observe any substantial erosion of the fine sandy soils on top of the mesa. We would conclude that almost all this sediment was transported from below the pipe outlets.

4. We believe that a comparison of peak flows and volumes of runoff is a better indication of the effects of the General Aviation Improvements than sedimentation yield computations.

5. The erosion protection recently installed below the outlets of the storm drains is a temporary measure to protect the pipes. Substantial erosion is beginning to occur below the protection (see Photo Pages 4 and 5).

6. Erosion is beginning to damage Fairway 2 on the west edge. Also, lateral erosion is threatening Fairway 17. (See Photo Page 6.)

7. Based on observations during this study, storm waters from this watershed should be conveyed from the outlets of the storm drains to the south diversion channel by hard lined channel or conduit. This would include future storm water collection systems between the top of the mesa and the golf course and between I-25 and the south diversion channel. Photo Pages 7 and 8 show the deterioration of the channel above and below I-25. The concrete boxes under I-25 are considerably larger than needed for the 100-year flow.

8. The immediate concern is the protection of the golf course. Alternatives for protection of the Golf Course were narrowed to the three following schemes for this report.

A. Convey the 100-year, 6-hour runoff in a conduit from the east boundary of the golf course to the west side of

Fairway 2. The conduit is designed to carry 238.3 cfs peak flow.

B. Convey the 100-year, 6-hour runoff in a conduit under Fairways 8 and 2 with an open channel between the fairways. The facility is designed to carry 238.3 cfs peak flow.

C. Convey the 10-year, 6-hour runoff in a conduit under Fairways 8 and 2 with an open channel between the fairways. The conduits would be designed to carry 78.3 cfs and the channel designed for 238.3 cfs. A swale will be provided to carry flows in excess of 78.3 cfs over the fairways.

For all three alternatives, bank protection is required along Fairway 17.

ESTIMATE ALTERNATE "A":

1.	Concrete Inlet Structure	$ 11,200.00
2.	Inlet Dikes & Riprap	4,000.00
3.	60" RCP (1250 LF @ $94)	117,500.00
4.	Outlet Structure (Stilling Basin)	12,000.00
5.	Wire Enclosed Riprap (120 CY @ $65)	7,800.00
	TOTAL ALTERANTE "A"	$ 152,500.00

ESTIMATE ALTERNATE "B":

1.	Concrete Inlet Structure	$ 11,200.00
2.	Inlet Dikes & Riprap	4,000.00
3.	60" RCP (800 LF @ $94)	75,200.00

4.	Concrete Channel (450 LF @ $80)	$ 36,000.00
5.	Outlet & Inlet Headwalls for Concrete Channel	6,150.00
6.	Outlet Structure (Stilling Basin)	12,000.00
7.	Wire Enclosed Riprap	7,800.00
	TOTAL ALTERNATE "B"	$ 152,350.00

ESTIMATE ALTERNATE "C":

1.	Concrete Inlet Structure	$ 10,500.00
2.	Inlet Dikes & Riprap	4,000.00
3.	48" RCP (1250 LF @ $70)	87,500.00
4.	Grading (swale between fairways)	2,500.00
5.	Outlet Structure (Stilling Basin & Drop)	12,000.00
6.	Wire Enclosed Riprap (120 CY @ $65)	7,800.00
	TOTAL ALTERNATE "C"	$ 124,300.00

8. Alternate Comparisons:

1. Costs

Alternate A	$152,500
Alternate B	152,350
Alternate C	124,300

The costs of each alternate are based on the preliminary design quantities. Unit prices were arrived at by using contracts let during 1982 in the Albuquerque area and adjusting to the south golf course's locale and conditions.

2. Alternate "A" will handle the 100-year event as the watershed is presently developed. Hydraulic entrance conditions governed the amount of flow thus the size of pipe. When more upstream development occurs, this segment of pipe can be connected to the existing upstream storm drainage system with intermediate junction boxes for development between the golf course and the top of the mesa. Also, when more of the mesa top is developed, the drainage can be injected into the same system. In a closed system, the 60-inch pipe would have a capacity over 500 cfs without being a pressurized pipe. Alternate "A" will be compatible with future developments.

Alternate "B" will handle the 100-year event as the watershed is presently developed. The channel between the two pipes breaks the continuity and creates two inlet control conditions. In order for this scheme to handle more flow, additional improvements would be required.

Alternate "C" will handle the 10-year event as the watershed is presently developed. The overflow swales for events of greater than 10-year would require maintenance expenses to repair erosion damage. The 25-year flow, which is approximately twice the 10-year flow, would cause substantial damage.

9. The long terms concern is to provide flood protection for all existing and future development within this watershed. The arroyos provide an excellent corridor for drainage conveyance improvements. The arroyo empties into the south diversion channel thence to the Rio Grande. As future development occurs, the ground cover and storm sewer facilities will reduce the sediment problem substantially. The Alternate A facility will be compatible with future drainage facilities and can be utilized by other upstream owners.

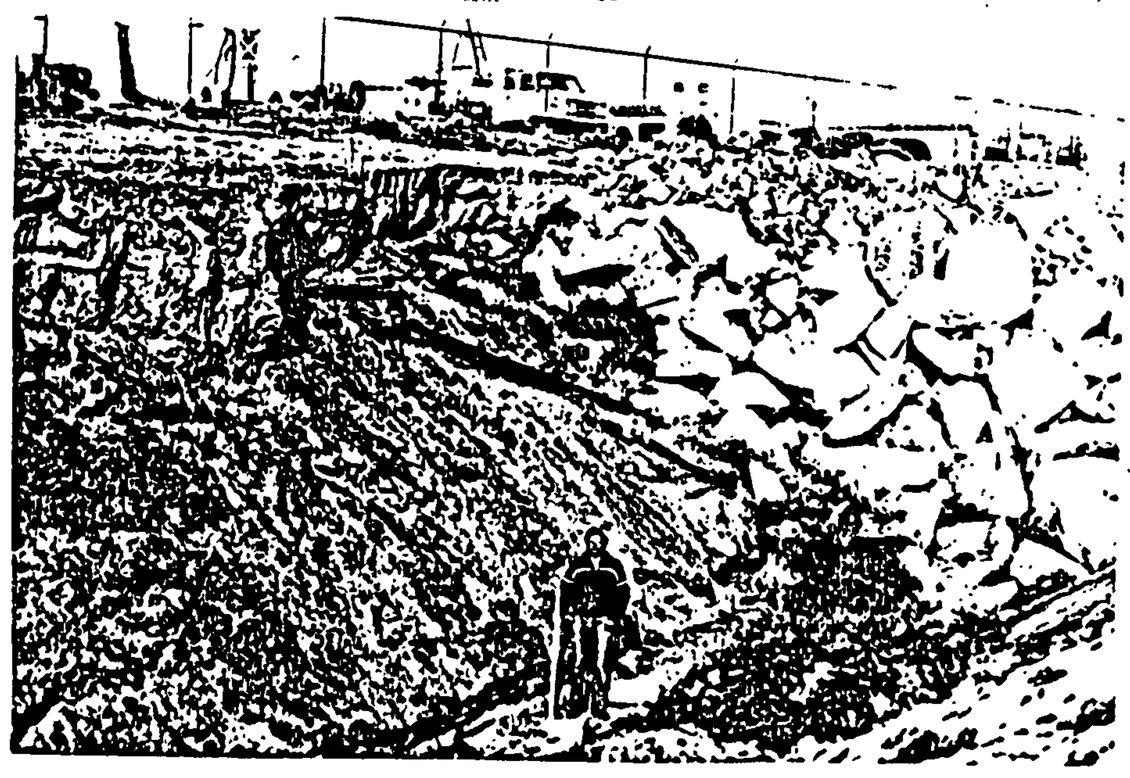
IV. RECOMMENDATIONS:

1. The watershed should be reanalyzed to obtain the 100-year peak flow at the east boundary of the Golf Course to serve as a check on the published peak flow. The drainage facility proposed in Alternate A should be designed and constructed immediately.

2. A master plan for drainage should be prepared for the entire watershed including areas west of I-25.

MAR 83

MAR 83



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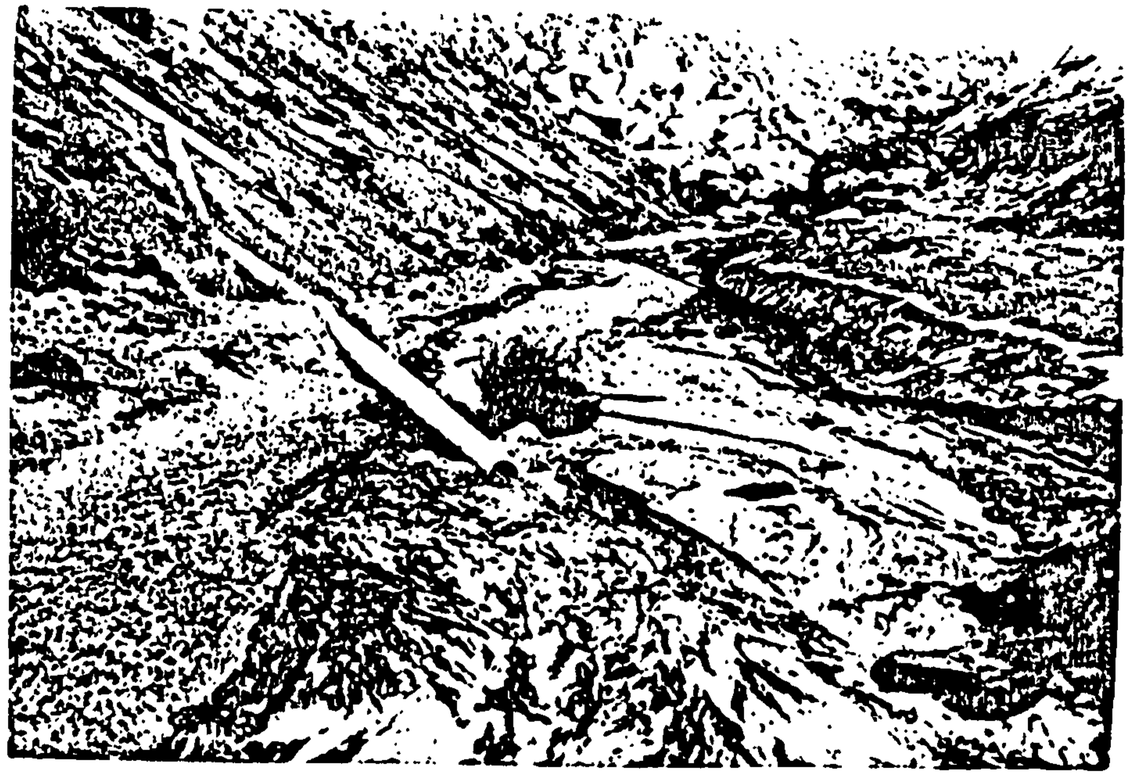
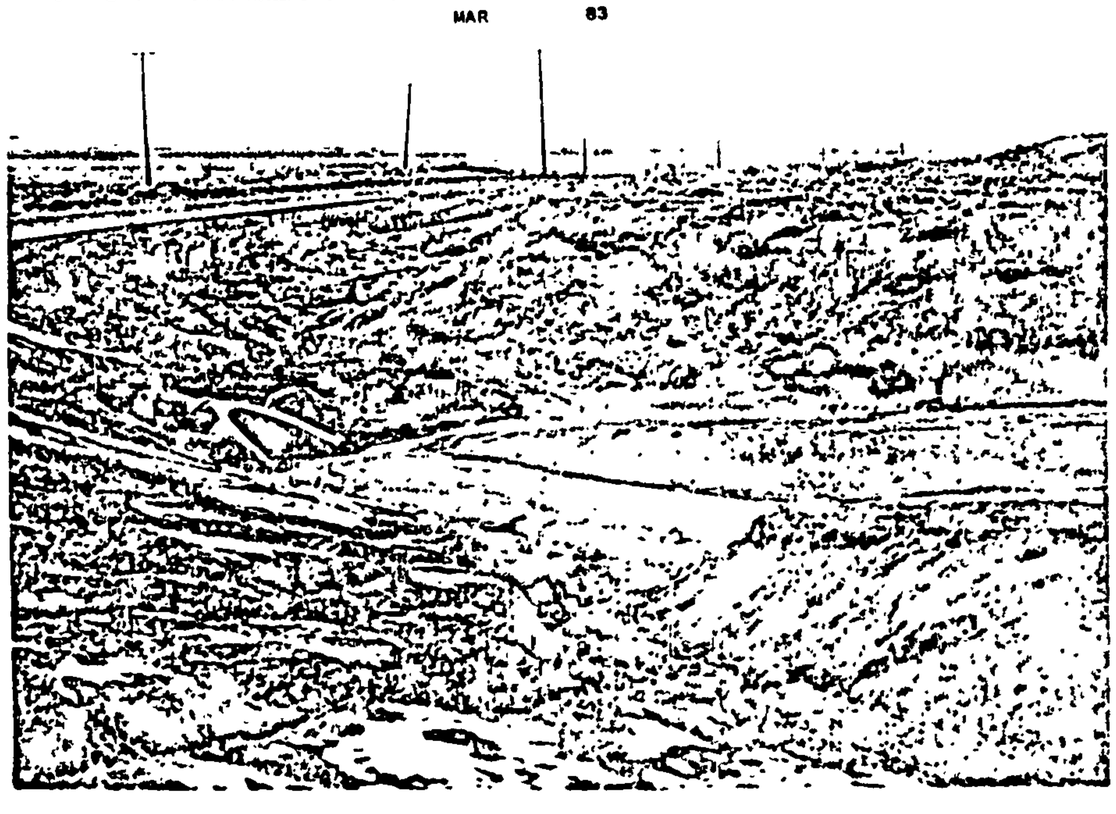
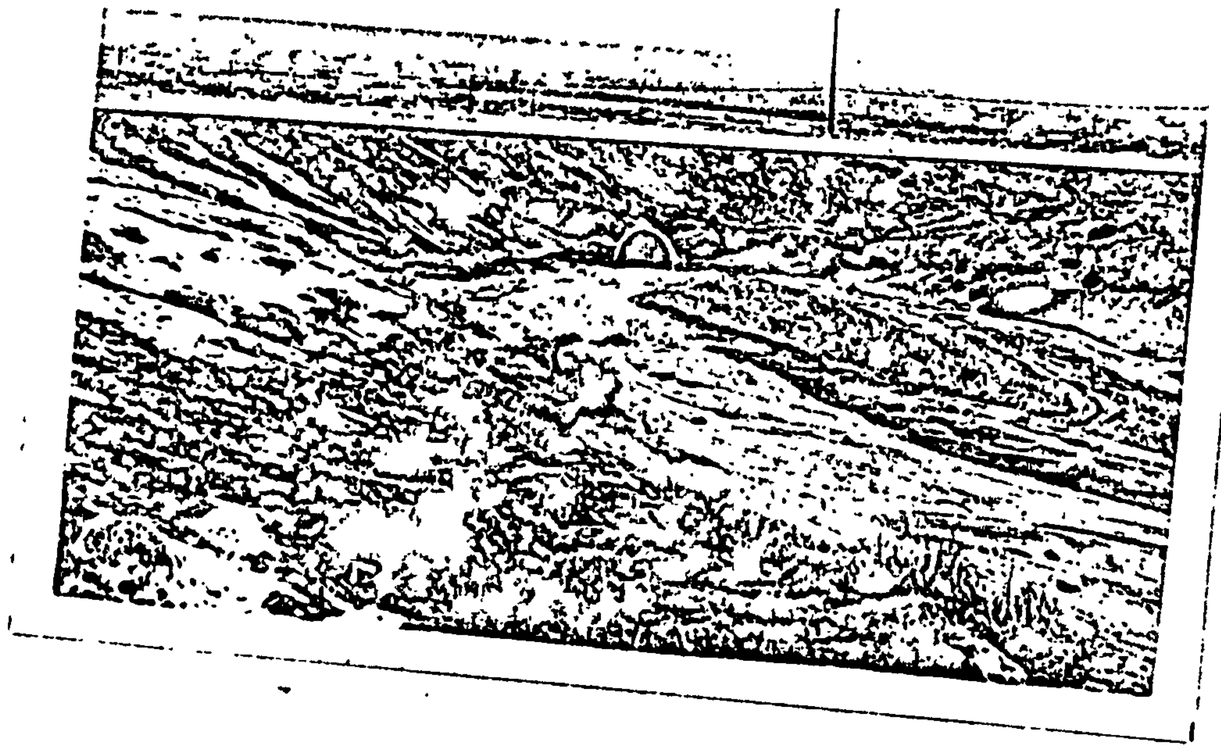


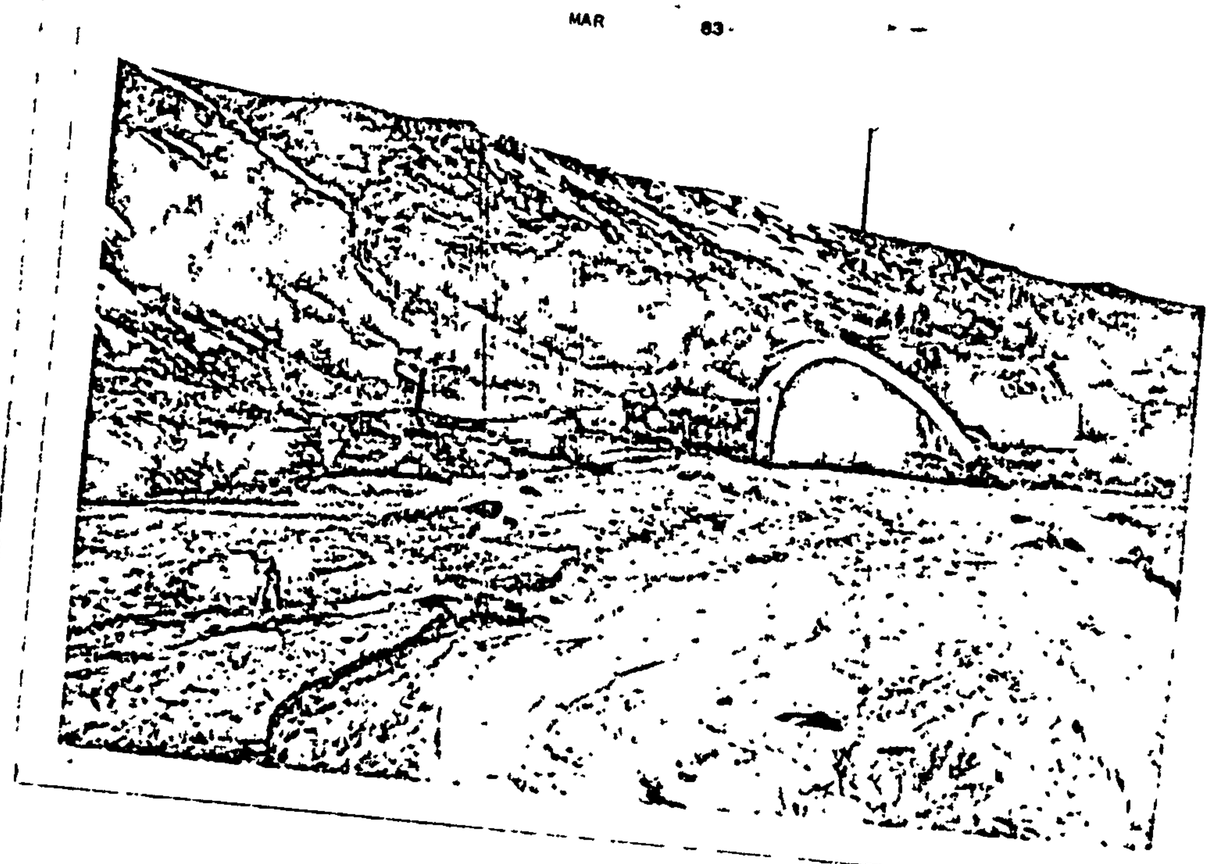
Arroyo Below North Storm Sewer Release
Photo Page 1

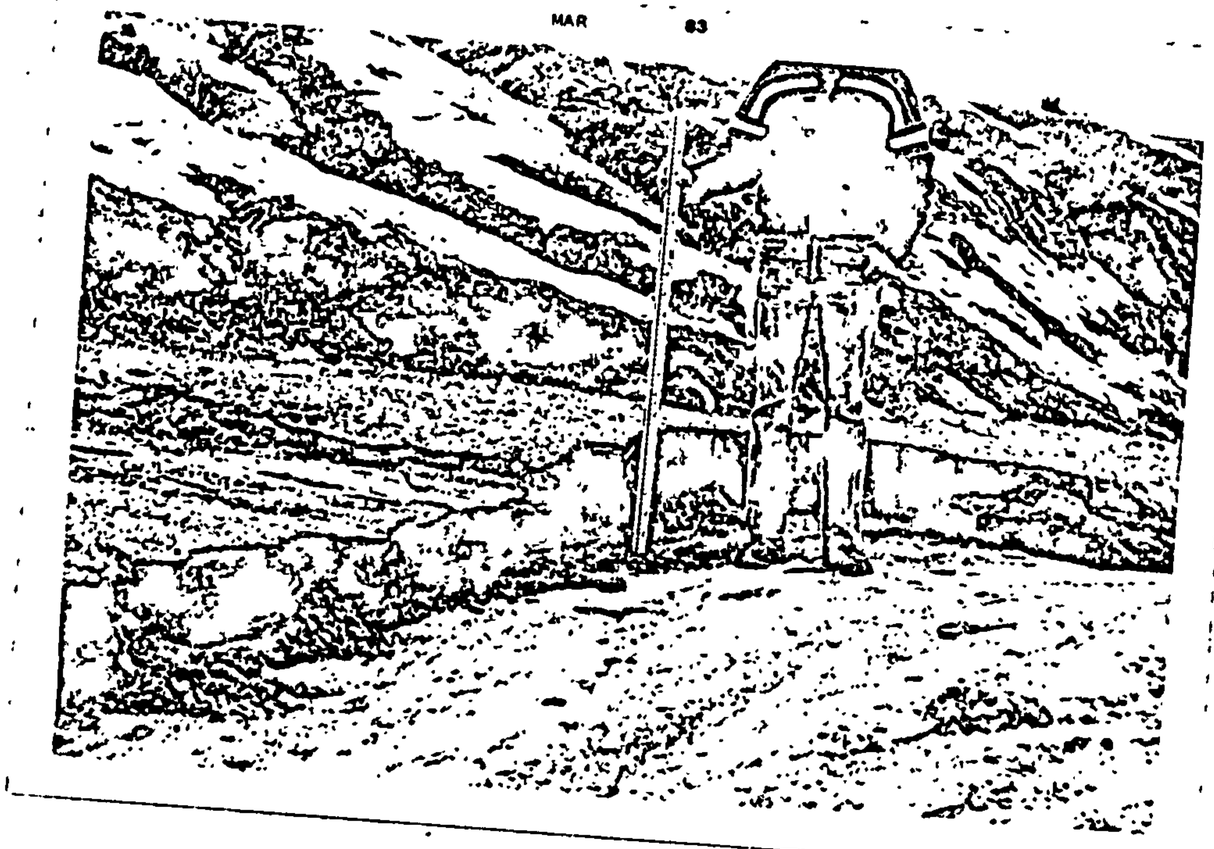
Arroyo Below North Storm Sewer Release

Photo Page 2

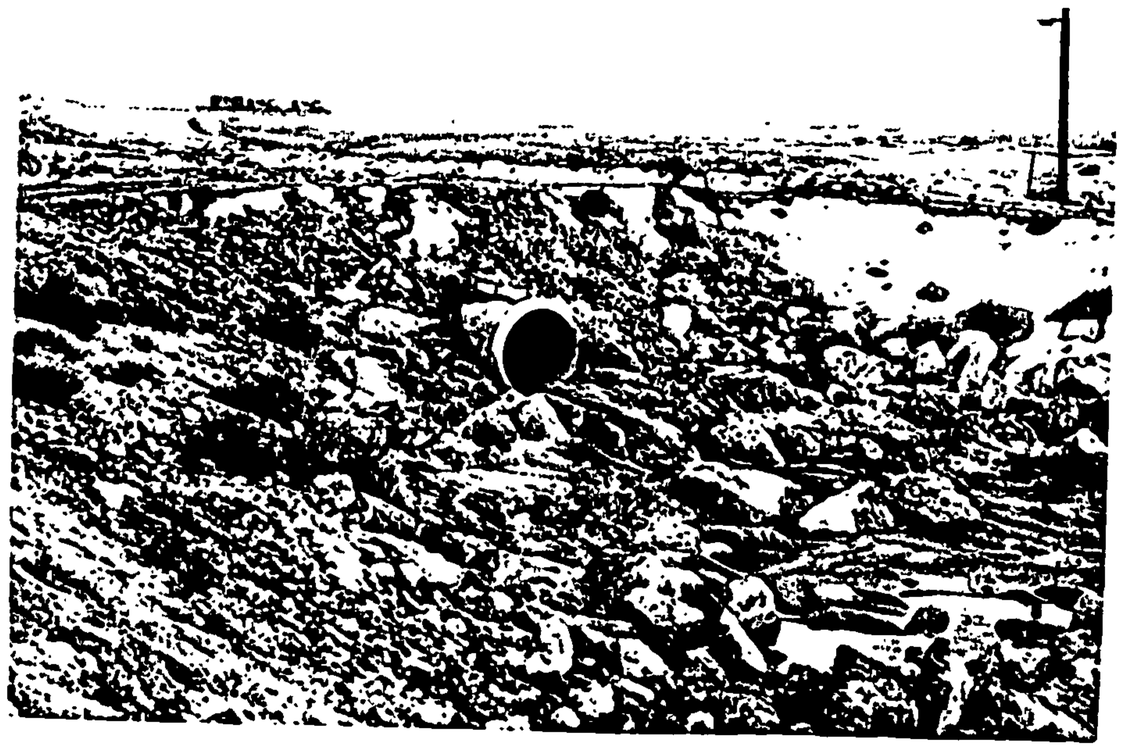








Arroyo Below North Storm Sewer Release
Photo Page 3



Arroyo Below South Storm Sewer Release

Photo Page 4

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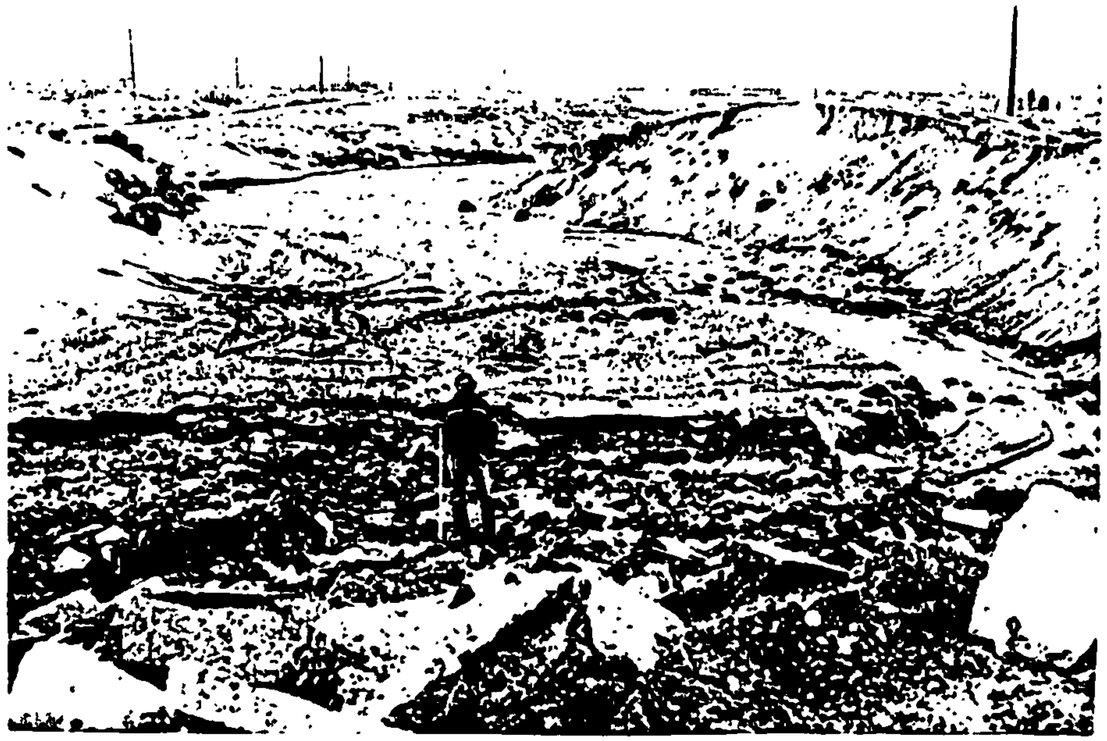

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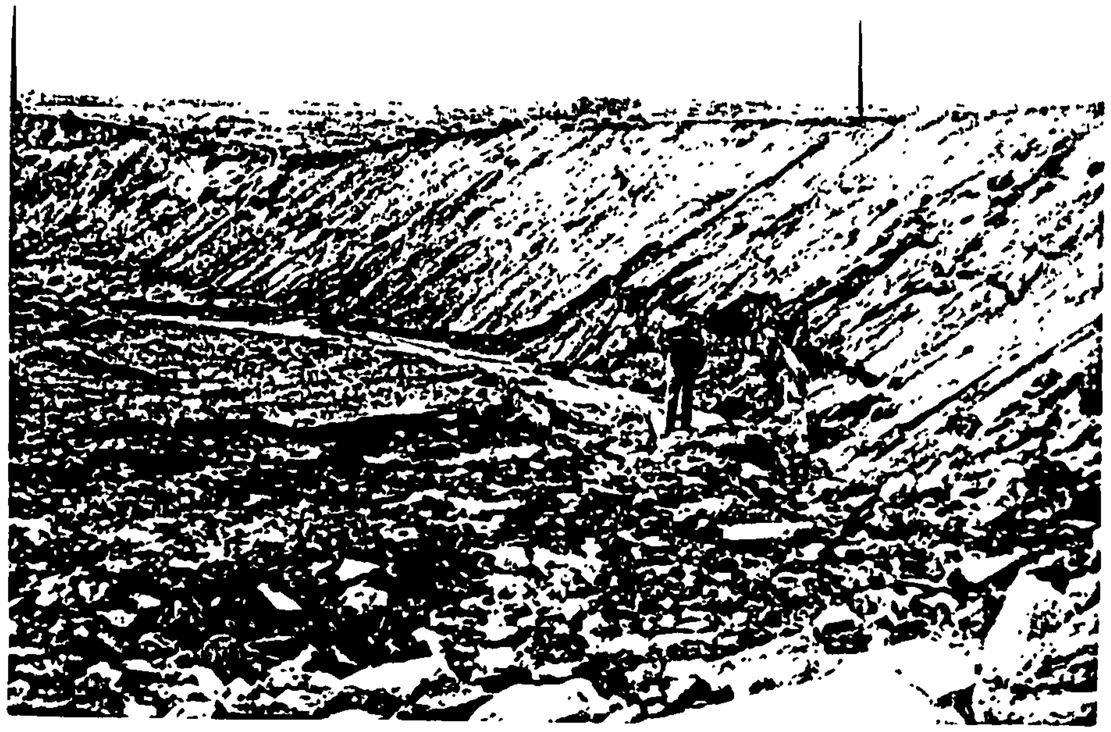

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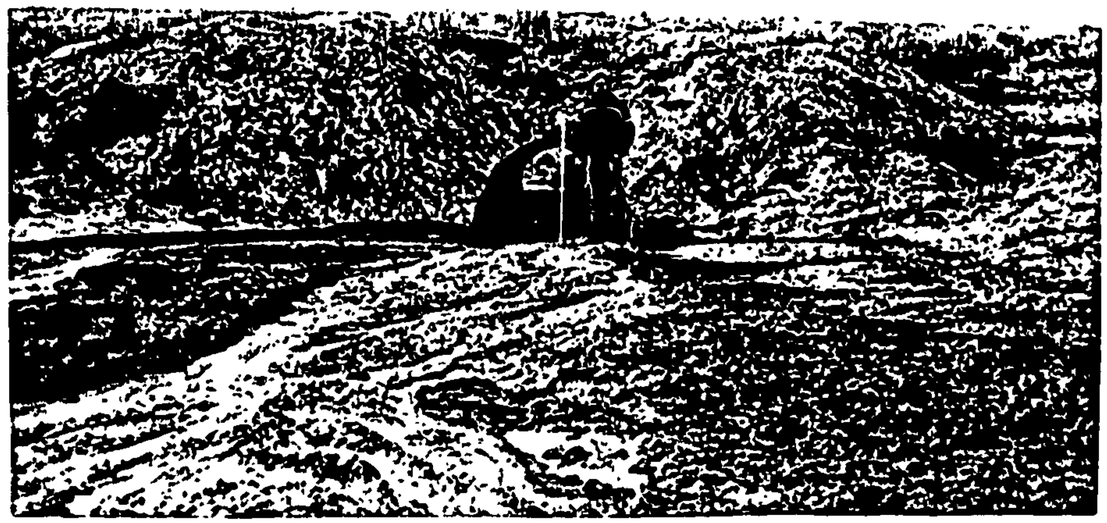
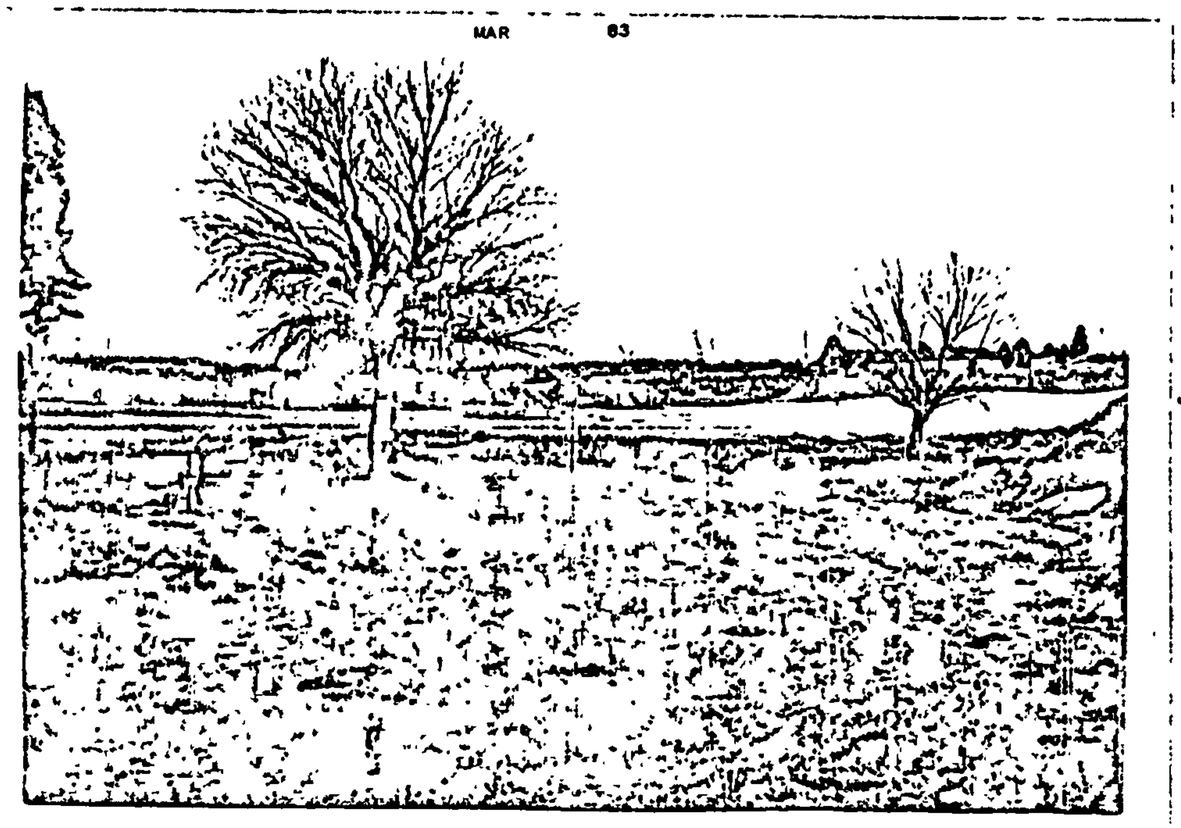


Arroyo Below South Storm Sewer Release

Photo Page 5



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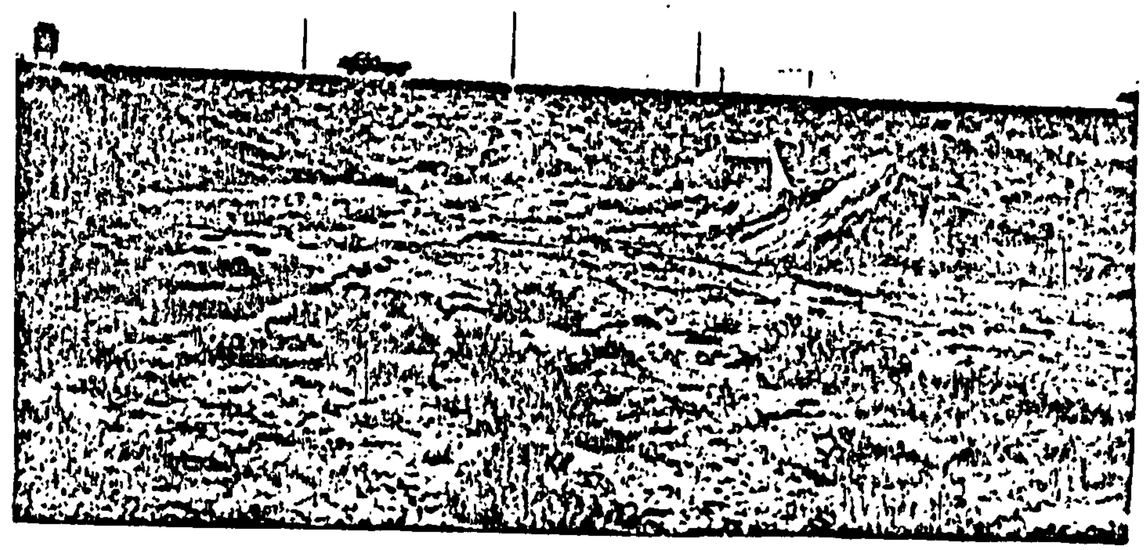
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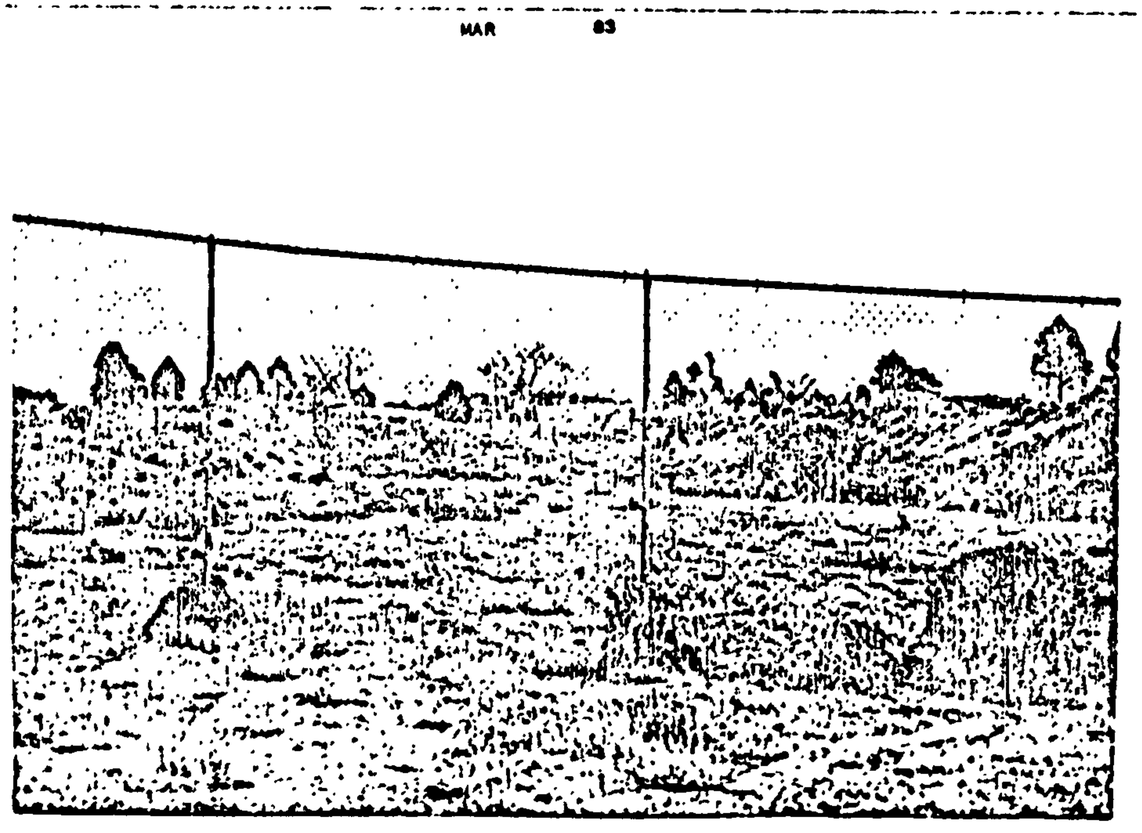
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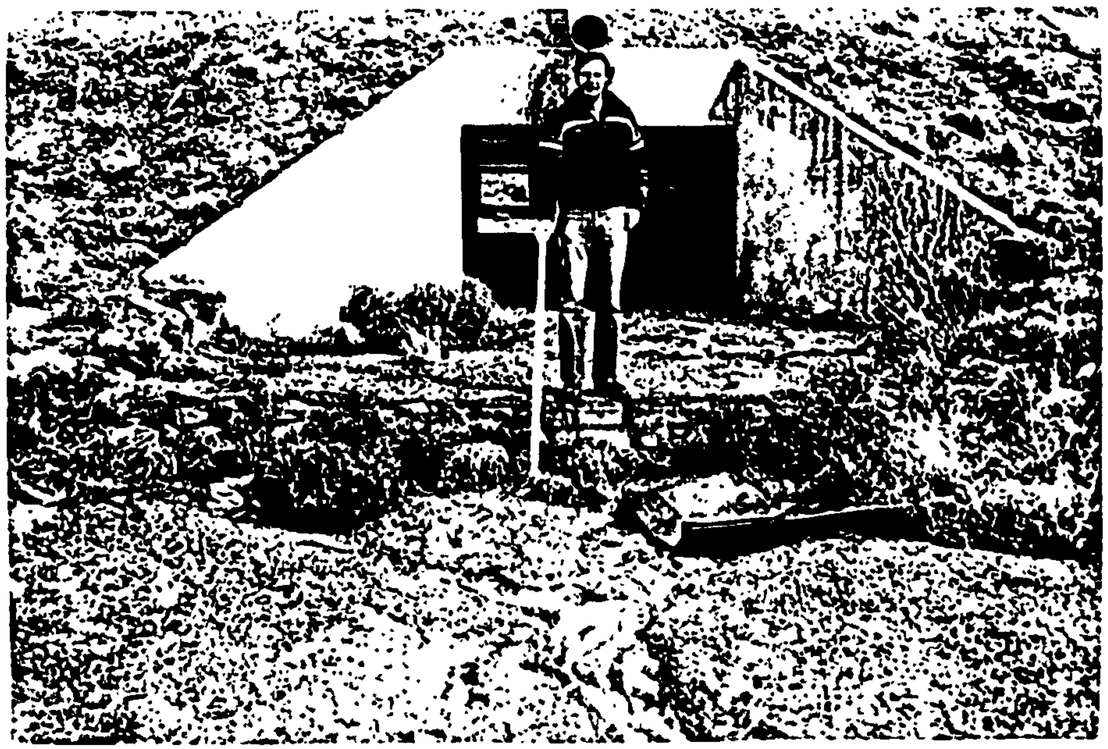

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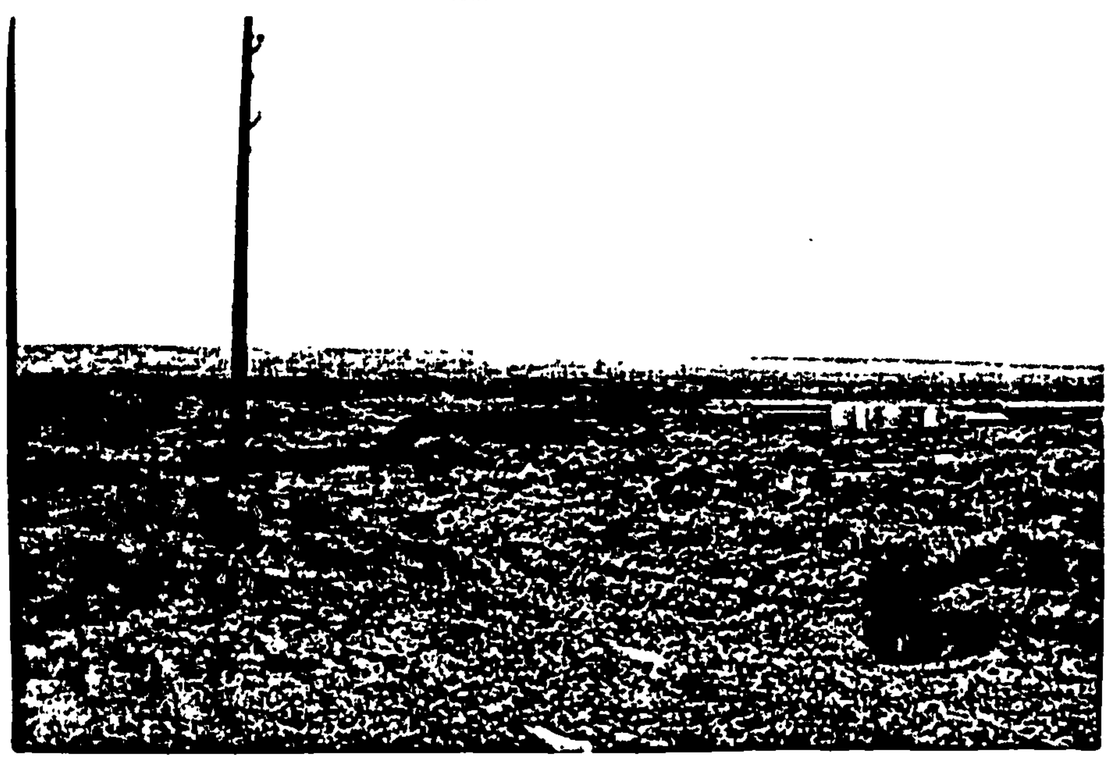

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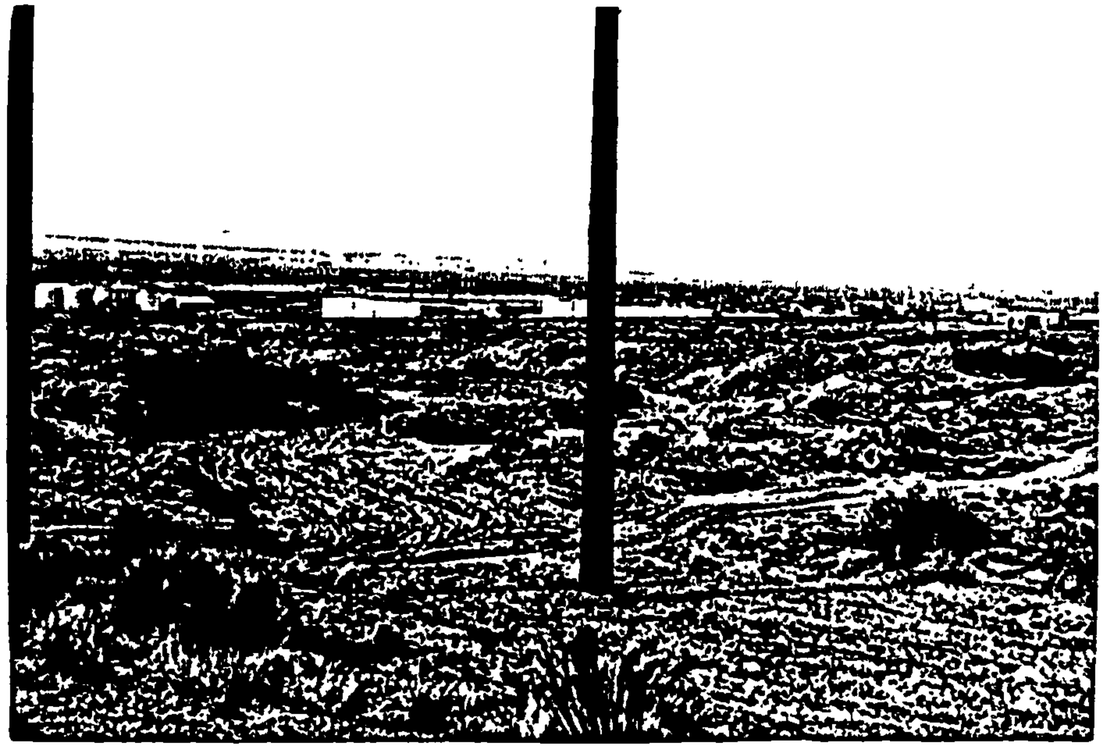


PHOTO-IDENTIFICATION MAP

MOLZEN-CORBIN & Associates
ENGINEERS/ARCHITECTS/PLANNERS

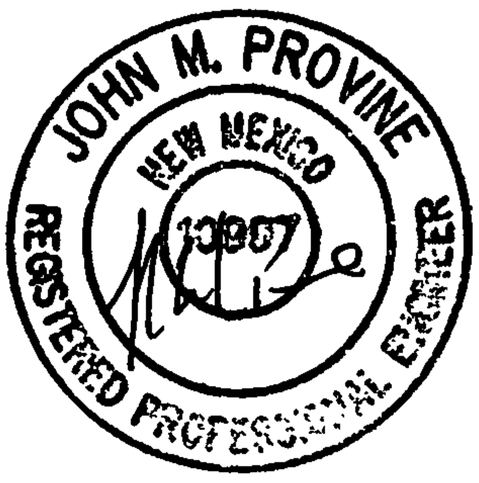
ALBUQUERQUE
INTERNATIONAL AIRPORT

STORM DRAINAGE MASTER PLAN

Project No. 4255.01

City of Albuquerque, NM
Aviation Department

May 1995



MOLZEN-CORBIN & Associates

ALBUQUERQUE INTERNATIONAL AIRPORT

DRAINAGE MASTER PLAN

June, 1994

Prepared for:

City of Albuquerque
Aviation Department
Po Box 9022
Albuquerque, New Mexico 87119

Prepared by:

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Project No. ABQ11-14 P-01

I, John M. Provine, New Mexico Registered Professional Engineer No. 10999, do hereby certify that this document was prepared under my direct supervision and is true and correct to the best of my knowledge and belief.

John M. Provine Date

DRAINAGE MASTER PLAN
Project No. 4255.01

✈ Albuquerque International Airport ✈

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SECTION I

INTRODUCTION

A. **General.** For many years, capital improvement projects at the Albuquerque International Airport (AIA) have been planned and completed without the aid of a formalized master plan for storm drainage. Numerous runway, taxiway, apron and terminal improvement projects have been completed along with storm drainage system components, generally designed within each project's limits. This system of individual project development has led to some problems and various inefficiencies in expenditures for storm sewer improvements. Recent increase in capital improvement activities has led to an increased need for understanding the overall nature of drainage at the AIA. While paving and/or building projects can normally be restricted to specific project limits, drainage improvements usually extend well beyond the limits of an individual project area, to include upstream runoff contribution areas and downstream runoff impacts.

There are several reasons which contributed to this situation. Individual projects came to life in rather short time frames based on need and available funding. While one project was being completed, it was uncertain just what the next project would encompass or when it would occur. However, in recent years, better planning efforts have been formulated to develop not only current construction projects but to line up several future projects. The need to better master plan all drainage network system components reached a peak in 1991 during the design of a project to reconstruct Taxiway E which is the southern parallel taxiway to Runway 8-26. At this time it was determined that reconstruction of Taxiway E would be immediately followed by a similar reconstruction project for Taxiway A which is the northern parallel taxiway to Runway 8-26. Since Runway 8-26 is the primary runway at the AIA, it was felt that Taxiway E could be reconstructed, leaving Runway 8-26 and Taxiway A in service, resulting in little disruption to air traffic. After Taxiway E was completed, Taxiway A could then be reconstructed leaving Runway 8-26 and Taxiway E in service, again resulting in little disruption of air traffic. Recognizing this planned construction sequence, it was soon discovered that the drainage system for the Taxiway E project should not be confined to the specific project limits, but should be tailored to accommodate the planned improvements for the future Taxiway A project as well. In addition, much of the contributing drainage area for the Taxiway E drainage structures actually extends northward through the Taxiway A location and much of the Kirtland Air Force Base (KAFB) areas.

There are numerous other activities at the airport which have also stressed the need for better planning of storm drainage at the AIA.

The major terminal renovation project during 1988 significantly changed the topographical nature of the airport. A significant portion of the airside infrastructure (runways, taxiways and aprons) drain through the terminal area. In addition, the terminal area contributes runoff to adjacent commercial development areas and the City of Albuquerque's storm drain system.

The AIA Airport Master Plan (AIA AMP) is to be completed in 1994. This master plan proposed substantial changes in the entire airport area which will be implemented in the very near future. The most significant changes include reconstruction of Runway 3-21 to be the primary air carrier cross wind runway (and to allow for simultaneous operations) and closure of Runway 17-35 and using the areas of Runway 17-35 to be abandoned for further expansion of the terminal facilities, and relocation of air freight operations to the southern and eastern area of Runway 3-21.

The Sunport Corridor, consisting of a new principal arterial and interchange to link the terminal area to Interstate 25 has been designed and will soon be under construction. This new roadway will change drainage patterns in the western portions of the AIA.

Construction of Spirit Drive has stimulated numerous developments on the west side of the AIA on properties owned by numerous private individuals including large areas owned by the University of New Mexico. Each of these development areas have potential impacts on the AIA drainage system and could be affected by AIA drainage.

It is now very clear that construction activities on and near the AIA property will increase significantly. A master planned storm drainage system is needed for orderly development of these proposed projects. Therefore, this document has been prepared in order to guide Aviation Department and City of Albuquerque Hydrology staff, developers and designers in implementing individual projects without adverse drainage impacts. An overall master plan such as this is not intended to final design all the details of the storm drainage system, however it is intended that the basic network character and size be determined at this time so that pipes, channels, ponds and other outfalls can be constructed at this time which will accommodate other future improvements.

B. **<u>Agencies.</u>** In master planning the storm drainage system it is necessary to first have an understanding of the numerous agencies affected by the drainage system. These agencies not only are interested in the drainage system, but will likely be involved in funding, constructing, operating and maintaining these systems.

The City of Albuquerque, Aviation Department is obviously the principal agency involved in implementing and maintaining the airport drainage system. The Aviation Department is the lead agency in overseeing all activities at the airport. KAFB currently has joint use of the AIA and also owns much of the lands that surround the airport. Some of the drainage system components will be owned and maintained by KAFB personnel. Since drainage runoff does not recognize property boundaries, the entire watersheds in many areas are also affected. This means that the City of Albuquerque Public Works Department is also involved. Much of the Gibson Boulevard storm drainage system as well as other City owned systems, is affected by this master plan. The University of New Mexico owns most of the lands west of Spirit Drive. These areas are the downstream recipient of much of the AIA flows. The New Mexico State Engineer's Office has been involved in the review and approval of at least two detention ponds constructed on or near the AIA. The Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA) has review authority over many of the drainage plans prepared for airport area development. In addition, the South Diversion Channel and Tijeras Arroyo which represent primary outfalls for airport drainage area under AMAFCA jurisdiction. The New Mexico State Highway and Transportation

Department (NMSHTD) was involved in the design of the Sunport Corridor project. In addition, much of the airport drainage system is dependent upon the capacity of culverts under I-25. The Federal Aviation Administration funds most of the airside construction projects and establishes criteria for airport design and is an integral part of this and nearly all project developments.

C. **Abbreviations.** In the following pages, numerous abbreviations will be utilized. The following list is given for a quick reference to these abbreviations:

a-f	acre-feet
AIA	Albuquerque International Airport
AMAFCA	Albuquerque Metropolitan Arroyo Flood Control Authority
cfs	cubic feet per second
CLV	Culvert
DPM	City of Albuquerque Development Process Manual
FAA	Federal Aviation Administration
FAA AC	FAA Advisory Circular
KAFB	Kirtland Air Force Base
NMSHTD	New Mexico State Highway and Transportation Department
SEO	New Mexico State Engineer's Office
SCS	Soil Conservation Service
SDC	South Diversion Channel
SD	Storm Drain

SECTION II

CRITERIA

A. **General.** Selection of specific design criteria is difficult for a project of this complexity. As discussed in Section I of this report, there are numerous agencies directly affected by the AIA drainage system, each with their own specific drainage design criteria. However, the criteria used in this study has been specifically tailored for the conditions at the AIA, but based upon a combination of both FAA and City of Albuquerque Criteria. While this criteria development entails a complex combination of variables, **simply stated, the criteria used was to contain the 10-year event in underground networks (pipes) and contain the 100-year event with storage.** The 10-year event was used to size all pipes and then checked against the 100 year event. If the pipes could not keep the 100 year event from ponding on paved surfaces, pipe sizes were increased to avoid ponding on paved areas when considering future development.

As stated, both FAA and City of Albuquerque criteria were used and modified in developing the criteria for the AIA DMP. Neither of these two criteria were directly useable in and of themselves. Therefore, a combination of these criteria, along with an understanding of the nature of the AIA area and operations was necessary.

City of Albuquerque storm drainage criteria states that the 10 year event should be handles in storm sewers. In addition, this criteria further states that the 100 year event shall be conveyed in the storm sewer and the street section such that one traffic lane in each direction remain dry during the 100 year event. The AIA has adopted this same criteria for the 10 year event. However, for the 100 year event some modifications were necessary. Obviously, on airports there is no curb and gutter, thereby eliminating the surface conveyance system available on city streets. On airports, the pavement surfaces must remain free of runoff. However, it must be noted that the airport does contain some street sections and furthermore, some runoff from the airport does enter City of Albuquerque street sections such as on Gibson Boulevard. Therefore, this criteria cannot be totally cast aside.

Drainage on airport areas is dictated by FAA Advisory Circular 150\5320-5B, "Airport Drainage". This AC does not give mandatory requirements, but rather provides suggestions for a reasonable approach to airport drainage design. Most of the AC is dedicated to charts, nomographs and formulas to aid designers in drainage design. However, it is prudent at this point to review some of the recommendations given in the AC since FAA funding is crucial in future airport development. Following the FAA criteria is important to ensure that this funding is not jeopardized. Some important points provided in the AC are listed here.

1. Airports shall have smooth, well drained operations areas to permit safe movement of aircraft under all weather conditions. In drainage design, there area two basic purposes, to permit movement of air traffic and to maintain stable pavement. It is important to remember that not only does drainage affect traffic but it is crucial in protecting the life of pavement areas. This is particularly difficult on airports due to the fact that airport

airside development is, by design, very flat. These flat grades, by their nature are difficult to drain.

2. Proper drainage does not assure an all-weather airport but it does shorten the interval of non-use. This requires an economically designed system to realize the full value of the investment made. More frequent storms are by definition less severe and require smaller, less expensive drainage systems. Conversely, less frequent storms are much more severe (for example, the 100 year event) and require significantly larger, more expensive storm drainage facilities. The challenge in drainage design is to properly locate the optimal point in terms of storm frequency, severity and cost of the systems.

3. Inlets are the principal problem on airports because there is normally differential settlement around the inlet. This can be prevented by careful compaction in these areas and use of expansion joints if located in concrete pavement. Medium or high head ponding will be unacceptable to personnel servicing aircraft with baggage, fuel, food, etc. This could also obscure pavement markings and thus inhibit parking of aircraft at gate positions. Aprons serving instrument or all weather air carrier operations warrant a head limitation of 0.4 feet. also airports where continuing operations are regularly conducted in severe weather conditions would warrant use of a similar limitation.

4. There are a few important points to remember in using the FAA AC in drainage design. This AC is intended to be used for all airports, from general aviation to air carrier. The AIA operates continuously on a 24 hour a day schedule and in adverse weather conditions. Therefore, FAA criteria should be followed as closely as possible but should most importantly be tailored to the specific needs of the AIA.

B. **Hydrology.** The FAA AC suggests using the 2, 5, or 10 year event for design. Storm sewers should be designed to drain ponding areas based on total storm volume and keeping the maximum water surface elevation out of pavement areas in order to keep water from infiltrating pavement bases which would cause pavement failures. This design should then be modified in order to ensure that ponding areas drain in the shortest time possible. Furthermore, the FAA AC also recommends using rainfall events of 30 minutes, 1 hour or 2 hour intensities. These are much shorter storm durations than are normally used in New Mexico.

City of Albuquerque criteria requires use of the 10 year and 100 year events as stated in previous sections of this report. In addition, these events are based on rainfall 6 hours in duration. This is not a great departure from the FAA design suggestions. While the events used by the City are less frequent and more severe than those suggested by FAA, they are also of longer duration which balances the differences. In other words, the 5 year 30 minute storm is roughly equivalent to a 10 year 6 hour storm when comparing the resulting peak flows from a watershed. Again, when considering the critical nature of operations at the AIA, the City of Albuquerque criteria can be justified to ensure maximum use of the airport, minimization of non-use times, and reasonable cost of drainage facilities.

The criteria used for this study is therefore the 10 year 6 hour event for storm sewer design, the 100 year 6 hour event for ponding and storm sewers (keeping the pavement dry) and the 100 year 24 hour event for sizing detention ponds (which is necessary for NM SOE

approvals). This criteria is not held for the Sunport Blvd. Basin which required using a 100 year, 24-hour event.

C. **Soils.** A complete summary of soils in the study area will not be presented in this report. The SCS Soil Survey for Bernalillo County was utilized in preparing this study. This survey shows that the AIA area has mostly well drained soils with some areas of impervious clays, particularly on the east end of Runway 8-26. If further information is needed regarding the soils in the area, the reader should refer to this SCS Soil Survey.

D. **Drainage Areas.** Drainage area determinations were made utilizing several topographical maps of the airport area obtained through aerial photographs prepared for this study. Areas were delineated using these topo maps and were verified through extensive field surveys and visual observation. Area maps are included in this report in the plates provided.

E. **Modelling.** This entire drainage master plan is based upon hydrologic and hydraulic modelling of the study area utilizing AMAFCA's AHYMO491 computer program. Complete printouts of the input and output data files are available under separate cover.

Computer printouts for a project of this magnitude are quite voluminous and comprehensive. In order to more accurately and rapidly illustrate the computer model methodology, flow charts and summary tables were developed to graphically show drainage area characteristics (area, peak flows, times of concentration, etc.), how this area was routed downstream, where it was added to other areas, and the resulting outfall data. In most cases, an entire basin network is shown on a single sheet and the reader can trace the theory and pertinent data without extensive revue of the computer printouts. These flowcharts and summary tables are contained in the appendices.

SECTION III

EXISTING CONDITIONS

A. General. This study began with an analysis of existing conditions at the AIA. Existing conditions for the purpose of this study includes the completion of the reconstruction of Taxiways A and E which were completed in 1994. Existing conditions does not include reconstruction of Runway 3-21 to air carrier standards and abandonment of Runway 17-35. Existing conditions includes Runway 3-21 as general aviation and the freight apron located west of Runway 3-21. It further includes completion of Spirit Drive. A basic description of the major basins is given in the following sections.

B. <u>Basin G17.</u> This 0.10140 square mile basin encompasses the northern end of Runway 17-35 and developed areas west of the runway end. Drainage from the US Customs area ramps is collected in a storm sewer, routed around the north end of the runway and through developed commercial areas west of the runway end and just south of Gibson Blvd. The system experiences problems just south of Gibson Blvd. near the old potato chip factory where the storm sewer makes a 90 degree bend to connect to the 36 inch storm sewer in Gibson. Because of this bend, the hydraulic grade line surfaces and flows leave the storm sewer and surface in the street.

TABLE III-1
Basin G17

[illegible]	[illegible]	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
101	0.0284	32.42	54.60	1.3824	2.394	18" SD	US Customs Apron
102	0.0566	33.74	63.21	2.267	4.221	18" SD	E. of R/W 17-35 N. End
103	0.0622	34.00	65.13	2.504	4.652	24" SD	N. of R/W 17-35
104	0.0710	38.05	72.59	2.921	5.384	24" SD	W. of R/W 17-35 N. End
105	0.0820	36.55	75.76	3.461	6.319	27" SD	Offices NW of R/W 17-35
106	0.0727	32.77	68.80	2.982	5.502	Surface	Columbia Drive
107	0.0894	47.40	81.72	3.835	6.971	30" SD	Offices (SAIC) NW of R/W 17-35
108	0.0087	15.82	24.74	0.5433	0.890	21" SD	Commercial area on Gibson
Σ	0.1014	66.97	102.89	4.599	8.215	30"SD	to Gibson Blvd. Storm Drain

C. **Basin N17.** This 0.4119 square mile basin extends along Runway 17-35 from the Runway 8-26 intersection up to the north runway end. This area also includes the main parking structure, car rental parking area, the loop road, the main terminal ramp and commercial development areas just north of the parking structures to Alamo Drive. Drainage is conveyed to the Yale Blvd. storm sewer system and eventually discharged into the Kirtland Channel. Problems with storm sewer flows surfacing have been experienced just north of the parking structure near the commercial development. Furthermore, the terminal expansion project included installation of a restrictor plate in the storm sewer conveying flows from the ramp through the terminal area. This restrictor plate causes ponding on the aircraft parking ramp south of the terminal. The restrictor plate was installed to correct the first problem described above, that is the flooding near the commercial development caused by stormwater exiting the storm drain.

TABLE III-2
Basin N17

[illegible]	AREA (sq. mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
201	0.0129	5.38	9.67	0.538	1.000	24" SD	E. Side T/W D
202							eliminated
203	0.0210	28.89	48.83	0.951	1.713	24" SD	Between R/W 17-35 & T/W D
204	0.0217	31.28	52.67	0.973	1.759	24" SD	Between R/W 17-35 & T/W D
205	0.0267	20.35	34.26	1.169	2.131	24" SD	Between R/W 17-35 & T/W D
206	0.0405	20.50	25.67	1.861	3.337	36" SD	Between R/W 17-35 & T/W C
207	0.1267	66.58	91.50	5.913	10.570	36" SD	Between R/W 17-35 & T/W C
208	0.0435	22.95	30.82	2.029	3.620	24" SD	Between R/W 17-35 & T/W C
209	0.1408	75.33	107.05	6.545	11.689	42" SD	Between T/W C & Access Road
210	0.1449	77.03	109.64	6.730	12.019	42" SD	Between Access Road & Girard
211	0.0268	38.18	60.46	1.637	2.687	24" SD	Girard Airport Entrance
212	0.0058	11.58	17.64	0.414	0.654	24" SD	Rental Car Parking
213	0.0480	59.98	75.86	3.327	5.299	24" SD	Rental Car Parking
214	0.1084	141.64	204.43	7.213	11.597	24" SD	S. Side Airport Loop Road
214.1	0.2533	193.20	281.40	13.943	23.616	10" SD	S. Side Airport Loop Road
214.2	0.2839	220.09	332.63	15.079	25.819	36" SD	S. Side Airport Loop Road
215	0.0124	10.52	22.05	0.270	0.642	surface	W. side of Girard
216	0.0134	7.00	18.49	0.193	0.535	Surface	W. Side of Girard; N. of 215
217	0.0535	15.46	37.22	0.766	2.310	Surface	Business complex
217.1	0.3374	232.83	358.61	15.845	21.130	Surface	S. Side of Alamo Drive

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
218	0.0932	116.18	165.12	6.224	9.995	24" SD	Lower parking lot
219	0.664	105.46	105.46	4.587	7.309	Surface	Parking Structure
220	0.0327	97.81	97.81	2.260	3.603	36" SD	E. Terminal Ramp
221	0.0370	46.43	46.43	2.542	4.058	36" SD	Terminal Entrance, East
222	0.0215	45.55	45.55	1.351	2.196	24" SD	Terminal Entrance, West
223	0.3651	418.23	418.23	17.521	30.882	60" SD	Areas E. of Alamo/Yale
224	0.2612	295.18	295.18	14.315	24.274	24" SD	Amfac Hotel, East
225	0.0227	49.80	49.80	0.765	1.545	Surface	Amfac Hotel, North
226	0.0428	87.14	87.14	2.397	4.003	Surface	Amfac Hotel, West
227	0.0353	70.75	70.75	2.011	3.344	24" SD	Airport Entrance by Yale
228	0.4119	514.03	514.03	20.169	35.293	24" SD	E. side of Yale
Σ	0.4119	328.37	514.53	20.169	35.293	2-42" CMP	to Basin KC

D. **Basin KC.** This 0.6810 square mile basin extends from Yale Blvd to the South Diversion Channel and includes commercial development on each side of Randolph. The principal drainage feature of this basin is the Kirtland Channel which begins just west of the Yale/Alamo intersection and discharges to the South Diversion Channel via a baffle chute drop structure west of Mulberry. The principal problems experienced in this basin are in the lower reaches of the Kirtland Channel where the concrete lined channel makes two rather severe horizontal bends. Very high velocities which exist in the channel have resulted in the flows overtopping the channel banks in this area.

The Kirtland Channel has experienced some problems near the box culvert crossing on Mulberry. These problems have been noted through visual observations and some damages that have occurred during past rainfall events. The Kirtland Channel in this area makes two horizontal bends on what appears to be rather severe horizontal curves. Both of these bends occur just upstream from the Mulberry to form an "S" curve. The downstream curve immediately upstream of the Mulberry crossing has experienced overtopping along the southern channel bank encroaching on some existing homes in this area. In addition to this curvilinear alignment, there is a side channel discharging into the Kirtland Channel just upstream of the Mulberry crossing. The Mulberry crossing itself is a two cell 6'X6' box culvert. Downstream of this culvert is a short segment of channel and then a baffle drop spillway discharging into the South Diversion Channel.

From visual observation it appears that the problem of flow overtopping the channel is caused by superelevation of flows with high velocities around this second curve. Additional freeboard was added to this area by constructing a one foot high concrete retaining wall along this channel

edge. However, the side channel inflow from the residential areas to the north cannot be modified in this same way as these flows cannot be blocked. It also appears that the box culvert may be causing some of the problem. If the box culvert is of insufficient capacity, then backwater might be causing some of the overtopping problem. These problems were analyzed and are discussed in the next section.

TABLE III-3
Basin KC

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
301	0.0197	37.04	57.39	1.304	2.099	2-54"	NE & SE Cor. Alamo/Yale
302	0.0251	47.59	73.55	1.678	2.694	2-54"	N. Side Randolph, W. of 301
303	0.4586	378.86	593.93	22.993	39.943	Surface	W. Side Yale, S. Side Randolph
304	0.0132	18.92	2.77	0.6425	1.113	Surface	W. Side Yale, S. of Randolph
305	0.0322	33.82	53.58	2.0962	3.373	Surface	S. of Randolph, W. of Yale
306	0.0163	31.40	48.31	1.1165	1.781	Surface	W. Side Yale @ Yale Airport Entrance
307	0.0235	40.32	64.18	1.410	2.316	Surface	N. Side Kirt. Chan., W. of 301
308	0.5012	440.23	691.32	25.548	44.141	Surface	BDM Complex
309	0.0527	44.08	70.93	3.078	5.075	Surface	S. Side Randolph, W. of 303
310	0.5119	459.35	720.25	26.201	45.213	Surface	S. Side Kirt. Chan., W. of 308
311	0.6619	553.86	921.55	31.081	54.644	Surface	N. Side Kirt. Chan.
312	0.6572	548.20	912.19	30.878	54.273	24" SD	S. Side Kirt. Chan. E. Side Univ.
313	0.0099	3.60	11.75	0.094	0.328	Surface	S. Side Randolph
313.1	0.0908	57.03	115.02	3.603	6.525	36" SD	SE Cor. Randolph/Univ.
314	0.0177	4.25	17.94	0.117	0.501	Surface	W. of 306
314.1	0.0282	11.29	33.97	0.431	1.122	Surface	E. Side Univ., S. of Randolph
315	0.0105	9.23	18.51	0.314	0.620	Surface	N. of Postal Facility
316	0.0090	10.78	19.47	0.345	0.645	Surface	N. Side Kirt. Chan., W. of Univ.
317	0.6995	578.50	977.11	31.956	56.593	Surface	N. Side Kirt. Chan., Univ. to SDC
Σ	0.6810	568.6	961.5	30.96	54.893	Kirt. Chan.	to South Diversion Channel

E. **Basin SP.** The Sunport Corridor basin consisting of 0.2010 square miles is currently undergoing alterations. The existing conditions of this basin has been modelled but will change with the construction of the Sunport Corridor which will be a new principal arterial connecting I-25 to the AIA. The proposed construction of the roadway was accompanied by a drainage report specifically for this roadway project and is further addressed in the next section of this report.

This basin originates at the west edge of the terminal area including the new postal facility. The flows are conveyed through earth channels south of George Road where they eventually cross University in culverts. Areas west of University drain primarily by surface conveyance and pass beneath I-25 via culverts and eventually discharge to the South Diversion Channel.

TABLE III-4
Basin SP

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
401	0.0280	51.96	80.77	1.839	2.959	24" SD	Post Office Site
402	0.1150	99.16	203.50	203.50	6.930	CLV	Adjacent to George Road
403	0.1560	60.30	128.40	128.40	7.761	SD	W of Univ, S of Randolf
404	0.1810	62.09	141.56	141.56	8.639	48" SD	University to I-25
405	0.1840	63.18	144.59	144.59	8.848	RR Bridge	University to RR tracks
406	0.0170	6.94	21.07	21.07	0.598	CLV	SE of I-25/SDC
407	0.1300	106.19	223.38	223.38	7.518	3-36" CLV	RR tracks to I-25
408						CLV	I-25
409						48" SD	SW of I-25/SDC
410						3-36" CLV	I-25
411						48" SD	I-25 to SDC
Σ	0.2010	60.19	150.6	3.846	9.445	Open Chan.	to South Diversion Channel

F. **WE Basin.** This 0.38 square mile basin includes the southern portion of the terminal area, the west ends of Runway 8-26 and the Taxiway's A and E, and the undeveloped escarpment area between the airport and University Boulevard. The existing drainage situation consists of a collection system that is throttled into a 24" outfall to the escarpment area, the overland flow within the escarpment area, and a 2-60" culvert structure at University Boulevard. In addition to being throttled at the outfall, the collection system contains many sections that are inadequate to handle the 5 year design storm. The existing peak discharge reaching the culvert crossing is 101 cubic feet per second (cfs). The capacity of the 2-60" structure is 370 cfs

The proposed system for this basin includes many upgrades to the existing collection system necessary to handle the 10 year design storm, and to solve existing drainage problems. To

accommodate future, additional flows under the master plan, a portion of the system has been upsized to accept runoff from areas scheduled for future development. The 24" outfall has been replaced with a 60" outfall that is sized to meet master plan requirements. Until the downstream drainage system and the matter of the unplatted arroyo is settled, an orifice plate should be installed to restrict the flow to existing levels. The storage capacity in the islands will be utilized to hold the peak discharge rate that exceeds the capacity of the orifice plate with this stored volume being released after the discharge rate falls below the orifice capacity. This 60" pipe discharges to a rip-rap lined channel that eventually outfalls at the University Boulevard culvert crossing. Several small arroyos within the escarpment area are intercepted by this channel. As a result of this project, the design flow in the channel is 166 cfs. The master plan discharge to be carried by the channel to the crossing is 347 cfs. Under the master plan, the additional runoff volume generated by the 100 year, 6 hour design storm will be contained in the islands.

Currently, subbasin 504, located between concourse C and B, has ponding problems during typical storms. A shallow swale carries the stormwater southwest from this portion of the terminal apron to an inlet and 24" drain in Subbasin 505. The flat grade and resulting shallow depth of the swale do not allow adequate drainage. Concern expressed by the Aviation Department over the life of the pavement with substantial ponding occurring concrete bring this situation up for resoluction.

A tee installed during the Taxiway E project near the west end of Runway 8 can be extended to this area. See Section IV for specific details on this improvement.

TABLE III-5
WE Basin

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
501	0.0430	77.56	121.58	2.679	4.384	24" SD	N side of 17-35/8-26
502	0.0810	61.20	94.09	4.833	8.001	54" SD	N side 8-26 W of 17-35
503	0.1020	78.32	115.51	6.312	10.348	30" SD	Terminal N ramp
504	0.0220	43.53	66.47	1.550	2.459	Surface	Terminal W ramp
505	0.1480	100.38	139.44	9.460	15.378	60" SD	8-26 W overrun
506	0.2692	159.48	246.62	14.497	24.721	60" SD	8-26 W overrun
507	0.0250	28.65	49.86	1.005	1.892	24" SD 18" SD	SW cor 17-35/8-26
508	0.0770	43.41	69.07	3.335	6.112	48" SD	S side 8-26
509	0.2372	139.91	202.20	13.537	22.713	48" SD	W end 12-30
510	0.2932	168.12	272.60	14.955	25.867	54" SD	S of 8-26 W overrun
511	0.3032	171.55	284.60	15.118	26.305	54" SD 24" SD	W 8-26 overrun, E of Univ

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
512						2-60" CLV	University crossing
513						surface	E of Univ, S of 8-26 overrun
514						arroyo	W of Univ
515						3-36" CLV	E of I-25
516							I-25
517						3-36" CLV	W of I-25
518						SDC	
Σ	0.3602	149.3	256.3	15.88	28.510	Open Chan.	to South Diversion Channel

G. **35SE Basin.** This 0.24 square mile basin includes a small area just east of Runway 17-35. The existing drainage system drains the area only after considerable ponding occurs along Taxiway E and an extension will be constructed to provide additional inlet capacity. This extension (28 cfs capacity) was connected to the existing system. The additional runoff volume generated by the 100 year, 6 hour design storm will be contained in the existing island. The necessary improvements in this basin were made with the Taxiway E construction project.

TABLE III-6
Basin 35SE

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
601						36" SD	SE cor 17-35/8-26
602						24" SD	SW cor 17-35/8-26
603						18" SD	W of 602
604						42" SD	17-35/3-21/TW E
605						24" SD	W of 17-35
606						24" SD	E of 3-21
607						24" SD	SE end 12-30
608						42" SD	12-30
609						24" SD	S of 12-30, E of 17-35
610						48" SD	E of 17-35
611						48" SD	Escarpment

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
Σ	0.2525	68.40	82.87	8.807	17.101	48"SD	to Tijeras Arroyo

H. Basin GA.

This 0.7446 square mile basin encompasses the general aviation area located south and west of the intersection of Runways 17-35 and 3-21. These drainage basins also include areas around Clark Carr, Spirit Drive, the UNM Golf Course and pass through I-25 near the old railroad track crossing.

Flows are conveyed mostly by storm sewers which discharge into channels and then through culverts beneath I-25. There are some problems with the crossing structures on University Avenue due to low inlet capacities of these structures.

TABLE III-7
Basin GA

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
701	0.0042	6.78	11.00	0.219	0.379	24" SD	R/W 12-35 NW of 17-35
702	0.0087	10.91	15.58	0.470	0.803	24" SD	R/W 3-21 SW of 17-35
703	0.0186	12.83	20.48	0.955	1.658	24" SD	R/W 12-35 NW of 17-35
704	0.0107	15.33	25.02	0.549	0.954	18" SD	R/W 12-35 NW of 17-35
705	0.0162	20.48	10.19	0.565	1.128	24" SD	Between R/W 12-30 & GA Area
706	0.0386	12.40	19.61	1.401	2.754	36" SD	Between R/W 12-30 & GA Area
707	0.0226	19.07	28.03	1.077	1.916	24" SD	Between T/W C & GA Area
708	0.0141	14.85	21.53	0.736	1.270	24" SD	Between R/W 3-21 & T/W C
709	0.0653	63.21	67.42	3.812	6.365	24" SD	Between R/W 3-21 & T/W C
710	0.0638	63.43	96.42	3.815	6.322	42" SD	GA Area North
711	0.0513	55.35	54.20	3.125	5.154	36" SD	GA Area SE
712	0.1259	107.56	136.09	7.330	12.237	24" SD	GA Area West
713	0.0699	66.46	72.42	4.008	6.726	42" SD	GA Area South
714	0.0232	36.99	60.20	1.191	2.068	Surface	GA Area SW
715	0.1119	75.01	114.48	5.615	9.814	42" SD	GA Area NW
716	0.1353	112.63	144.56	7.812	13.074	48" SD	GA Area West

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
717	0.0159	19.10	34.27	0.631	11.58	24" SD	GA Area NW
718	0.1271	90.23	139.16	6.361	11.29	42" SD	GA Area NW
719	0.0209	18.26	37.68	0.468	1.10	36" SD	W. Side Spirit Drive, S. Side Clark C
720	0.4077	193.01	294.80	16.885	3.809	60" SD	E. Side Univ., N. of Clark Carr
721	0.4542	198.05	320.18	17.477	32.544	60" SD	E. Side Univ., N. Side Clark Carr
722	0.0419	14.00	28.51	0.925	2.149	24" SD	Between Freight Apron & Spirit Drive
723	0.0582	20.38	43.36	1.281	29.62	24" SD	Spirit Drive
723.1	0.4784	182.19	32.06	15.454	27.116	Surface	W. Side Spirit Drive
724	0.0465	20.75	59.72	0.592	1.736	48" SD	Between Spirit Drive & Univ.
725	0.5022	189.34	285.40	19.047	35.582	24" SD	W. Side Univ., NE Side Golf Course
726	0.4784	177.42	268.81	17.832	33.509	8'X6' CLV	UNM Golf Course
726.1	0.6575	196.46	384.64	21.402	42.149	60"	W. Side Univ., E. Side Golf Course
727	0.7446	207.03	449.84	23.007	46.031	3-36" CLV	Between I-25 & RR Tracks
728	0.6919	198.79	395.97	22.036	43.682	arroyo	Between I-25 & SDC
730	0.1043	83.47	103.48	5.825	9.846	42"50	E Access Road C
Σ	0.7446	207.0	449.84	23.007	46.031	Open Chan.	to South Diversion Channel

I. **Basin SW35.** This is a 0.123 square mile basin surrounding the southern end of runway 17-35 from the intersection with Runway 3-21 to the south runway end. No capacity problems exist in this area, however, the system outfall on the escarpment has experienced erosion problems. This area is scheduled for major renovations as the AMP has planned this area to be the new freight operations area. See Section IV for proposed Improvements.

TABLE III-8
Basin SW35

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
801	0.0431	26.90	47.05	1.551	2.980	24" SD	W. Side R/W 17-35, S. of R/W 3-21
802	0.0491	17.10	19.97	1.867	3.524	24" SD	W. Side R/W 17-35, S. end

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
803	0.0603	17.41	26.16	2.203	4.239	24" SD	S. Runway overrun
804	0.0129	4.24	10.38	0.228	0.559	18" SD	W. of R/W 17-35
805	0.1130	29.86	72.89	2.986	6.365	Surface	W. of R/W 17-35
806	0.0099	11.46	20.89	0.293	0.627	Surface	S. Runway overrun
Σ	0.1130	29.86	72.89	2.986	6.635	48" SD	to Tijeras Arroyo

J. **Basin M3.** This 0.172 square mile basin encompasses the middle portions of Runway 3-21 just south of the general aviation area. The outfall is a storm sewer discharging off the escarpment to the Tijeras arroyo. This system was constructed in 1990 with the Runway 3-21 renovation project and experiences no capacity problems but does exhibit some erosion at the discharge point.

TABLE III-9
Basin M3

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
901	0.1375	51.19	85.95	5.487	10.323	24" SD	E. Side R/W 3-21
902	0.0702	50.12	82.33	3.601	6.252	54" SD	R/W 3-21 by Freight Apron
903	0.0319	37.56	59.95	1.667	2.878	48" SD	T/W F
904	0.0417	43.13	70.16	2.353	3.968	24" SD	Between T/W F & Freight Apron
905	0.0098	7.91	12.75	0.686	1.090	24" SD	N. End Freight Apron
906	0.0192	35.05	53.63	1.156	1.913	48" SD	N. of Freight Apron
907	0.0344	3.92	12.45	0.393	1.226	72" SD	S. of R/W 3-21
Σ	0.1719	54.02	94.11	5.880	11.549	72" SD	to Tijeras Arroyo

K. **Basin US.** This basin is made up of areas between University Boulevard and the South Diversion Channel in the UNM Golf Course area. There are no storm sewers in this system. Most of the area is undeveloped and also includes the UNM Golf Course. All runoff is surface transported to culverts which convey the flows beneath roadways through culverts. The upper reaches of the watershed are limited by University Boulevard. University diverts all upstream flows east of University to the north or south so that there is no further upstream offsite flow contributions. This is important to the integrity of the golf course as such drainage courses may have adverse impact on the golf course facilities. The golf course conveys flows by surface transport to several culverts beneath I-25 where flows are discharged to loosely defined earth

channels on undeveloped property. These flows then eventually reach the South Diversion Channel. There are no identified problems within this basin relating to airport development.

L. **Basin W3.** This 0.13 square mile basin includes the southern most end of Runway 3-21 including most of the existing freight apron. The upper reaches of this basin includes the concrete freight apron. This apron contains several drop inlets and a 36" storm sewer. A second storm sewer exists between Runway 3-21 and the parallel taxiway and these two systems connect near the south end of the runway where a 48" storm sewer conveys flows beneath the runway to the southeast, eventually discharging the flows to the Tijeras Arroyo. No problems exist with capacity but erosion problems do occur at the discharge point on the escarpment where flows are directed to the Tijeras Arroyo.

TABLE III-10
Basin W3

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
1101	0.0140	26.71	41.19	0.936	1.506	36" SD	Freight Apron
1102	0.0037	7.39	9.51	0.260	0.417	24" SD	Freight Apron to TW F
1103	0.0591	37.10	67.92	2.437	4.508	48" SD	Runway 3-21
1104	0.0500	28.33	56.03	1.422	3.056	48" SD	SE of Runway
1105	0.0516	29.38	54.83	2.006	3.786	48" SD	R/W 3-21 to T/W F
1106	0.0376	17.81	34.40	1.443	2.723	36" SD	S of Freight apron
1107	0.0590	28.90	54.61	0.987	3.015	surface	S of Freight apron
1108	0.0805	45.15	76.62	2.997	5.759	24" SD	R/W 3 overrun
Σ	0.1305	73.04	131.31	4.419	8.815	66" SD	to Tijeras Arroyo

M. **Basin UE.** This 0.36680 square mile basin includes areas between the west airport boundary and the South Diversion Channel in the Ethicon Plant area. No airport area flows reach these watersheds and almost all drainage is conveyed through culverts. The higher reaches of the watershed include those areas adjacent to the University/Spirit Drive intersection. Flows are directed to this intersection where they are collected by several drop inlets and a short segment of storm sewer. These flows are then discharged through a 48" culvert beneath University into a natural channel. Several more drainage areas between the airport boundary and University are currently undeveloped and flows are conveyed beneath University through a series of culvert pipes. The middle portion of the basin includes the Ethicon plant area. Flows are then conveyed under I-25 through culvert pipes discharging to natural channels. Finally all flows discharge to the South Diversion Channel just north of Rio Bravo.

The character of this area will change significantly in the very near future. A grading and drainage plan for this area, known as the UNM Business Park has been submitted for approval to develop the areas adjacent to the Spirit Drive/University Intersection.

TABLE III-11
Basin UE

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
1201	0.0459	20.48	58.95	0.584	1.713	DI	Spirit Drive to University
1202	0.0692	32.90	91.38	0.946	2.673	DI	AIA Boundary to Spirit Drive
1203	0.0970	35.77	111.64	1.328	3.728	48" CLV	AIA Boundary to University
1204	0.0544	23.80	68.13	0.748	2.064	42" CLV	AIA Boundary to University
1205	0.2015	72.10	205.16	2.999	8.071	54" CLV	University to I-25
1206	0.0625	32.10	78.46	1.152	2.785	SDC	I-25 to Broadway
1207	0.3043	57.06	132.46	4.621	12.263	42" CLV 36" CLV 48" CLV	University to I-25
Σ	0.3668	71.83	189.7	5.773	15.048	Open Chan.	to South Diversion Channel

N. **Basin RB.** This basin includes areas immediately adjacent to Rio Bravo Boulevard from the south side of Ethicon, along Broadway to the South Diversion Channel. This area was included in the computer models for this AIA DMP, however, this is not an area of great concern as no airport property flows reach this basin. There are no known problems in this basin.

O. **Middle Basin.** This 1.454 square mile basin is the largest basin in this design project and includes approximately 60 percent of the Runway 8-26 area to the east of Runway 17-35 and the area north to Gibson Boulevard. Developed areas within KAFB make up roughly 80 percent of the basin. The existing drainage situation consists of several collection systems within KAFB and along Runway 8-26 that drain to a 60" (84 cfs capacity) and parallel 2-24" outfalls (22 cfs capacity). Under the 10 year 6 hour design storm, these outfalls have insufficient capacity to handle the runoff from the various collection system. The outfalls discharge to the area south of the control tower through a system of channels and a 66" culvert at Kirtland Road. Below Kirtland Road, the existing arroyo carries the runoff to the Tijeras Arroyo. Including the local runoff from the areas south of the airport, the existing peak discharge reaching Kirtland Road is 117 cfs and 60.4 AF of runoff volume.

The master planned system includes new collection branches to provide drainage capacity along the reconstructed taxiway E, a parallel 72" outfall will collect the flows from the various collection systems and carry the combined flow to the detention facility. The proposed detention facility consists of an excavated reservoir on KAFB just north of Kirtland Road with the road serving as the embankment for the structure. Under the master plan flows, the facility is

designed to contain the 10 year design storm with a principal spillway discharge of 86 cfs and a 100 year design storm runoff with a principal spillway discharge of 118 cfs with no flow through the emergency spillway. The emergency spillway consist of a soil cement lined low-flow channel sized to carry the 100 year 6 hour inflow of 540 cfs. One hundred percent of the runoff produced by the 100 year 6 hour rainfall will drain from the reservoir in 48 hours. Since considerable capacity is being provided for KAFB runoff that cannot be collected by the existing systems, the master planned system will carry 329 cfs and is expected to release well below the 10 year master plan level. The proposed system to be constructed in conjunction with Taxiway E will carry 186 cfs to the detention facility that will have a discharge rate of 81 cfs and 64.4 acre-feet of maximum storage. This master planned system is increasing the collection capacity within the airport and extending branches to a couple of areas that have had historical drainage problems. Also, some areas that have freely drained to the small arroyos south of the airport will be within the expanded Taxiway E and this runoff is being diverted to the detention facility. Under the 100 year, 6 hour design storm, the AIA property will generate 4.3 ac-ft of additional volume that will be contained in the islands.

All required improvements in this basin were made with the Taxiway E and A construction projects.

TABLE III-12
Middle Basin

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
1400	0.0327	62.66	96.52	2.202	3.534	48" SD	E of 17-35
1401	0.0639	102.69	111.47	3.762	6.265	48" SD	17-35/TW A
1402	0.1179	115.52	175.96	6.094	10.546	66" SD	17-35/TW E
1403	0.1279	122.40	141.41	6.598	11.424	30" SD	TW E/3-21
1404	1.0419	315.52	391.05	48.851	86.240	66" SD	Detention Pond
1405							eliminated
1406							eliminated
1407							eliminated
1408							eliminated
1409							eliminated
1410	0.1620	55.16	96.50	7.798	13.579	42sd	FAA ATC/Base Ops
1411	0.2472	79.35	89.03	10.465	21.019	36sd	West Base Ops Apron
1412	0.1653	55.16	57.16	8.034	13.952	42sd	West Base Ops Apron
1413	0.2522	79.50	91.15	10.735	21.480	54sd	Island
1414	0.0191	33.87	53.35	1.150	1.903	36sd	Perimeter Rd.

[illegible]	[illegible] AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
1415	0.7490	200.08	246.08	36.050	63.024	48sd	Island
1420	0.4700	134.50	255.49	21.315	37.915	54sd	Ross Apron/Hangar
1421	0.0899	33.23	33.41	2.447	7.312	42sd	Apron C
1422	0.1057	63.24	64.94	3.574	9.094	54sd	Center Base Ops Apron
1423	0.4457	111.97	129.86	22.359	36.595	66sd	Island
1424A	0.4531	112.18	119.78	22.778	37.301	72&66sd	Island
1424B	0.7108	191.83	194.91	33.834	59.318	66sd	Island
1425	0.2959	105.65	111.53	7.928	24.994	30sd	Island
1426	0.3062	318.80	335.60	16.875	29.019	30sd	Island
1430	0.9060	227.56	264.22	41.972	74.263	24sd	Island
1431	0.2750	280.40	309.96	14.695	24.781	42sd	Island
1432	0.0142	22.18	36.57	0.730	1.294	30sd	Arm/Dearm Apron
1433	0.0174	27.18	44.81	0.895	1.586	30sd	Island
1440	0.1570	167.20	283.75	8.182	14.069	60sd	KAFB Office Area
1441	0.1725	168.54	193.81	9.288	15.817	72sd	East Base Ops Apron
1442A	0.0266	42.94	43.82	1.434	2.452	72sd	Perimeter Rd.
1442B	0.2116	206.35	230.89	11.309	19.315	42sd	Perimeter Rd.
1443	0.0168	27.99	44.93	0.922	1.568	42sd	Expire Ramp
1444	0.1780	171.36	175.18	9.473	16.192	42sd	Island
1445	0.0172	27.12	44.29	0.868	1.515	36sd	Island
1446	0.2751	281.42	310.96	14.725	25.162	48sd	Island
1447	0.2256	220.53	232.78	12.156	20.718	78sd	Island
1448	0.3059	318.45	335.43	16.552	28.199	72sd	Island
1450	1.1629	358.29	493.50	51.927	93.044	2-72sd	Fire Train/Old FAA Tower
1451	1.5460	98.32	130.30	68.031	119.703	surface	S. of Hot Pads
Σ	1.4688	664.32	811.89	68.480	121.243	Deten. Pond	to Tijeras Arroyo

P. **East End and Air Guard Basins.** These basins cover 0.78 and 0.21 square miles respectively and are discussed together because of the drainage situation within KAFB. Over 80 percent of these two basins are within the base with two existing systems available that outfall south of the airport. All of the arroyos draining these two basins are entirely within KAFB property from the

airport to their confluence with the Tijeras Arroyo. One small collection system is in the west end of the East End basin and requires much overland/channel flow before the outfall is reached. The existing 42" pipe and 24" pipe under the airport (with a combined 82 cfs capacity) is inadequate to handle the flows reaching it from the collection system. Even under small storms, considerable ponding occurs within KAFB. This system outfalls into an old landfill site where extensive erosion has occurred. KAFB has expressed concerns about increased erosion and expects to address the problems at the landfill with a future construction project. The existing flows from the basin under the 10 year design storm is 95 cfs. The other system at the Air Guard site is in the eastern end of the two basins and has 30" and 48" pipes that terminate at the north side of Taxiway A for future connections. This system (with 164 cfs capacity) has an existing 10 year discharge of 33 cfs. Currently, no erosion problems have been identified in the arroyo downstream of the outlet structure.

The proposed system in the East End basin includes a collection branch along the extension of Taxiway E, replacing the existing 42" outfall with a 48" outfall at a steeper slope, and a new 66" outfall that will handle future collection systems within KAFB. The culvert under Kirtland Road at the outfall of the existing 42" pipe will also be replaced. The proposed 66" outfall will be capped at each end and will not carry any flow at this time. KAFB is proposing to construct improvements south of the airport to accept the flows from this master planned facility. The proposed system under this design project will carry 102 cfs under the 10 year design storm with the additional 100 year volume contained in the island ponding. Where the proposed collection branch serving Taxiway E will be intercepted by the future 66" line, an orifice plate has been installed to temporarily restrict flows to the existing level (95 cfs) at the existing outfall.

The new system in the Air Guard basin added a small collection branch added to serve the extension of Taxiway E. This branch adds 14 cfs (for a total of 47 cfs) to the system that has a capacity of 164 cfs. Based on the current conditions, we do not anticipate that this additional flow will adversely impact the downstream situation. Further, this branch intercepts a portion of the runoff that currently drains to the system that outfalls into the landfill.

TABLE III-13
East End and Air Guard Basin

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
1500	0.1200	78.24	145.00	4.618	8.575		E. Kirtland Road
1501	0.3500	58.60	125.44	7.818	16.107	surface	Along Kirtland Drive
1502	0.1100	80.09	159.60	12.756	24.204	pond	VA Hospital
1503	0.1350	95.38	159.28	5.892	10.093	42" SD	
1504	0.1215	81.33	148.42	5.650	10.396	48" SD	Vacant

SUB-BASIN NO.	AREA (sq. mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL L	DESCRIPTION
1505	0.1365	35.53	47.56	6.564	11.215	42" SD	S. Expire Ramp
1506	0.1445	41.64	57.80	6.906	11.790		Between T/W A & Ramp
1507	0.1705	46.50	59.12	7.323	12.537	42" SD	Between R/W 8-26 & T/W A
1508	.03694	53.51	69.09	7.693	13.201	48" SD	Between R/W 8-26 & T/W E
1509	0.3780	81.68	97.93	8.785	15.223	24" SD	Between R/W 8-26 & T/W E
1510	0.3694	84.21	167.41	13.611	25.749	30" SD	Ramp
1511	0.3780	86.69	114.31	14.071	26.538	54" SD	N. side T/W A
1512	0.3914	89.43	118.95	14.664	27.619	24" SD	Between R/W 8-26 & T/W A
1513	0.1365	82.36	84.04	5.746	10.552		N. side T/W A
1514	0.5389	165.60	219.55	20.965	39.139	48" SD	N. side T/W A
1515	0.5835	180.45	285.03	22.977	42.785	66" SD	Between T/W A & R/W 8-26
1516	0.0300	32.57	55.36	1.340	2.428	24" SD	Between R/W 8-26 & T/W E
Σ1	0.1705	81.67	97.46	8.785	15.223	48" CLV	to landfill
Σ2	0.6135	193.6	317.04	24.317	45.213	66" CLV	to RR Spur
1600	0.1890	136.70	230.18	11.121	18.441	54" SD	Air Guard
1601	0.0160	25.67	41.72	0.843	1.447	54" SD	E end 8-26
Σ	.2050	128.1	187.14	11.963	19.888	60"SD	to E. of 6000 area

Q. **Far East Basin.** This 0.24 square mile basin is almost entirely within KAFB with the northeast portion of the basin consisting of developed, technical areas and the open, graded approach areas for Runway 8-26. The drainage system consists of a series of roadside channels and culverts that pass near the southeast corner of the airport. The existing crossing onto AIA property consists of 3-24" culverts that have an approximate capacity of 45 cfs with the excess runoff spilling and ponding to the east of the runway. Including the airport runoff, the system discharges 45 cfs into the arroyo south of the airport.

Since the extension of Taxiway E crosses the existing drainage facilities, the new system consists of providing a 48" culvert to pass this runoff under the taxiway, then to its historic arroyo and a proposed inlet to drain the far east end of the taxiway. Under the 10 year design storm, the airport drainage has a design discharge of 9 cfs with the majority of the runoff occurring prior to the peak discharge from KAFB. Based on the master plan, the culvert extension was designed to handle the entire peak discharge if KAFB upgrades their system in the future with this flow being 81 cfs. For the 100 year design storm, excess ponding capacity within the taxiway island is sufficient to detain the excess runoff beyond the 10 year storm. The arroyo that drains this basin is also entirely within KAFB property.,

TABLE III-15
Far East Basin

SUB-BASIN NO.	AREA (sq mi)	Q10 (cfs)	Q100 (cfs)	V10 (ac-ft)	V100 (ac-ft)	OUTFALL	DESCRIPTION
1700	0.2260	79.46	153.96	9.261	17.265	30" SD 48" SD	NE of R/W 8-26 end
1701	0.0160	24.24	40.13	0.772	1.363	54" SD	SE of R/W 8-26 end
Σ	0.2420	46.40	96.82	10.03	18.628	54" SD	to KAFB

R. **Other Areas.** There are numerous other drainage areas which are technically inside the airport property and drain offsite. These areas include those steep slopes along the side of Tijeras Arroyo between the airport and the railroad at the toe of this slope. These areas do not affect airport activities, and in most cases, are not impacted by airport activities. A summary of these drainage areas is shown in the following table.

S. **Summary of Existing Area Problems.** The following is a summary of the problems encountered in the existing system.

1. **Basin G17.** The storm sewer collecting runoff from the areas around the extreme north end of Runway 17-35 discharges into the storm sewer in Gibson Boulevard. Approximately half way between Yale and Girard and just south of Gibson this sewer makes a 90 degree bend to enter the Gibson system. At this severe bend, water leaves the manhole and floods the surface.

2. **Basin N17.** There are two principal problems in this basin. First, there is a restrictor plate installed in the storm sewer conveying around the terminal building. Second, there is some stormwater exiting the storm sewer just north of the lower parking lot and adjacent to the commercial development areas along Alamo Drive. The restrictor plate was installed to correct the flooding problem at this location.

3. **Basin WE.** The 2-60" culverts under University have a capacity of 100 cfs whereas the existing flows are 370 cfs.

4. **Basin WE.** Subbasin 504, between the B and C Concourses on the terminal apron ponds significantly during typical storms.

5. **Basin GA.** Crossing structures on University Boulevard have inadequate inlet capacities.

6. **Basins SE35, SW35, W3, and SE3.** AMAFCA and City have expressed concerns over erosion off the south side of the airport in the steep areas contributing to sedimentation problems ownstream.

SECTION IV

PROPOSED CONDITIONS

A. **General.** The proposed development at AIA considered in this study is based on recommendations contained in the Albuquerque International Airport Master Plan, December 1993, prepared by Coffman Associates, Inc. In addition, several improvements to the storm drainage system are analyized in this study to relieve several problems found with the existing system. The improvements recommended to relieve the existing problems are identified in-depth in Section III, Existing Conditions.

B. **Proposed Improvements - Master Plan.** Several major changes to Albuquerque International Airport are recommended in the 1993 master plan update that will significantly affect the storm drainage systems over the planning period. The proposed improvements are illustrated on Plate 3 in Appendix G. In summary the major improvements are listed below:

1. Upgrade of Runway 3-21 to air carrier status including construction of new connector taxiways and upgrade of existing taxiways.

2. Closure of Runway 17-35 and expansion of the terminal to the east of the existing parking structure.

3. Construction of new Air Cargo Facility south east of Runway 3-21.

4. Development of airport support areas generally on the west side of the airport property and east of University Blvd.

In addition to development on AIA property, it is expected that UNM will continue to develop the property west and south of Spirit Drive. Drainage impacts from this development will be seen in the GA Basin discharge and in the crossing at University Blvd. and Spirit Drive. UNM has addressed the impacts from development that affects the University Blvd/Spirit Drive crossing in the UNM Business Park Drainage Report, February 1993. Impact from additional flow and volume discharging through GA Basin will be addressed in this study.

C. **Basins East of Runway 17-35.** Far East End, Air Guard, Eend, and Middle Basins are considered to be at full development with the existing conditions. There is no expansion planned by the Airport in these areas within City property. KAFB and the NMANG may develop the areas to the north of the airfield contained in these basins but the existing storm systems, upgraded recently under the Taxiway E and A projects, have been designed and constructed to limit discharges to existing values.

The discharge from the Middle Basin is controlled at 81cfs and 64.4 AF for the 100-year, 6-hour storm. These values assume the KAFB will improve their collection system in the future, tieing onto the AIA system at connection points constructed in the Taxiway A Reconstruction project.

The existing EEND Basin is considered fully developed under this study. The portion within Airport property was expanded under the Taxiway E and Taxiway A projects. Under the master plan, two connector taxiways are planned in the south central portion of the basin. The addition of these taxiways will not significantly impact the recently constructed system. The discharges from the EEND Basin fall within KAFB property. Improvements are planned by KAFB for both of these discharge points due to their impact on an abandoned landfill. The improvements are not considered in this study since they fall downstream of the Airport and are within KAFB jurisdiction.

The Far East End Basin lies predominantly within KAFB property off the east end of the Airport. Improvements made during the Taxiway E project were sized to handle developed conditions on the airfield. If future development occurs on Air Force property, KAFB should be responsible for any increase in discharge or volume to the Airport system.

C. **G-17 Basin.** The existing conditions in Basin G17 is are assumed to be fully developed under this study. Subbasins 101 and 102 lie within KAFB and are predominantly vacant. Capacity of the storm drainage system serving these areas will not allow additional flows to be discharged. Diversion to the storm sewer system in Gibson Blvd. is not an option according to City of Albuquerque Public Works because of the existing stress on the system from current conditions. Diversion of subbasins 101, 102, 103 and 104 to N17 Basin at subbasin 206 is recommended after improvements are made within N17 Basin. The diversion will help relieve capacity problems in subbasin 108 at the 90-degree bend upstream of the basin discharge to Gibson Blvd.

Downstream of subbasin 104, the existing storm sewer system serves a light industrial and business park. It does not appear that additional significant development can occur in these subbasins. The existing drainage system is adequate for the 10-year event with the exception of the 90 degree turn in subbasin 108. Improvement to this bend will relieve flooding problems on the surrounding private property. Appendix H contains a conceptual scope for the improvements in this area.

D. **N17 - KC Basins.** Significant changes are proposed for N-17 Basin in the AIA master plan. Most significant is the proposed terminal expansion in the upper reaches of N-17 basin, specifically in subbasins 203, 204, 207, 208, 209, 210, 211, 212, 213, and 220. Currently, this area contains most of the northern portion of Runway 17-35 and Taxiway C. If Runway 17-35 is closed and the terminal is expanded into this area, it is assumed that the majority of the area will be impervious.

Under the proposed conditions outlined in the master plan, the new aircraft parking apron which will surround the new terminal area will cover subbasins 203, 204, 207, 208 and 209. Subbasin 209 is a consolidation of subbasins 209, 210, 212, 213 and a portion of 211. An increase in the storm drain system will be required to carry the increased flows from the apron and terminal as outlined below in Table IV-1.

To initiate the analysis of the lower reaches of the Kirtland Channel, general flow characteristics of the channel were computed. These are as follows:

Q	=	950 cfs			
Yn	=	2.58'	Yc	=	4.92'
n	=	0.016			
B	=	10'			
H/V	=	1.74			
Vn	=	25.44 fps	Vc	=	10.41 fps
S	=	0.0334 ft/ft	Sc	=	0.00282 ft/ft
TWn	=	18.98'			
An	=	37.38sf			

Obviously, flow in the channel is well within the supercritical flow regime and velocities are quite high. Using the DPM criteria, superelevation of the water surface was checked as follows:

$$Se = 1.3\ \frac{V^2(b+2zD)}{(g)(r)} = (1.3)\ \frac{(25.44^2)(10+(2)(1.74)(2.58)}{(32.2)(800)} = 0.62 \text{ ft.}$$

Based on these calculations, the problems experienced with the overtopping the channel side should not be occurring as the maximum outside radius depth of flow is only 3.20 feet. The channel has a lined depth of five feet. Additional freeboard is needed to maintain a two foot minimum but the overtopping of flows should not be occurring strictly from the superelevation.

Another source of the problem could involve the box culvert capacity. Using the DPM criteria, the pier losses were calculated and the results show that this box does, indeed, have proper capacity. Constructing a HEC-2 (Water surface profiles analysis) was not done as it was felt that HEC-2 would not accurately model the complex variables occuring at this location.

It is somewhat apparent that the problem is being caused by the reverse curves in the channel alignment. Since the flow in the channel is supercritical, the box itself cannot be causing a backwater affect unless the box has insufficient capacity. Calculations show that the box does have capacity. Therefore, the only affect that the box has on the flow is some disturbance at the center pier, an affect which cannot be recognized further upstream. If the flow in the channel were subcritical, then such an affect would be felt some distance upstream. While some effort has been made to increase freeboard along the channel, the success of these improvements is limited due to the presence of an additional channel discharge just upstream of the box culvert. This side channel cannot be blocked by increasing freeboard on the Kirtland Channel.

The problems experienced in this area are complex. Solutions cannot be achieved by computer analysis, but rather by a more logical approach. The Kirtland Channel should be reconstructed in a new location, further south of the existing alignment. This reconstruction should occur between the most upstream horizontal curve and the South Diversion Channel. This area is currently undeveloped and some flexibility would exist in both horizontal and vertical alignment. This reconstruction would allow the two horizontal curves to be lengthened causing less superelevation or eliminated completely. Furthermore, the side discharge from the residential areas to the north could be realigned to enter the Kirtland Channel at a more desirable location, without the potential of Kirtland flows backing up into this side channel. The box culvert could also be eliminated with this channel realignment. This is dependent upon the future planning of

the undeveloped areas surrounding the Mulberry crossing. If Mulberry will not be extended south in the future, no channel crossing will be needed.

TABLE IV-1
Improvements to N17 Basin

SUBBASIN NO.	PIPE DIAMETER	FLOW, 10 and 100 year, cfs	COMMENTS
204-207	30-inch	19.6 (10), 32 (100)	North N17 to center N17.
203-208	36-inch	24 (10), 37 (100)	Flow from 203 to 208
208-207	48-inch	49.3 (10), 76.2 (100)	South N17 to center N17
207-209	66-inch	164 (10), 255 (100)	Flow from 207 to 209
216-215	30-inch	13.8 (10), 20.3 (100)	Currently vacant parcel
220	60-inch	272.3 (10), 430.7 (100)	Flow from east terminal area
209-215-220	66-inch	272.3 (10), 430.7 (100)	Flow from new apron and terminal
214.2	42-inch	112 (10)	Flow from exist. terminal area, parking, rental lots, Craddock.
214.2-Alamo	72-inch	403 (10) 558 (100)	Flow from exist. terminal area, parking, rental lots, and new loop road curbside to new terminal.

The proposed terminal expansion will be drained through subbasin 207 collecting flows from the northern and southern portion of N17 Basin with new 30-inch and 48-inch drains, respectfully. Subbasin 209, consolidated, will accept the flow from subbasin 207 and discharge through a new 60-inch drain. This drain will increase in size to a 66-inch drain running west on the south side of the Craddock Business Park. After flow is collected from subbasin 214.2, incorporating discharges from the existing terminal area, parking area and rental car lots, a 72-inch drain will carry the 10-year flow to Alamo St. The existing 60-inch drain in Alamo St. will discharge across Yale Blvd. into the upstream reaches of the Kirtland Channel.

The current storm sewer system serving the existing terminal, parking, loop road, and rental car lot subbasins are restricted by the capacity of the 36-inch drain across subbasin 214.2 An orifice plate was installed at the discharge point of subbasin 220 when the existing terminal was expanded to control the discharge to approximately 30 cfs. Removal of this plate is recommended with the increase in capacity of the downstream system in subbasin 214.2 where the 36-inch drain will be replaced with a new 42-inch drain through subbasin 214.2. A new 72-

inch drain from northwest corner of subbasin 214.2 to Alamo St. is recommended to replace the existing 60-inch drain.

Subbasins 227 and 228, which serve the south end of Yale Blvd., will continue to discharge into the Kirtland Channel system through the 60-inch Yale crossing near Alamo St. and Yale Blvd. The conditions in the Yale system will be relieved with the construction of Sunport Blvd. which diverts a significant amount of the storm water from the north and west side of the existing terminal. This diversion is discussed in and Section IV-E.

E. **SP Basin.** SP Basin contains Sunport Boulevard, George Road, a portion of University Blvd., and various commercial tracts of land between the airport and the South Diversion Channel. Sunport Blvd. and the relocation of George Road are currently under design with construction scheduled to start Fall, 1994.

Incorporated into the Sunport Blvd. project is a system which will collect storm water runoff from the new roadways from the terminal building to Interstate 25 and from surrounding property. The storm drainage system within the Sunport right of way was designed using the 100-year, 6-hour event. Sunport Blvd. will be an elevated roadway from the Interstate to a point approximately 700-feet east of University Blvd. The drainage system will be installed at an elevation which will enable a significant amount of the surrounding properties to discharge into it. Discharge from the new Sunport System will be through existing crossing points at Interstate 25.

A major diversion is proposed in the Sunport project at its eastern terminus with Yale Blvd. The existing system which transports storm flows from the west terminal area, subbasin 222 on Plate 1, will be severed by the new Sunport system. The diversion will amount to approximately 77 cfs and 3.2 a-f under present conditions. Storm runoff from the old airport terminal building and the USPS Air Mail Facility will be collected by the new system installed during the relocation of George Road.

A detention basin is proposed south of Sunport Blvd. and west of University Blvd. for the attenuation of peak flows discharged from subbasins 400, 401, 401.1, 401.2, 402, 401.1 and 402.3. The basin is required to protect the existing 3-36" RCP culverts under Interstate 25 west of subbasin 404. Without the detention basin, an additional crossing would be required at the Interstate. Discharge from the basin through a riser with an orifice plate will be controlled at approximately 85 cfs. Total pond storage volume to the emergency spillway crest is approximately 7.6 a-f. This facility was designed using the 100-year, 24-hour event per the City of Albuquerque and New Mexico State Highway and Transportation Department.

The Master Plan identifies the parcels of land adjacent to the relocated George Road as future airport support areas. The design of the drain systems in the Sunport project have taken into account the development of these properties. In addition, property within the SP Basin was considered fully developed for the design of the Sunport storm drainage system an as such, this study does not incorporate any additional development scenarios for analysis. Reference is made to the Sunport Boulevard Drainage Report, February, 1994, Molzen-Corbin & Associates, for additional and specific information regarding the design of the Sunport storm drainage system.

F. **WE Basin.** The WE Basin incorporates the area south and west of the existing terminal area with boundaries of Runway 12-30 to the south, Taxiway C to the east, the existing terminal building to the north and its discharge point to the South Diversion Channel to the west.

Changes to the facilities in this basin that are proposed under the Master Plan include the demolition of the old Air Freight Building located within subbasin 505. Extension of the existing storm drain system is recommended to follow the demolition of the building to relieve ponding occuring on the terminal apron between the B and C concourses. The extension of the drain (a 36-inch enlarged to a 42-inch connecting to the existing 60-inch) was planned during the Taxiway E Improvements project. A stub was installed at the 60-inch tee constructed near the northwest corner of Runway 8.

Downstream of the airfield, WE Basin discharges into a rip-rap lined channel and detention pond east of University Blvd. In order to maintain the future flows to near existing conditions, it is recommended that a restrictor plate be installed upstream. A 48-inch equivalent orifice installed at the 60" x 60" x 48" junction near the southwest corner of Runway 8 would help maintain existing conditions.

There is expected to be development west of University Blvd. in the Sunport Business Park, specifically in subbasins 514 and 515. Discussions between the developer and the City indicate that the natural arroyo that carries the basin discharge may be relocated to the south property boundary, approximately on the southern border of subbasins 514 and 515. The channel will run west until it nears Interstate 25 where it will be turned north to discharge through the Interstate at the 6'x 8' concrete box culvert. The future conditions modelling does not indicate the relocation of the arroyo and does not indicate that there will be any capacity problems at the Interstate crossing.

The Table IV-2 indicates the addition of the storm drain into subbasin 504 from the 60-inch drain. There are not any other improvements recommended as far as new drain systems in WE basin.

TABLE IV-2
Improvements to WE Basin

SUBBASIN NO.	PIPE DIAMETER	FLOW, 10 and 100-year, cfs	COMMENTS
504-505A	36-inch	39.2 (10) 43.4 (100)	Pipe to collect discharge from subbasin 504.
505A-60" tee	42-inch	57.6 (10) 59.0 (100)	Pipe collects NW corner of 505

G. **GA Basin.** The Master Plan identifies several parcels within the GA Basin which are recommended for development. With these parcels developed, the GA Basin should be

considered essentially fully developed. Subbasins 720, 721, and 724 contain the parcels which have been identified as airport support areas and will be typical of a light industrial or business park.

The capacity problem at the University crossing (60" and 36" CMP) with the 100-year event will be resolved with the proposed diversion of portions of the GA Basin to the M3 Basin discussed later in this section. Modelling indicates that the pond on the east side of University Blvd. will exceed the available capacity. During the 100-year event, a peak flow, 100-year, will reach approximately 320 cfs with a volume of 32.5 a-f resulting in over topping of University Blvd. With the recommended diversion upstream, the 100-year future flow is 299 cfs with a volume of approximately 28 a-f. This results in approximately two feet lowering of the maximum water surface.

There are no other improvements proposed for GA Basin in this study. As development is continued as recommended in the Master Plan, discharge to the University Arroyo should be metered to maintain the existing conditions. Construction of a detention pond near the discharges of the three branches of GA Basin with an outflow controlled to existing conditions would also help maintain the crossing under University Blvd.

H. <u>M3 Basin (SE35, SW35, W3, SE3).</u> Under the future conditions, SE35, SW35, W3, SE3 and M3 Basins have been consolidated to form one basin with a detention pond at the discharge point off of the airfield. The intent of the consolidation is to provide a common controlled discharge point off the southeast side of upgraded Runway 3-21, new Air Cargo Freight Facility, expansion on the existing Air Freight Ramp, and to relieve GA Basin by diverting flow from the north and east sides of the General Aviation Area. Consolidation of five discharge points at natural arroyos into one discharge from a detention pond with a controlled outlet will also allow more efficient storm water monitoring if required under AIA's EPA NPDES storm water permit.

The changes proposed in the Master Plan for this part of the airport will fully develop the basins. Additional capacity is required in the storm drains as well as additional trunk lines required to consolidate the flow. It anticipated that the majority of the storm drains will be constructed as part of the Runway 3-21 Upgrade project. An exception may be the new trunk line that will collect discharge from the proposed Air Cargo Facility. Installation of this line could occur when the new apron is constructed delaying the diversion of existing SE35 and approximately half of SW35 flows to the detention pond. Existing drains have the capacity for the additional flows which will result with construction of the northeastern reach of the southern parallel taxiway for Runway 3-21 and the extension of Taxiway D, south of Taxiway E.

Table IV-3 contains the new drains that would serve a consolidated M3 Basin as shown on Plate 3. All of the collector lines are not included in Table IV-XX. In most cases, these lines conceptually are 24 and 30-inch drains. The middle branch depends primarily on existing lines with the exception of new collector lines in subbasins 902 and 901.1

TABLE IV-3
Improvements to M3 Basin

SUBBASIN NO.	PIPE DIAMETER	FLOWS, 10 and 100 year, cfs	COMMENTS
609-608	60-inch	56.3 (10) 82.9 (100)	Drain for SE35
608-607	66-inch	113.4 (10) 171.6 (100)	Across new taxiway. Carries flow to new Air Cargo Apron
607-616	72-inch	442 (10) 676 (100)	GA diversion, SE35 diversion, most of 616
616-Pond	84-inch	442 (10) 700 (100)	East branch discharge to pond
1104-1103	30-inch	19.4 (10)	Collector in 1104 and 1103
1103-pond	60-inch	144 (10) 179 (100)	Discharge from west branch to pond

There are three branches in M3 Basin. SE35 and SW35 along with diversion from a portion of the north side of GA Basin make up the east branch. The middle branch contains the diversion from the southeastern portion of GA Basin as well as M3 Basin and a portion of W3 Basin. The south branch contains SE3 and a portion of W3.

The location of the detention pond is conceptually selected for its central location and because it is currently a major discharge point for existing flows from M3 and W3 Basins. The detention pond is sized to accept 100-year, 6-hour flows and volumes. The volume of the pond should accept approximately 75 a-f of storm water at a peak flow of approximately 1000 cfs. Discharge from the detention pond will be controlled at approximately 95.2 cfs with a volume of approximately 83 a-f. This discharge is equivalent to the 10-year flow for the existing outfalls combined. Consolidation of five discharge points in the existing basins will also allow more efficient storm water discharge monitoring.

I. **UE, SW3 Basin.** The UE and SW3 Basins within airport property lie off of the airfield on steep terrain. Future development is restricted in these basins due to the terrain and proximity to the protected zones for Runway 3-21 after it has been upgraded to air carrier status. UE Basin does contain developable property which is owned by UNM and identified as subbasins 1201, 1202 and 1203 on Plate 1. Subbasins are addressed as Basin C1 in Drainage Report and Conceptual Grading Plan for UNM Business Park, Jan. 1993, Bohannon-Huston Inc. This report indicates that the existing 48-inch crossing under University Blvd. has sufficient capacity to handle developed flows from this basin.

Subbasin 1201 is identified as Basins A-4, A-3 and C-2 in the Bohannon-Huston report. Flows from these A-3 and A-4, the most northern portion of subbasin 1201, will be carried by a storm drain to University Blvd. then to a detention basin located to the northwest of the existing 48-

inch crossing. The drain system will designed to accommodate the 100-year runoff. Discharge from the pond will be controlled to a level less than existing. Pond capacity is proposed at 4 a-f.

J. **Middle, Eend, Air Guard, Far East End Basins.** These basins, which lie to the east of Runway 17-35 and discharge to the south through KAFB property, were analyzed under the Taxiway E and A improvement projects in order that systems which would handle fully developed flows from KAFB and the airfield were constructed while Runway 8-26, Taxiway A and Taxiway E were closed. This study does not address additional development or future conditions for these basins due to the capacity of the upgraded systems constructed. Section III, Existing Conditions, contains as-constructed information on these basins. The existing systems are considered adequate to accommodate foreseeable future conditions.

SECTION V

COST ESTIMATES

The attached conceptual cost estimates have been developed for the improvements outlined in Section III and IV of this study. It is understood that priorities and scheduling of projects will play an important role in the construction of the improvements. Construction of upgrades to Runway 3-21 in the near future could incorporate many of the improvements recommended for the M3 Basin. Furthermore, the Master Plan CIP program has outlined a schedule for the other capital projects recommended for Albuquerque International Airport including the expansion of the terminal and apron with the closure of Runway 17-35. This document should be viewed in tandem with the Master Plan CIP to schedule construction of the drainage system improvements.

Albuquerque International Airport
Drainage Master Plan
Conceptual Cost Estimate
Improvements to N17 Basin and KC Basin

ITEM NO.	ITEM	QUANTITY	UNIT	UNIT PRICE	TOTAL COST
	N-17/KC Basin				
1.	30" SD, complete in place	1500	LF	$55.00	$82,500.00
2.	36" SD, completed in place	300	LF	$65.00	$19,500.00
3.	42" SD, completed in place	400	LF	$73.00	$29,200.00
4.	48" SD, completed in place	1300	LF	$83.00	$107,900.00
5.	60" SD, completed in place	1400	LF	$95.00	$133,000.00
6.	72" SD, completed in place	550	LF	$130.00	$71,500.00
7.	Remove Exist. SD,< 36" dia.	1700	LF	$20.00	$34,000.00
8.	Remove Exist. SD,> 36" dia.	3000	LF	$35.00	$105,000.00
9.	6' dia. Manhole/Drop Inlet	20	EA	$2,500.00	$50,000.00
10.	Remove Restrictor Plate	1	LS	$2,500.00	$2,500.00
11.	Remove and Replace AC Pavement	5000	SY	$28.00	$140,000.00
12.	Remove Manhole	10	EA	$300.00	$3,000.00
13.	New Channel Section	1500	LF	$110.00	$165,000.00
14.	Outlet Structure	1	LS	$10,000.00	$10,000.00
15.	Remove Old Channel Section incl. backfill	1500	LF	$20.00	$30,000.00
				Subtotal	$983,100.00
				Contingencies, Eng., Insp., Testing @25%	$245,775.00
				NMGRT @ 5.8125%	$71,428.36
				TOTAL	**$1,300,303.36**

Albuquerque International Airport
Drainage Master Plan
Conceptual Cost Estimate
Improvements to G17 Basin

ITEM NO.	ITEM	QUANTITY	UNIT	UNIT PRICE	TOTAL COST
	G-17 Basin				
1.	30" SD, complete in place	500	LF	$55.00	$27,500.00
2.	36" SD, completed in place	400	LF	$65.00	$26,000.00
3.	Remove Exist. SD,< 36" dia.	900	LF	$20.00	$18,000.00
4.	6' dia. Manhole/Drop Inlet	1	EA	$2,500.00	$2,500.00
5.	Junction Box	1	LS	$10,000.00	$10,000.00
6.	Remove Manhole	1	EA	$300.00	$300.00
7.	Remove and Replace AC Pavement	1500	SY	$28.00	$42,000.00
				Subtotal	$126,300.00
				Contingencies, Eng., Insp., Testing @25%	$31,575.00
				NMGRT @ 5.8125%	$9,176.48
				TOTAL	**$167,051.48**

Albuquerque International Airport
Drainage Master Plan
Conceptual Cost Estimate
Improvements to M3 Basin

ITEM NO.	ITEM	QUANTITY	UNIT	UNIT PRICE	TOTAL COST
	M3 Basin				
1.	24" SD, complete in place	2200	LF	$45.00	$99,000.00
2.	30" SD, complete in place	1300	LF	$55.00	$71,500.00
3.	36" SD, completed in place	2800	LF	$65.00	$182,000.00
4.	42" SD, completed in place	400	LF	$73.00	$29,200.00
5.	48" SD, completed in place	2000	LF	$82.00	$164,000.00
6.	60" SD, completed in place	2100	LF	$95.00	$199,500.00
7.	66" SD, completed in place	800	LF	$110.00	$88,000.00
8.	72" SD, completed in place	3000	LF	$130.00	$390,000.00
9.	84" SD, completed in place	1000	LF	$150.00	$150,000.00
10.	Remove Exist. SD,< 36" dia.	9900	LF	$20.00	$198,000.00
11.	Remove Exist. SD,> 36" dia.	5400	LF	$35.00	$189,000.00
12.	6' dia. Manhole/Drop Inlet	62	EA	$2,500.00	$155,000.00
13.	Remove Manhole/Drop Inlets	45	EA	$300.00	$13,500.00
14.	Remove and Replace AC Pavement	800	SY	$28.00	$22,400.00
15.	Detention Basin Excavation	105000	CY	$2.15	$225,750.00
16.	Slope Protection	10000	SY	$10.00	$100,000.00
17.	Outlet Structure	1	LS	$5,000.00	$5,000.00
18.	Discharge Pipe - 60"	2500	LF	$95.00	$237,500.00
				Subtotal	$2,519,350.00
				Contingencies, Eng., Insp., Testing @25%	$629,837.50
				NMGRT @ 5.8125%	$183,046.52
				TOTAL	**$3,332,234.02**

SECTION VI

WATER QUALITY

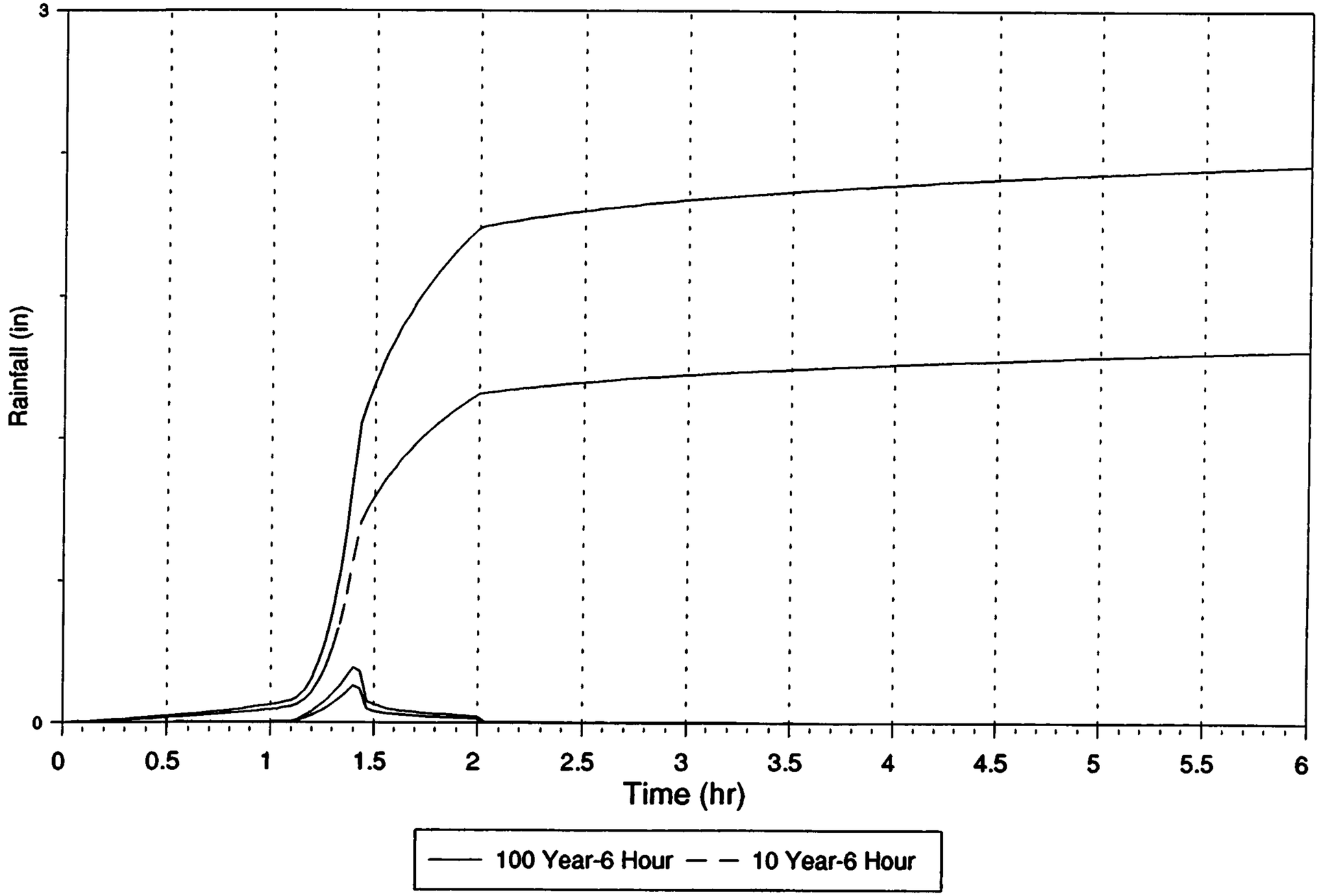
Albuquerque International Airport has submitted a statement of intent to follow the guidelines and requirements for the general Storm Water NPDES permit issued by U.S. Environmental Protection Agency. The scope of this project does not contain requirements to analyze storm water quality. Several of the recommendations will have certain impacts on this issue including consolidation of discharge points of the south side of the airport into Tijeras Arroyo, installation of piped systems and eliminating surface paths and consolidation of inflow points from offsite flows originating on KAFB property. It is not the intent of this study and report to improve the quality of the storm water discharged from the airport, however.

APPENDIX A

FIGURES

1 Design Storm Rainfall Distributions

AIA MASTER DRAINAGE PLAN
DESIGN STORM RAINFALL DISTRIBUTIONS



APPENDIX B

SYSTEM FLOW CHARTS

(See separate binder)

APPENDIX C

SUMMARY PRINTOUTS

APPENDIX C

SUMMARY PRINTOUTS
BASIN G17

AHYMO SUMMARY TABLE (AHYM0491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/17/1994

INPUT FILE = g17[redacted] USER NO.= MOLZCONM.101

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	1.570
COMPUTE NM HYD	101.00	-	3	.02840	32.42	1.382	.91269	1.567	1.784	PER IMP=	50.00
ROUTE	102.10	3	7	.02840	14.86	1.382	.91269	1.833	.819		
COMPUTE NM HYD	102.00	-	3	.02820	22.68	.384	.58806	1.500	1.256	PER IMP=	15.00
ADD HYD	102.20	3& 7	2	.05660	33.74	2.267	.75092	1.633	.931		
ROUTE	103.10	2	7	.05660	31.21	2.267	.75092	1.733	.862		
COMPUTE NM HYD	103.00	-	3	.00560	7.38	.237	.79281	1.500	2.060	PER IMP=	45.00
ADD HYD	103.20	3& 7	2	.06220	34.00	2.504	.75468	1.700	.854		
ROUTE	104.10	2	7	.06220	33.46	2.304	.75469	1.766	.841		
COMPUTE NM HYD	104.00	-	3	.00880	12.63	.417	.88890	1.500	2.242	PER IMP=	55.00
ADD HYD	104.20	3& 7	2	.07100	38.05	2.921	.77131	1.733	.837		
ROUTE RESERVOIR	504.00	2	4	.07100	32.37	2.921	.77131	1.900	.712	AC-FT=	.237
COMPUTE NM HYD	106.10	-	3	.00170	2.06	.061	.67705	1.500	1.898	PER IMP=	30.00
ADD HYD	106.20	3& 4	2	.07270	32.77	2.982	.76910	1.900	.704		
ROUTE	105.10	2	7	.07270	32.74	2.982	.76911	1.900	.704		
COMPUTE NM HYD	105.00	-	3	.00930	14.00	.479	.96578	1.500	2.353	PER IMP=	65.00
ADD HYD	105.20	3& 7	2	.08200	36.55	3.461	.79141	1.866	.696		
ROUTE	107.10	2	7	.08200	36.55	3.461	.79141	1.866	.696		
COMPUTE NM HYD	107.00	-	3	.00740	11.59	.374	.94856	1.500	2.448	PER IMP=	55.00
ADD HYD	107.20	3& 7	2	.08940	47.40	3.835	.80441	1.533	.828		
COMPUTE NM HYD	106.00	-	3	.00330	6.30	.221	1.25417	1.500	2.984	PER IMP=	90.00
ADD HYD	107.30	3& 2	4	.09270	53.33	4.056	.82042	1.533	.899		
ROUTE	108.10	4	7	.09270	52.06	4.056	.82042	1.567	.877		
COMPUTE NM HYD	108.00	-	3	.00870	15.82	.543	1.17087	1.500	2.842	PER IMP=	80.00
ADD HYD	108.20	3& 7	2	.10140	66.97	4.599	.85048	1.533	1.032		

AHYMO SUMMARY TABLE (AHYM0491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/17/1994

INPUT FILE = [redacted] USER NO.= MOLZCONM.101

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	2.350
COMPUTE NM HYD	101.00	-	3	.02840	54.60	2.394	1.58050	1.567	3.004	PER IMP=	60.00
ROUTE	102.10	3	7	.02840	27.10	2.394	1.58050	1.833	1.491		
COMPUTE NM HYD	102.00	-	3	.02820	42.56	1.927	1.21480	1.600	2.358	PER IMP=	15.00
ADD HYD	102.20	3& 7	2	.05660	63.21	4.221	1.39827	1.633	1.745		
ROUTE	103.10	2	7	.05660	59.69	4.221	1.39828	1.733	1.648		
COMPUTE NM HYD	103.00	-	3	.00560	12.87	.431	1.44195	1.500	3.592	PER IMP=	45.00
ADD HYD	103.20	3& 7	2	.06220	65.13	4.652	1.40219	1.700	1.636		
ROUTE	104.10	2	7	.06220	64.12	4.652	1.40220	1.733	1.611		
COMPUTE NM HYD	104.00	-	3	.00880	21.35	.732	1.55992	1.500	3.790	PER IMP=	55.00
ADD HYD	104.20	3& 7	2	.07100	72.59	5.384	1.42174	1.733	1.597		
ROUTE RESERVOIR	504.00	2	4	.07100	67.79	5.384	1.42174	1.833	1.492	AC-FT=	1.161
COMPUTE NM HYD	106.10	-	3	.00170	3.71	.119	1.30780	1.500	3.408	PER IMP=	30.00
ADD HYD	106.20	3& 4	2	.07270	68.80	5.502	1.41907	1.833	1.479		
ROUTE	105.10	2	7	.07270	68.74	5.502	1.41908	1.833	1.477		
COMPUTE NM HYD	105.00	-	3	.00930	23.25	.817	1.64736	1.500	3.906	PER IMP=	65.00
ADD HYD	105.20	3& 7	2	.08200	75.76	6.319	1.44496	1.833	1.444		
ROUTE	107.10	2	7	.08200	75.81	6.319	1.44496	1.833	1.444		
COMPUTE NM HYD	107.00	-	3	.00740	18.99	.651	1.65033	1.500	4.009	PER IMP=	55.00
ADD HYD	107.20	3& 7	2	.08940	81.72	6.971	1.46195	1.800	1.428		
COMPUTE NM HYD	106.00	-	3	.00330	9.72	.355	2.01662	1.500	4.604	PER IMP=	90.00
ADD HYD	107.30	3& 2	4	.09270	85.10	7.326	1.48170	1.800	1.434		
ROUTE	108.10	4	7	.09270	84.87	7.326	1.48170	1.833	1.431		
COMPUTE NM HYD	108.00	-	3	.00870	24.74	.890	1.91787	1.500	4.444	PER IMP=	80.00
ADD HYD	108.20	3& 7	2	.10140	102.89	8.215	1.51912	1.533	1.586		

APPENDIX C

SUMMARY PRINTOUTS
BASIN N17

AHYMO SUMMARY TABLE (AHYMO491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/17/1994

INPUT FILE = [redacted] USER NO.= MOLZCONM.I01

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	1.570
COMPUTE NM HYD	201.00	-	3	.01290	5.38	.538	.78198	2.166	.652	PER IMP=	36.00
COMPUTE NM HYD	205.00	-	7	.01380	20.14	.631	.85709	1.500	2.281	PER IMP=	45.00
ADD HYD	205.10	3& 7	2	.02670	20.35	1.169	.82077	1.500	1.191		
ROUTE RESERVOIR	505.00	2	4	.02670	10.50	1.169	.82077	1.700	.614	AC-FT=	.186
ROUTE	205.20	4	7	.02670	10.46	1.169	.82079	1.700	.612		
COMPUTE NM HYD	206.00	-	3	.01380	21.38	.692	.94057	1.500	2.421	PER IMP=	55.00
ROUTE RESERVOIR	506.00	3	4	.01380	10.08	.692	.94055	1.667	1.141	AC-FT=	.206
ADD HYD	206.10	4& 7	2	.04050	20.50	1.861	.86159	1.700	.791		
ROUTE	206.20	2	7	.04050	20.34	1.861	.86160	1.733	.785		
COMPUTE NM HYD	204.00	-	3	.02170	31.28	.973	.84040	1.500	2.252	PER IMP=	43.00
ROUTE RESERVOIR	504.00	3	4	.02170	8.56	.973	.84038	1.800	.617	AC-FT=	.485
ROUTE	504.10	4	5	.02170	8.56	.973	.84039	1.833	.617		
ADD HYD	207.10	5& 7	2	.06220	28.78	2.834	.85418	1.766	.723		
COMPUTE NM HYD	203.00	-	7	.02100	28.89	.951	.84875	1.500	2.150	PER IMP=	44.00
ROUTE RESERVOIR	503.00	7	6	.02100	8.93	.951	.84873	1.800	.664	AC-FT=	.430
ROUTE	203.20	6	7	.02100	8.92	.951	.84873	1.833	.664		
COMPUTE NM HYD	208.00	-	3	.02250	26.54	1.079	.89883	1.567	1.843	PER IMP=	50.00
ROUTE RESERVOIR	508.00	3	4	.02250	14.03	1.079	.89881	1.800	.974	AC-FT=	.340
ADD HYD	208.10	4& 7	5	.04350	22.95	2.029	.87461	1.800	.824		
ROUTE	208.20	5	7	.04350	22.78	2.029	.87462	1.866	.818		
ADD HYD	207.20	2& 7	8	.10570	51.27	4.863	.86259	1.800	.758		
COMPUTE NM HYD	207.00	-	3	.02190	28.34	1.050	.89883	1.533	2.022	PER IMP=	50.00
ROUTE RESERVOIR	507.00	3	4	.02190	15.46	1.050	.89881	1.733	1.103	AC-FT=	.289
ADD HYD	207.30	4& 8	2	.12760	66.58	5.913	.86881	1.766	.815		
ROUTE	207.40	2	7	.12760	66.52	5.913	.86881	1.800	.815		
COMPUTE NM HYD	209.00	-	3	.01320	9.41	.633	.89883	1.833	1.114	PER IMP=	50.00
ROUTE RESERVOIR	509.00	3	4	.01320	9.26	.633	.89877	1.866	1.096	AC-FT=	.045
ADD HYD	209.10	4& 7	2	.14080	75.33	6.545	.97161	1.800	.836		
COMPUTE NM HYD	210.00	-	3	.00410	5.70	.185	.84625	1.500	2.172	PER IMP=	50.00
ADD HYD	210.10	3& 2	4	.14490	77.03	6.730	.87090	1.800	.831		
ROUTE	210.20	4	9	.14490	76.92	6.730	.87090	1.833	.829		
COMPUTE NM HYD	220.00	-	3	.03270	63.83	2.260	1.29582	1.500	3.050	PER IMP=	95.00
ROUTE RESERVOIR	520.00	3	4	.03270	30.26	2.260	1.29581	1.667	1.446	AC-FT=	.527
COMPUTE NM HYD	221.00	-	3	.00430	7.99	.282	1.23127	1.500	2.903	PER IMP=	90.00
ADD HYD	221.10	3& 4	2	.03700	38.21	2.542	1.28830	1.500	1.614		
ROUTE	221.20	2	8	.03700	38.10	2.542	1.28831	1.500	1.609		
COMPUTE NM HYD	213.00	-	3	.01100	21.96	.785	1.33747	1.500	3.120	PER IMP=	100.00
ROUTE	213.10	3	7	.01100	21.89	.785	1.33748	1.500	3.109		
ADD HYD	213.20	8& 7	2	.04800	59.98	3.327	1.29956	1.500	1.953		
ROUTE	213.30	2	7	.04800	59.27	3.327	1.29957	1.533	1.929		
COMPUTE NM HYD	219.00	-	3	.01840	24.46	1.260	1.28437	1.633	2.077	PER IMP=	95.00
ADD HYD	219.10	3& 7	2	.06640	78.26	4.587	1.29534	1.567	1.841		
ROUTE	219.20	2	6	.06640	78.33	4.587	1.29535	1.567	1.843		
COMPUTE NM HYD	211.18	-	3	.02680	38.18	1.637	1.14544	1.567	2.226	PER IMP=	80.00
ROUTE	211.10	3	7	.02680	37.85	1.637	1.14544	1.567	2.207		
ADD HYD	218.00	6& 7	8	.09320	116.18	6.224	1.25223	1.567	1.948		
COMPUTE NM HYD	212.00	-	3	.00580	11.58	.414	1.33747	1.500	3.121	PER IMP=	100.00
ROUTE RESERVOIR	520.00	3	4	.00580	11.47	.414	1.33741	1.500	3.091	AC-FT=	.007
ROUTE	212.10	4	7	.00580	11.37	.414	1.33750	1.533	3.063		
COMPUTE NM HYD	214.00	-	3	.00560	9.44	.312	1.04592	1.500	2.634	PER IMP=	65.00
ADD HYD	214.30	3& 7	2	.01140	20.37	.726	1.19420	1.500	2.792		
ROUTE	214.40	2	7	.01140	20.57	.726	1.19425	1.533	2.819		
COMPUTE NM HYD	214.10	-	3	.00380	7.43	.263	1.29582	1.500	3.054	PER IMP=	95.00
ADD HYD	214.11	3& 7	2	.01520	27.54	.989	1.21959	1.533	2.831		

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		
ADD HYD	214.50	3& 7	2	.10840	141.64	7.213	1.24766	1.567	2.042		
ROUTE	214.60	2	7	.10840	141.80	7.213	1.24766	1.567	2.044		
ADD HYD	214.70	9& 7	2	.25330	193.20	13.943	1.03213	1.600	1.192		
ROUTE	214.80	2	6	.25330	193.28	13.943	1.03213	1.600	1.192		
COMPUTE NM HYD	224.00	-	3	.00790	11.59	.371	.88095	1.500	2.293	PER IMP=	50.00
ROUTE	224.10	3	7	.00790	11.54	.371	.88098	1.500	2.283		
ADD HYD	224.20	3& 6	2	.26120	201.26	14.315	1.02756	1.600	1.204		
COMPUTE NM HYD	214.20	-	3	.01220	13.10	.319	.47021	1.500	1.678	PER IMP=	.00
COMPUTE NM HYD	225.00	-	4	.01050	14.97	.446	.79602	1.500	2.213	PER IMP=	35.00
ADD HYD	225.10	3& 4	5	.02270	27.97	.765	.63165	1.500	1.925		
ROUTE RESERVOIR	525.00	5	4	.02270	18.83	.765	.63165	1.600	1.296	AC-FT=	.116
ADD HYD	214.90	2& 4	5	.28390	220.09	15.079	.99590	1.600	1.211		
ROUTE	214.91	5	9	.28390	220.28	15.079	.99590	1.600	1.212		
COMPUTE NM HYD	216.00	-	3	.01340	7.00	.193	.27007	1.533	.817	PER IMP=	5.00
ROUTE	216.10	3	6	.01340	6.58	.193	.27008	1.567	.768		
COMPUTE NM HYD	217.11	-	3	.00520	9.53	.331	1.19336	1.500	2.864	PER IMP=	34.00
ADD HYD	217.12	3& 6	6	.01860	15.40	.524	.52816	1.533	1.294		
COMPUTE NM HYD	215.00	-	3	.01240	10.52	.270	.40850	1.500	1.326	PER IMP=	5.00
ROUTE	215.10	3	4	.01240	10.65	.270	.40852	1.533	1.343		
COMPUTE NM HYD	217.00	-	3	.02250	40.90	1.405	1.17087	1.500	2.841	PER IMP=	90.00
ADD HYD	217.10	3& 4	2	.03490	50.04	1.675	.89998	1.500	2.240		
ROUTE RESERVOIR	517.00	2	4	.03490	.17	.242	.12994	6.133	.007	AC-FT=	1.612
ADD HYD	217.20	6& 4	2	.05350	15.46	.766	.26839	1.533	.451		
ADD HYD	217.14	2& 9	4	.33740	232.83	15.845	.88054	1.600	1.078		
ROUTE	217.30	4	7	.33740	233.26	15.845	.88053	1.600	1.080		
COMPUTE NM HYD	223.00	-	3	.02770	48.15	1.676	1.13463	1.500	2.716	PER IMP=	80.00
ADD HYD	223.10	3& 7	2	.36510	268.80	17.521	.89980	1.567	1.150		
ROUTE	223.20	2	8	.36510	268.93	17.521	.89980	1.567	1.151		
COMPUTE NM HYD	222.00	-	3	.02150	28.89	1.351	1.17817	1.600	2.099	PER IMP=	85.00
COMPUTE NM HYD	229.00	-	7	.00510	9.47	.335	1.23127	1.500	2.903	PER IMP=	90.00
ADD HYD	229.10	3& 7	2	.02660	35.49	1.686	1.18832	1.567	2.084		
ROUTE	229.20	2	7	.02660	35.35	1.686	1.18834	1.600	2.077		
COMPUTE NM HYD	227.00	-	3	.00870	10.05	.325	.70029	1.500	1.805	PER IMP=	40.00
ADD HYD	227.10	3& 7	2	.03530	43.48	2.011	1.06804	1.567	1.925		
COMPUTE NM HYD	226.00	-	3	.00750	11.30	.386	.96578	1.500	2.353	PER IMP=	65.00
ADD HYD	226.10	3& 2	4	.04280	53.36	2.397	1.05011	1.533	1.948		
ROUTE	226.20	4	7	.04280	53.24	2.397	1.05012	1.567	1.944		
COMPUTE NM HYD	228.00	-	3	.00400	7.15	.251	1.17817	1.500	2.794	PER IMP=	85.00
ADD HYD	228.10	3& 7	2	.04680	59.63	2.648	1.06105	1.533	1.991		
ADD HYD	228.20	2& 8	5	.41190	327.94	20.169	.91812	1.567	1.244		
ROUTE	100.10	5	7	.41190	328.37	20.169	.91811	1.567	1.246		
COMPUTE NM HYD	301.00	-	3	.01970	37.04	1.304	1.24145	1.500	2.938	PER IMP=	90.00
COMPUTE NM HYD	302.00	-	6	.00540	10.55	.373	1.29582	1.500	3.052	PER IMP=	95.00
ADD HYD	302.10	3& 6	2	.02510	47.59	1.678	1.25312	1.500	2.963		
COMPUTE NM HYD	303.00	-	3	.00310	4.33	.148	.89555	1.500	2.181	PER IMP=	60.00
ROUTE	303.10	3	4	.00310	3.71	.148	.89562	1.567	1.869		
ADD HYD	303.15	2& 4	5	.02820	50.80	1.826	1.21381	1.500	2.814		
ADD HYD	303.16	5& 7	5	.44010	370.31	21.995	.93705	1.567	1.315		
ROUTE	303.20	5	4	.44010	372.23	21.994	.93704	1.567	1.322		
COMPUTE NM HYD	307.00	-	3	.02350	40.32	1.410	1.12508	1.500	2.681	PER IMP=	80.00
COMPUTE NM HYD	308.00	-	2	.01910	32.77	1.146	1.12508	1.500	2.681	PER IMP=	80.00
ADD HYD	308.10	3& 2	7	.04260	73.10	2.556	1.12506	1.500	2.681		
ADD HYD	308.20	4& 7	2	.48270	431.29	24.550	.95363	1.567	1.396		
ROUTE	308.30	2	4	.48270	434.75	24.550	.95361	1.567	1.407		
COMPUTE NM HYD	310.00	-	3	.01070	18.88	.654	1.14543	1.500	2.757	PER IMP=	80.00
ADD HYD	310.10	3& 4	7	.49340	450.02	25.203	.95777	1.567	1.425		
ROUTE	308.30	7	8	.49340	451.94	25.203	.95775	1.567	1.431		
COMPUTE NM HYD	304.00	-	3	.01320	18.92	.643	.91269	1.500	2.240	PER IMP=	60.00
ROUTE RESERVOIR	504.00	3	6	.01320	1.78	.642	.91264	2.166	.210	AC-FT=	.468
ROUTE	304.10	6	4	.01320	1.76	.642	.91251	2.333	.209		
COMPUTE NM HYD	306.00	-	3	.01630	31.40	1.117	1.28437	1.500	3.010	PER IMP=	95.00
ROUTE RESERVOIR	505.00	3	6	.01630	11.16	1.117	1.28435	1.800	1.069	AC-FT=	.494
COMPUTE NM HYD	305.00	-	7	.01590	27.54	.980	1.15536	1.500	2.706	PER IMP=	85.00
ADD HYD	305.10	6& 7	2	.03220	33.82	2.096	1.22062	1.500	1.641		
ADD HYD	305.20	4& 2	2	.04540	34.34	2.238	1.13103	1.500	1.132		

COMPUTE NM HYD	309.00	-	3	.00730	9.77	.340	.37201	1.500	2.090	PER IMP=	60.00
ADD HYD	309.10	3& 4	2	.05270	44.08	3.078	1.09514	1.533	1.307		
ROUTE	309.20	2	9	.05270	42.39	3.078	1.09515	1.567	1.257		
COMPUTE NM HYD	315.00	-	3	.01050	9.23	.314	.56141	1.500	1.374	PER IMP=	35.00
ROUTE	315.10	3	4	.01050	7.33	.314	.56144	1.567	1.090		
COMPUTE NM HYD	314.00	-	3	.01770	4.25	.117	.12345	1.533	.375	PER IMP=	.00
ADD HYD	314.10	3& 4	2	.02820	11.29	.431	.28651	1.567	.626		
ROUTE	314.20	2	4	.02820	11.33	.431	.28653	1.567	.628		
COMPUTE NM HYD	313.00	-	3	.00990	3.60	.094	.17766	1.533	.568	PER IMP=	.00
ADD HYD	313.01	3& 4	2	.03810	14.64	.525	.25823	1.567	.600		
ADD HYD	313.02	2& 9	2	.09080	57.03	3.603	.74397	1.567	.981		
ROUTE	313.03	2	10	.09080	56.56	3.603	.74397	1.567	.973		
COMPUTE NM HYD	314.10	-	3	.02440	9.98	.319	.24485	1.500	.639	PER IMP=	10.00
COMPUTE NM HYD	313.10	-	7	.00990	12.72	.365	.69847	1.500	2.028	PER IMP=	25.00
ADD HYD	314.20	3& 7	2	.03420	22.70	.684	.37482	1.500	1.037		
ADD HYD	313.12	10& 2	6	.12500	76.10	4.286	.64297	1.567	.951		
ROUTE	313.13	6	7	.12500	75.76	4.286	.64297	1.567	.947		
COMPUTE NM HYD	312.00	-	3	.02030	12.67	.391	.36088	1.500	.975	PER IMP=	15.00
ADD HYD	312.10	3& 7	2	.14530	86.79	4.677	.60356	1.567	.933		
ADD HYD	312.20	2& 8	7	.63870	538.73	29.880	.87718	1.567	1.318		
COMPUTE NM HYD	311.00	-	3	.00470	6.60	.203	.81004	1.500	2.195	PER IMP=	40.00
ADD HYD	311.10	3& 7	2	.64340	544.23	30.083	.87668	1.567	1.322		
ROUTE	311.20	2	4	.64340	544.47	30.083	.87667	1.567	1.322		
ROUTE	311.20	4	6	.64340	544.37	30.083	.87667	1.567	1.322		
COMPUTE NM HYD	316.00	-	3	.00900	10.78	.345	.71912	1.500	1.871	PER IMP=	40.00
COMPUTE NM HYD	317.00	-	5	.02860	15.23	.530	.34734	1.567	.832	PER IMP=	10.00
ADD HYD	317.10	3& 5	2	.03760	25.47	.875	.43631	1.533	1.058		
ADD HYD	317.20	2& 6	3	.68100	568.63	30.957	.85235	1.567	1.305		

AHYMO SUMMARY TABLE (AHYMO491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/16/1994
INPUT FILE = n17ex100.dat USER NO.= MOLZCONM.101

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	2.350
COMPUTE NM HYD	201.00	-	3	.01290	9.67	.998	1.45038	2.166	1.171	PER IMP=	36.00
COMPUTE NM HYD	205.00	-	7	.01380	33.78	1.133	1.53935	1.500	3.824	PER IMP=	45.00
ADD HYD	205.10	3& 7	2	.02670	34.26	2.131	1.49633	1.500	2.005		
ROUTE RESERVOIR	505.00	2	4	.02670	13.29	2.131	1.49633	2.133	.778	AC-FT=	.516
ROUTE	205.20	4	7	.02670	13.28	2.131	1.49635	2.166	.777		
COMPUTE NM HYD	206.00	-	3	.01380	35.15	1.206	1.63837	1.500	3.980	PER IMP=	55.00
ROUTE RESERVOIR	506.00	3	4	.01380	12.62	1.206	1.63835	1.766	1.429	AC-FT=	.445
ADD HYD	206.10	4& 7	2	.04050	25.67	3.337	1.54472	1.900	.990		
ROUTE	206.20	2	7	.04050	25.66	3.337	1.54473	1.933	.990		
COMPUTE NM HYD	204.00	-	3	.02170	52.67	1.759	1.51957	1.500	3.793	PER IMP=	43.00
ROUTE RESERVOIR	504.00	3	4	.02170	13.58	1.759	1.51956	1.900	.978	AC-FT=	.933
ROUTE	504.10	4	5	.02170	13.58	1.759	1.51957	1.900	.978		
ADD HYD	207.10	5& 7	2	.06220	39.24	5.095	1.53594	1.900	.986		
COMPUTE NM HYD	203.00	-	7	.02100	48.83	1.713	1.52946	1.500	3.633	PER IMP=	44.00
ROUTE RESERVOIR	503.00	7	6	.02100	10.89	1.713	1.52944	1.966	.810	AC-FT=	.938
ROUTE	203.20	6	7	.02100	10.89	1.713	1.52945	2.000	.810		
COMPUTE NM HYD	208.00	-	3	.02250	44.45	1.907	1.58883	1.567	3.087	PER IMP=	50.00
ROUTE RESERVOIR	508.00	3	4	.02250	20.02	1.907	1.58881	1.866	1.391	AC-FT=	.696
ADD HYD	208.10	4& 7	5	.04350	30.82	3.620	1.56013	1.866	1.107		
ROUTE	208.20	5	7	.04350	30.67	3.620	1.56014	1.933	1.102		
ADD HYD	207.20	2& 7	8	.10570	69.90	8.715	1.54590	1.933	1.033		
COMPUTE NM HYD	207.00	-	3	.02190	47.35	1.856	1.58883	1.533	3.378	PER IMP=	50.00
ROUTE RESERVOIR	507.00	3	4	.02190	22.19	1.856	1.58881	1.766	1.583	AC-FT=	.616
ADD HYD	207.30	4& 8	2	.12760	91.50	10.570	1.55326	1.866	1.120		
ROUTE	207.40	2	7	.12760	91.45	10.570	1.55326	1.866	1.120		
COMPUTE NM HYD	209.00	-	3	.01320	16.01	1.119	1.58883	1.833	1.895	PER IMP=	50.00
ROUTE RESERVOIR	509.00	3	4	.01320	15.63	1.118	1.58878	1.900	1.850	AC-FT=	.067
ADD HYD	209.10	4& 7	2	.14080	107.05	11.689	1.55659	1.866	1.188		
COMPUTE NM HYD	210.00	-	3	.00410	9.75	.330	1.50919	1.500	3.714	PER IMP=	50.00
ADD HYD	210.10	3& 2	4	.14490	109.64	12.019	1.55525	1.866	1.182		
ROUTE	210.20	4	9	.14490	109.53	12.019	1.55525	1.900	1.181		
COMPUTE NM HYD	220.00	-	3	.03270	97.81	3.603	2.06599	1.500	4.674	PER IMP=	95.00
ROUTE RESERVOIR	520.00	3	4	.03270	40.84	3.603	2.06598	1.733	1.951	AC-FT=	73.907
COMPUTE NM HYD	221.00	-	3	.00430	12.42	.454	1.98165	1.500	4.514	PER IMP=	90.00
ADD HYD	221.10	3& 4	2	.03700	46.43	4.058	2.05617	1.667	1.961		
ROUTE	221.20	2	8	.03700	46.41	4.058	2.05619	1.667	1.960		
COMPUTE NM HYD	213.00	-	3	.01100	33.45	1.241	2.11537	1.500	4.752	PER IMP=	100.00
ROUTE	213.10	3	7	.01100	33.33	1.241	2.11538	1.500	4.735		
ADD HYD	213.20	8& 7	2	.04800	75.86	5.299	2.06973	1.500	2.469		
ROUTE	213.30	2	7	.04800	74.54	5.299	2.06974	1.533	2.427		
COMPUTE NM HYD	219.00	-	3	.01840	37.87	2.010	2.04851	1.633	3.216	PER IMP=	95.00
ADD HYD	219.10	3& 7	2	.06640	105.46	7.309	2.06384	1.600	2.482		
ROUTE	219.20	2	6	.06640	105.52	7.309	2.06385	1.600	2.483		
COMPUTE NM HYD	211.18	-	3	.02680	60.46	2.687	1.87962	1.567	3.525	PER IMP=	80.00
ROUTE	211.10	3	7	.02680	59.60	2.687	1.87962	1.600	3.475		
ADD HYD	218.00	6& 7	8	.09320	165.12	9.995	2.01086	1.600	2.768		
COMPUTE NM HYD	212.00	-	3	.00580	17.64	.654	2.11537	1.500	4.753	PER IMP=	100.00
ROUTE RESERVOIR	520.00	3	4	.00580	17.48	.654	2.11532	1.500	4.709	AC-FT=	.011
ROUTE	212.10	4	7	.00580	17.32	.654	2.11540	1.533	4.665		
COMPUTE NM HYD	214.00	-	3	.00560	15.10	.529	1.76975	1.500	4.213	PER IMP=	65.00
ADD HYD	214.30	3& 7	2	.01140	31.85	1.183	1.94555	1.500	4.366		
ROUTE	214.40	2	7	.01140	31.92	1.183	1.94559	1.533	4.376		
COMPUTE NM HYD	214.10	-	3	.00380	11.38	.419	2.06599	1.500	4.681	PER IMP=	95.00
ADD HYD	214.11	3& 7	2	.01520	42.61	1.602	1.97564	1.533	4.380		
ROUTE	214.12	2	7	.01520	42.84	1.602	1.97568	1.533	4.403		

ROUTE	214.60	2	7	.10840	204.72	11.597	2.00593	1.567	2.951		
ADD HYD	214.70	9& 7	2	.25330	281.40	23.616	1.74812	1.600	1.736		
ROUTE	214.80	2	6	.25330	281.64	23.616	1.74812	1.600	1.737		
COMPUTE NM HYD	224.00	-	3	.00790	19.42	.658	1.56192	1.500[1]	3.840	PER IMP=	50.00
ROUTE	224.10	3	7	.00790	19.38	.658	1.56195	1.500[1]	3.833		
ADD HYD	224.20	3& 6	2	.26120	295.18	24.274	1.74248	1.600[1]	1.766		
COMPUTE NM HYD	214.20	-	3	.01220	24.63	.720	1.10680	1.500	3.154	PER IMP=	.00
COMPUTE NM HYD	225.00	-	4	.01050	25.17	.825	1.47351	1.500	3.746	PER IMP=	35.00
ADD HYD	225.10	3& 4	5	.02270	49.80	1.545	1.27641	1.500	3.428		
ROUTE RESERVOIR	525.00	5	4	.02270	37.45	1.545	1.27641	1.600	2.578	AC-FT=	.650
ADD HYD	214.90	2& 4	5	.28390	332.63	25.819	1.70522	1.600	1.831		
ROUTE	214.91	5	9	.28390	332.78	25.819	1.70522	1.600	1.832		
COMPUTE NM HYD	215.00	-	3	.01340	18.49	.535	.74921	1.500	2.156	PER IMP=	5.00
ROUTE	216.10	3	6	.01340	18.72	.535	.74923	1.533	2.183		
COMPUTE NM HYD	217.11	-	3	.00520	14.88	.538	1.94129	1.500	4.470	PER IMP=	84.00
ADD HYD	217.12	3& 6	6	.01860	32.73	1.074	1.08245	1.533	2.749		
COMPUTE NM HYD	215.00	-	3	.01240	22.05	.642	.97029	1.500	2.779	PER IMP=	5.00
ROUTE	215.10	3	4	.01240	22.07	.642	.97030	1.533	2.782		
COMPUTE NM HYD	217.00	-	3	.02250	63.96	2.301	1.91787	1.500	4.442	PER IMP=	80.00
ADD HYD	217.10	3& 4	2	.03490	85.39	2.943	1.58118	1.500	3.823		
ROUTE RESERVOIR	517.00	2	4	.03490	26.94	1.236	.66430	1.833	1.206	AC-FT=	2.006
ADD HYD	217.20	6& 4	2	.05350	37.22	2.310	.80967	1.800	1.087		
ADD HYD	217.14	2& 9	4	.33740	358.61	28.130	1.56322	1.600	1.661		
ROUTE	217.30	4	7	.33740	359.17	28.129	1.56319	1.600	1.663		
COMPUTE NM HYD	223.00	-	3	.02770	76.41	2.753	1.86326	1.500	4.310	PER IMP=	80.00
ADD HYD	223.10	3& 7	2	.36510	418.23	30.882	1.58596	1.567	1.790		
ROUTE	223.20	2	8	.36510	418.38	30.882	1.58596	1.567	1.791		
COMPUTE NM HYD	222.00	-	3	.02150	45.55	2.196	1.91479	1.600	3.310	PER IMP=	85.00
COMPUTE NM HYD	229.00	-	7	.00510	14.73	.539	1.98165	1.500	4.513	PER IMP=	90.00
ADD HYD	229.10	3& 7	2	.02660	55.91	2.735	1.92758	1.567	3.284		
ROUTE	229.20	2	7	.02660	55.74	2.735	1.92760	1.600	3.274		
COMPUTE NM HYD	227.00	-	3	.00870	18.38	.609	1.31307	1.500	3.300	PER IMP=	40.00
ADD HYD	227.10	3& 7	2	.03530	70.75	3.344	1.77612	1.567	3.132		
COMPUTE NM HYD	226.00	-	3	.00750	18.75	.659	1.64736	1.500	3.906	PER IMP=	65.00
ADD HYD	226.10	3& 2	4	.04280	97.14	4.003	1.75355	1.533	3.181		
ROUTE	226.20	4	7	.04280	86.56	4.003	1.75356	1.567	3.160		
COMPUTE NM HYD	228.00	-	3	.00400	11.25	.408	1.91479	1.500	4.394	PER IMP=	85.00
ADD HYD	228.10	3& 7	2	.04680	96.22	4.411	1.76733	1.533	3.212		
ADD HYD	228.20	2& 8	5	.41190	514.03	35.293	1.60656	1.567	1.950		
ROUTE	100.10	5	7	.41190	514.53	35.293	1.60655	1.567	1.952		
COMPUTE NM HYD	301.00	-	3	.01970	57.39	2.099	1.99749	1.500	4.552	PER IMP=	90.00
COMPUTE NM HYD	302.00	-	6	.00540	16.17	.595	2.06599	1.500	4.678	PER IMP=	95.00
ADD HYD	302.10	3& 6	2	.02510	73.55	2.694	2.01221	1.500	4.579		
COMPUTE NM HYD	303.00	-	3	.00310	7.35	.257	1.55256	1.500	3.703	PER IMP=	60.00
ROUTE	303.10	3	4	.00310	6.49	.257	1.55264	1.533	3.270		
ADD HYD	303.15	2& 4	5	.02820	79.33	2.950	1.96167	1.500	4.396		
ADD HYD	303.16	5& 7	5	.44010	580.19	38.243	1.62930	1.567	2.060		
ROUTE	303.20	5	4	.44010	582.44	38.242	1.62928	1.567	2.068		
COMPUTE NM HYD	307.00	-	3	.02350	64.18	2.316	1.84793	1.500	4.267	PER IMP=	80.00
COMPUTE NM HYD	308.00	-	2	.01910	52.16	1.882	1.84793	1.500	4.267	PER IMP=	80.00
ADD HYD	308.10	3& 2	7	.04260	116.34	4.198	1.84792	1.500	4.267		
ADD HYD	308.20	4& 7	2	.48270	676.84	42.441	1.64857	1.567	2.191		
ROUTE	308.30	2	4	.48270	681.22	42.440	1.64855	1.567	2.205		
COMPUTE NM HYD	310.00	-	3	.01070	29.81	1.073	1.87962	1.500	4.353	PER IMP=	80.00
ADD HYD	310.10	3& 4	7	.49340	705.38	43.513	1.65356	1.567	2.234		
ROUTE	308.30	7	8	.49340	708.08	43.512	1.65354	1.567	2.242		
COMPUTE NM HYD	304.00	-	3	.01320	31.90	1.113	1.58050	1.500	3.776	PER IMP=	60.00
ROUTE RESERVOIR	504.00	3	6	.01320	2.77	1.113	1.58038	2.166	.328	AC-FT=	.860
ROUTE	304.10	6	4	.01320	2.74	1.113	1.58033	2.333	.325		
COMPUTE NM HYD	306.00	-	3	.01630	48.31	1.781	2.04851	1.500	4.631	PER IMP=	95.00
ROUTE RESERVOIR	505.00	3	6	.01630	15.97	1.781	2.04849	1.833	1.531	AC-FT=	.798
COMPUTE NM HYD	305.00	-	7	.01590	43.50	1.592	1.87774	1.500	4.275	PER IMP=	85.00
ADD HYD	305.10	6& 7	2	.03220	53.58	3.373	1.96414	1.500	2.600		
ADD HYD	305.20	4& 2	2	.04540	54.36	4.486	1.85254	1.500	1.871		
ROUTE	305.30	2	4	.04540	55.00	4.486	1.85255	1.533	1.893		

ADD HYD	309.10	3& 4	2	.05270	70.93	5.075	1.80580	1.533	2.103	
ROUTE	309.20	2	9	.05270	69.92	5.076	1.80581	1.533	2.073	
COMPUTE NM HYD	315.00	-	3	.01050	18.51	.620	1.10746	1.500	2.755 PER IMP=	35.00
ROUTE	315.10	3	4	.01050	16.36	.620	1.10748	1.567	2.434	
COMPUTE NM HYD	314.00	-	3	.01770	17.94	.501	.53121	1.533	1.584 PER IMP=	.00
ADD HYD	314.10	3& 4	2	.02820	33.97	1.122	.74576	1.533	1.882	
ROUTE	314.20	2	4	.02820	33.83	1.122	.74578	1.567	1.874	
COMPUTE NM HYD	313.00	-	3	.00990	11.75	.328	.62056	1.533	1.855 PER IMP=	.00
ADD HYD	313.01	3& 4	2	.03810	45.10	1.449	.71323	1.533	1.849	
ADD HYD	313.02	2& 9	2	.09080	115.02	6.525	1.34735	1.533	1.979	
ROUTE	313.03	2	10	.09080	115.65	6.525	1.34736	1.567	1.990	
COMPUTE NM HYD	314.10	-	3	.02440	29.47	.897	.68963	1.500	1.887 PER IMP=	10.00
COMPUTE NM HYD	313.10	-	7	.00980	22.22	.708	1.35383	1.500	3.543 PER IMP=	25.00
ADD HYD	314.20	3& 7	2	.03420	51.69	1.605	.87994	1.500	2.362	
ADD HYD	313.12	10& 2	6	.12500	161.56	8.130	1.21947	1.533	2.020	
ROUTE	313.13	6	7	.12500	162.20	8.130	1.21947	1.567	2.028	
COMPUTE NM HYD	312.00	-	3	.02030	30.24	.930	.85908	1.500	2.328 PER IMP=	15.00
ADD HYD	312.10	3& 7	2	.14530	188.97	9.060	1.16911	1.567	2.032	
ADD HYD	312.20	2& 8	7	.63870	897.05	52.572	1.54334	1.567	2.195	
COMPUTE NM HYD	311.00	-	3	.00470	11.23	.371	1.48196	1.500	3.732 PER IMP=	40.00
ADD HYD	311.10	3& 7	2	.64340	906.41	52.944	1.54289	1.567	2.201	
ROUTE	311.20	2	4	.64340	908.84	52.943	1.54287	1.567	2.207	
ROUTE	311.20	4	6	.64340	908.71	52.943	1.54287	1.567	2.207	
COMPUTE NM HYD	316.00	-	3	.00900	19.47	.645	1.34324	1.500	3.380 PER IMP=	40.00
COMPUTE NM HYD	317.00	-	5	.02860	36.40	1.305	.85576	1.567	1.988 PER IMP=	10.00
ADD HYD	317.10	3& 5	2	.03760	54.74	1.950	.97243	1.533	2.275	
ADD HYD	317.20	2& 6	3	.68100	961.50	54.893	1.51137	1.567	2.206	

AHYMO SUMMARY TABLE (AHYMO491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/16/1994

INPUT FILE = n17fut10.dat USER NO.= MOLZDONM.I01

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	1.570
COMPUTE NM HYD	101.00	-	3	.02840	32.42	1.382	.91269	1.367	1.784	PER IMP=	60.00
ROUTE RESERVOIR	505.00	3	4	.02840	25.00	1.382	.91267	1.667	1.375	AC-FT=	.273
ROUTE	102.10	4	7	.02840	23.17	1.382	.91262	1.766	1.275		
COMPUTE NM HYD	102.00	-	3	.02820	22.68	.684	.58806	1.600	1.256	PER IMP=	15.00
ROUTE RESERVOIR	505.00	3	4	.02820	15.70	.884	.58805	1.766	.870	AC-FT=	.246
ADD HYD	102.20	4& 7	2	.05660	38.87	2.267	.75086	1.766	1.073		
ROUTE	103.10	2	7	.05660	38.77	2.267	.75087	1.766	1.070		
COMPUTE NM HYD	103.00	-	3	.00560	11.18	.399	1.33747	1.500	3.121	PER IMP=	100.00
ROUTE RESERVOIR	505.00	3	4	.00560	4.94	.399	1.33741	1.700	1.379	AC-FT=	.152
ADD HYD	103.20	4& 7	2	.06220	43.64	2.666	.80364	1.766	1.096		
ROUTE	104.10	2	7	.06220	43.26	2.666	.80365	1.800	1.087		
COMPUTE NM HYD	104.00	-	3	.00880	17.57	.628	1.33747	1.500	3.120	PER IMP=	100.00
ROUTE RESERVOIR	505.00	3	4	.00880	9.59	.628	1.33743	1.633	1.704	AC-FT=	.197
ADD HYD	104.20	4& 7	2	.07100	51.75	3.294	.86977	1.800	1.139		
ROUTE	104.10	2	10	.07100	51.66	3.294	.86978	1.800	1.141		
COMPUTE NM HYD	201.00	-	3	.01290	5.75	.572	.83205	2.166	.697	PER IMP=	42.00
COMPUTE NM HYD	205.00	-	7	.01380	27.55	.984	1.33747	1.500	3.120	PER IMP=	100.00
ADD HYD	205.10	3& 7	2	.02670	27.77	1.557	1.09324	1.500	1.625		
ROUTE RESERVOIR	505.00	2	4	.02670	13.36	1.557	1.09324	1.733	.782	AC-FT=	.365
ROUTE	205.20	4	7	.02670	13.35	1.557	1.09319	1.766	.781		
ADD HYD	205.10	10& 7	2	.09770	65.18	4.850	.93082	1.800	1.042		
COMPUTE NM HYD	206.00	-	3	.01380	27.55	.984	1.33747	1.500	3.120	PER IMP=	100.00
ROUTE RESERVOIR	505.00	3	4	.01380	12.84	.984	1.33744	1.667	1.454	AC-FT=	.354
ADD HYD	206.10	2& 4	2	.11150	77.45	5.834	.98113	1.800	1.085		
ROUTE	206.20	2	7	.11150	77.04	5.834	.98113	1.800	1.080		
COMPUTE NM HYD	204.00	-	3	.02170	36.74	1.220	1.05425	1.500	2.645	PER IMP=	66.00
ROUTE RESERVOIR	504.00	3	4	.02170	19.52	1.220	1.05424	1.633	1.405	AC-FT=	.402
ROUTE	504.10	4	5	.02170	19.55	1.220	1.05417	1.667	1.408		
ADD HYD	207.10	5& 7	2	.13320	94.49	7.054	.99303	1.800	1.108		
COMPUTE NM HYD	203.00	-	7	.02100	41.93	1.498	1.33747	1.500	3.119	PER IMP=	100.00
ROUTE RESERVOIR	503.00	7	6	.02100	24.21	1.498	1.33745	1.633	1.801	AC-FT=	.437
ROUTE	203.20	6	7	.02100	24.04	1.498	1.33738	1.633	1.769		
COMPUTE NM HYD	208.00	-	3	.02250	44.92	1.605	1.33747	1.500	3.119	PER IMP=	100.00
ROUTE RESERVOIR	508.00	3	4	.02250	26.74	1.605	1.33745	1.633	1.857	AC-FT=	.459
ADD HYD	208.10	4& 7	5	.04350	50.79	3.103	1.33736	1.633	1.824		
ROUTE	208.20	5	7	.04350	49.34	3.103	1.33737	1.700	1.772		
ADD HYD	207.20	2& 7	8	.17670	141.10	10.157	1.07760	1.766	1.248		
COMPUTE NM HYD	207.00	-	3	.02190	43.72	1.562	1.33747	1.500	3.119	PER IMP=	100.00
ROUTE RESERVOIR	507.00	3	4	.02190	25.73	1.562	1.33745	1.633	1.835	AC-FT=	.448
ADD HYD	207.30	4& 8	2	.19860	164.00	11.719	1.10642	1.733	1.290		
ROUTE	207.40	2	7	.19860	164.00	11.719	1.10642	1.766	1.290		
COMPUTE NM HYD	209.00	-	3	.04170	73.75	2.975	1.33747	1.533	2.765	PER IMP=	100.00
ROUTE RESERVOIR	509.00	7	4	.04170	50.68	2.975	1.33746	1.667	1.969	AC-FT=	.693
ADD HYD	209.10	4& 7	2	.24030	211.97	14.694	1.14650	1.733	1.378		
ROUTE	210.20	2	9	.24030	212.02	14.694	1.14650	1.733	1.379		
COMPUTE NM HYD	216.00	-	3	.01340	13.75	.956	1.33746	1.500	3.120	PER IMP=	100.00
ROUTE RESERVOIR	509.00	3	4	.01340	13.24	.956	1.33745	1.667	1.546	AC-FT=	.326
ROUTE	210.30	4	5	.01340	13.09	.956	1.33734	1.700	1.525		
COMPUTE NM HYD	215.00	-	3	.01240	24.76	1.385	1.33747	1.500	3.120	PER IMP=	100.00
ROUTE RESERVOIR	509.00	3	4	.01240	11.77	.884	1.33744	1.667	1.483	AC-FT=	.313
ADD HYD	201.10	5& 4	2	.02580	24.82	1.840	1.33729	1.700	1.503		
ADD HYD	201.10	9& 2	10	.26610	236.69	16.834	1.16500	1.733	1.390		
COMPUTE NM HYD	220.00	-	3	.03270	63.83	3.260	1.29582	1.500	3.030	PER IMP=	95.00
ROUTE RESERVOIR	520.00	3	4	.03270	61.53	3.260	1.29581	1.533	2.940	AC-FT=	.123
ROUTE	210.20	4	5	.03270	55.71	2.760	1.29576	1.567	2.447		

ROUTE	216.20	11	9	.29660	272.35	18.794	1.17931	1.700	1.424		
COMPUTE NM HYD	221.00	-	3	.00430	7.99	.282	1.23127	1.500	2.903	PER IMP=	90.00
ROUTE	221.20	3	8	.00430	7.81	.282	1.23132	1.533	2.829		
ROUTE	213.30	8	7	.00430	7.45	.282	1.23132	1.533	2.707		
COMPUTE NM HYD	219.00	-	3	.01240	24.46	1.260	1.28437	1.633	2.077	PER IMP=	95.00
ADD HYD	219.10	3& 7	2	.02270	30.19	1.543	1.27429	1.633	2.078		
ROUTE	219.20	2	6	.02270	30.27	1.543	1.27430	1.633	2.084		
COMPUTE NM HYD	211.18	-	3	.01920	29.03	1.261	1.23127	1.567	2.362	PER IMP=	90.00
ROUTE	211.10	3	7	.01920	28.80	1.261	1.23127	1.567	2.344		
ADD HYD	218.00	6& 7	8	.04190	58.75	2.304	1.25456	1.600	2.191		
COMPUTE NM HYD	214.00	-	3	.00560	9.44	.312	1.04592	1.500	2.634	PER IMP=	85.00
ROUTE	214.40	3	7	.00560	9.30	.312	1.04596	1.500	2.595		
COMPUTE NM HYD	214.10	-	3	.00380	7.43	.263	1.29582	1.500	3.054	PER IMP=	95.00
ADD HYD	214.11	2& 7	2	.00940	16.73	.575	1.14688	1.500	2.780		
ROUTE	214.12	2	7	.00940	16.57	.575	1.14693	1.500	2.755		
ADD HYD	214.50	8& 7	2	.05130	71.59	3.378	1.23483	1.567	2.180		
ROUTE	214.50	2	7	.05130	71.52	3.379	1.23484	1.567	2.178		
ADD HYD	214.70	9& 7	2	.35010	331.45	22.172	1.18745	1.667	1.479		
ROUTE	214.80	2	6	.35010	331.40	22.172	1.18745	1.667	1.479		
COMPUTE NM HYD	224.00	-	3	.00790	11.59	.371	.88095	1.500	2.293	PER IMP=	50.00
ROUTE	224.10	3	7	.00790	11.54	.371	.98098	1.500	2.283		
ADD HYD	224.20	7& 6	2	.35800	337.22	22.543	1.18068	1.667	1.472		
COMPUTE NM HYD	214.20	-	3	.01220	13.10	.319	.49021	1.500	1.678	PER IMP=	.00
COMPUTE NM HYD	225.00	-	4	.01050	14.87	.446	.79602	1.500	2.213	PER IMP=	35.00
ADD HYD	225.10	3& 4	5	.02270	27.97	.765	.63165	1.500	1.925		
ROUTE RESERVOIR	525.00	5	4	.02270	23.03	.765	.63165	1.567	1.585	AC-FT=	.069
ADD HYD	214.90	2& 4	5	.38070	357.49	23.308	1.14794	1.667	1.467		
ROUTE	214.91	5	9	.38070	357.58	23.308	1.14794	1.667	1.468		
COMPUTE NM HYD	217.00	-	3	.02250	40.90	1.405	1.17087	1.500	2.841	PER IMP=	80.00
ROUTE RESERVOIR	517.00	3	4	.02250	.14	.202	.16818	6.133	.010	AC-FT=	1.352
COMPUTE NM HYD	217.11	-	2	.00520	9.53	.331	1.19336	1.500	2.864	PER IMP=	84.00
ADD HYD	217.20	2& 4	1	.02770	9.57	.533	.36062	1.500	.540		
ADD HYD	217.14	9& 1	4	.40840	362.32	23.841	1.09454	1.667	1.386		
ROUTE	217.30	4	7	.40840	362.44	23.840	1.09453	1.667	1.387		
COMPUTE NM HYD	223.00	-	3	.02770	48.15	1.676	1.13463	1.500	2.716	PER IMP=	80.00
ADD HYD	223.10	3& 7	2	.43610	386.00	25.517	1.09708	1.667	1.383		
ROUTE	223.20	2	8	.43610	386.10	25.517	1.09708	1.667	1.383		
COMPUTE NM HYD	222.00	-	3	.02150	28.89	1.351	1.17817	1.600	2.099	PER IMP=	85.00
COMPUTE NM HYD	229.00	-	7	.00510	9.47	.335	1.23127	1.500	2.903	PER IMP=	90.00
ADD HYD	229.10	3& 7	2	.02660	35.49	1.686	1.18832	1.567	2.084		
ROUTE	229.20	2	7	.02660	35.35	1.686	1.18834	1.600	2.077		
COMPUTE NM HYD	227.00	-	3	.00870	10.05	.325	.70029	1.500	1.805	PER IMP=	40.00
ADD HYD	227.10	3& 7	2	.03530	43.48	2.011	1.06804	1.567	1.925		
COMPUTE NM HYD	226.00	-	3	.00750	11.30	.386	.96578	1.500	2.353	PER IMP=	65.00
ADD HYD	226.10	3& 2	4	.04280	53.36	2.397	1.05011	1.533	1.948		
ROUTE	226.20	4	7	.04280	53.24	2.397	1.05012	1.567	1.944		
COMPUTE NM HYD	228.00	-	3	.00400	7.15	.251	1.17817	1.500	2.794	PER IMP=	85.00
ADD HYD	229.10	3& 7	2	.04680	59.63	2.648	1.06105	1.533	1.991		
ADD HYD	228.20	2& 8	5	.48290	435.53	28.165	1.09358	1.633	1.409		
ROUTE	100.10	5	7	.48290	435.29	28.165	1.09357	1.633	1.408		
COMPUTE NM HYD	301.00	-	3	.01970	37.04	1.304	1.24145	1.500	2.938	PER IMP=	90.00
COMPUTE NM HYD	302.00	-	6	.00540	10.55	.373	1.29582	1.500	3.052	PER IMP=	95.00
ADD HYD	302.10	3& 6	2	.02510	47.59	1.678	1.25312	1.500	2.963		
COMPUTE NM HYD	303.00	-	3	.00310	4.33	.148	.89555	1.500	2.181	PER IMP=	60.00
ROUTE	303.10	3	4	.00310	3.71	.148	.89562	1.567	1.869		
ADD HYD	303.15	2& 4	1	.02820	50.80	1.826	1.21381	1.500	2.814		
ADD HYD	303.16	1& 7	5	.51110	465.27	29.990	1.10021	1.633	1.422		
ROUTE	303.20	5	4	.51110	465.17	29.990	1.10019	1.633	1.422		
COMPUTE NM HYD	307.00	-	3	.02350	40.32	1.410	1.12508	1.500	2.681	PER IMP=	60.00
COMPUTE NM HYD	308.00	-	2	.01910	32.77	1.146	1.12508	1.500	2.681	PER IMP=	90.00
ADD HYD	308.10	3& 2	7	.04260	73.10	2.556	1.12506	1.500	2.681		
ADD HYD	308.20	4& 7	2	.55370	506.56	32.546	1.10211	1.633	1.429		
ROUTE	306.30	2	4	.55370	507.57	32.545	1.10209	1.633	1.432		
COMPUTE NM HYD	310.00	-	3	.01070	18.88	.654	1.14543	1.500	2.757	PER IMP=	80.00
ADD HYD	310.10	3& 4	7	.56440	512.27	33.199	1.10291	1.633	1.435		

COMPUTE NM HYD	304.00	-	3	.01320	18.92	.643	.91269	1.500	2.240	PER IMP=	60.00
ROUTE RESERVOIR	304.00	3	6	.01320	1.78	.642	.91264	2.166	.210	AC-FT=	.483
ROUTE	304.10	6	4	.01320	1.76	.642	.91251	2.333	.209		
COMPUTE NM HYD	306.00	-	3	.01630	31.40	1.117	1.28437	1.500	3.010	PER IMP=	95.00
ROUTE RESERVOIR	305.00	3	6	.01630	11.16	1.117	1.28435	1.800	1.069	AC-FT=	.494
COMPUTE NM HYD	305.00	-	7	.01590	27.54	.980	1.15536	1.500	2.706	PER IMP=	85.00
ADD HYD	305.10	6& 7	2	.03220	33.82	2.096	1.22062	1.500	1.541		
ADD HYD	305.20	4& 2	2	.04540	34.36	2.739	1.13103	1.500	1.162		
ROUTE	305.30	2	4	.04540	34.84	2.739	1.13104	1.533	1.199		
COMPUTE NM HYD	309.00	-	3	.00730	9.77	.340	.87201	1.500	2.090	PER IMP=	60.00
ADD HYD	309.10	3& 4	2	.05270	44.08	3.079	1.09514	1.533	1.307		
ROUTE	309.20	2	9	.05270	42.39	3.078	1.09515	1.567	1.257		
COMPUTE NM HYD	313.00	-	3	.01050	9.23	.314	.56141	1.500	1.374	PER IMP=	35.00
ROUTE	313.10	3	4	.01050	7.33	.314	.56144	1.567	1.090		
COMPUTE NM HYD	314.00	-	3	.01770	4.25	.117	.12345	1.533	.375	PER IMP=	.00
ADD HYD	314.10	3& 4	2	.02820	11.29	.431	.28651	1.567	.628		
ROUTE	314.20	2	4	.02820	11.33	.431	.28653	1.567	.628		
COMPUTE NM HYD	313.00	-	3	.00990	3.60	.094	.17766	1.533	.569	PER IMP=	.00
ADD HYD	313.01	3& 4	2	.03810	14.64	.525	.25823	1.567	.600		
ADD HYD	313.02	2& 9	2	.09080	57.03	3.603	.74397	1.567	.981		
ROUTE	313.03	2	10	.09080	56.56	3.603	.74397	1.567	.973		
COMPUTE NM HYD	314.10	-	3	.02440	9.98	.319	.24485	1.500	.639	PER IMP=	10.00
COMPUTE NM HYD	313.10	-	7	.00980	12.72	.365	.69847	1.500	2.028	PER IMP=	25.00
ADD HYD	314.20	3& 7	2	.03420	22.70	.684	.37482	1.500	1.037		
ADD HYD	313.12	10& 2	6	.12500	76.10	4.286	.64297	1.567	.951		
ROUTE	313.13	6	7	.12500	75.76	4.286	.64297	1.567	.947		
COMPUTE NM HYD	312.00	-	3	.02030	12.67	.391	.36088	1.500	.975	PER IMP=	15.00
ADD HYD	312.10	3& 7	2	.14530	86.79	4.677	.60356	1.567	.933		
ADD HYD	312.20	2& 8	7	.70970	597.99	37.876	1.00067	1.600	1.317		
COMPUTE NM HYD	311.00	-	3	.00470	6.60	.203	.81004	1.500	2.195	PER IMP=	40.00
ADD HYD	311.10	3& 7	2	.71440	602.58	38.079	.99941	1.600	1.318		
ROUTE	311.20	2	4	.71440	601.67	38.079	.99940	1.600	1.316		
ROUTE	311.29	4	6	.71440	601.65	38.078	.99940	1.600	1.316		
COMPUTE NM HYD	316.00	-	3	.00900	10.78	.345	.71912	1.500	1.871	PER IMP=	40.00
COMPUTE NM HYD	317.00	-	5	.02860	15.23	.530	.34734	1.567	.932	PER IMP=	10.00
ADD HYD	317.10	3& 5	2	.03760	25.47	.875	.43631	1.533	1.058		
ADD HYD	317.20	2& 6	3	.75200	623.45	38.953	.97125	1.600	1.295		

AHYMO SUMMARY TABLE (AHYMO491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/21/1994

INPUT FILE = N17F100R.DAT USER NO.= MOLZCONM.I01

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION
RAINFALL TYPE= 1										RAIN6= 2.350
COMPUTE NM HYD	101.00	-	3	.02840	54.60	2.394	1.58050	1.567	3.004	PER IMP= 60.00
ROUTE	102.10	3	7	.02840	49.08	2.394	1.58050	1.633	2.700	
COMPUTE NM HYD	102.00	-	3	.02820	42.56	1.827	1.21480	1.600	2.358	PER IMP= 15.00
ADD HYD	102.20	3& 7	2	.05660	91.16	4.221	1.39828	1.600	2.516	
ROUTE	103.10	2	7	.05660	90.77	4.221	1.39829	1.633	2.506	
COMPUTE NM HYD	103.00	-	3	.00560	17.03	.632	2.11537	1.500	4.753	PER IMP= 100.00
ADD HYD	103.20	3& 7	2	.06220	100.56	4.653	1.46284	1.600	2.526	
ROUTE	104.10	2	7	.06220	100.34	4.853	1.46285	1.633	2.521	
COMPUTE NM HYD	104.00	-	3	.00880	26.76	.993	2.11536	1.500	4.752	PER IMP= 100.00
ADD HYD	104.20	3& 7	2	.07100	115.82	5.846	1.54371	1.600	2.549	
ROUTE	104.10	2	10	.07100	115.96	5.846	1.54372	1.633	2.552	
COMPUTE NM HYD	201.00	-	3	.01290	10.12	1.039	1.50969	2.166	1.226	PER IMP= 42.00
COMPUTE NM HYD	205.00	-	7	.01380	41.96	1.557	2.11537	1.500	4.751	PER IMP= 100.00
ADD HYD	205.10	3& 7	2	.02670	42.45	2.596	1.82270	1.500	2.484	
ROUTE	205.20	2	7	.02670	41.14	2.596	1.82271	1.533	2.407	
ADD HYD	205.10	10& 7	2	.09770	147.10	8.441	1.61995	1.600	2.353	
COMPUTE NM HYD	206.00	-	3	.01380	41.96	1.557	2.11537	1.500	4.751	PER IMP= 100.00
ADD HYD	206.10	2& 3	2	.11150	180.49	9.998	1.68127	1.567	2.529	
ROUTE	206.20	2	7	.11150	178.87	9.998	1.68127	1.567	2.507	
COMPUTE NM HYD	204.00	-	3	.02170	58.67	2.060	1.77963	1.500	4.225	PER IMP= 66.00
ROUTE	504.10	3	5	.02170	57.87	2.060	1.77964	1.500	4.167	
ADD HYD	207.10	5& 7	2	.13320	229.32	12.058	1.69729	1.567	2.690	
COMPUTE NM HYD	203.00	-	3	.02100	63.85	2.369	2.11536	1.500	4.751	PER IMP= 100.00
ROUTE	203.20	3	7	.02100	62.11	2.369	2.11538	1.500	4.621	
COMPUTE NM HYD	208.00	-	3	.02250	68.41	2.538	2.11537	1.500	4.751	PER IMP= 100.00
ADD HYD	208.10	3& 7	5	.04350	130.52	4.908	2.11535	1.500	4.688	
ROUTE	208.20	5	7	.04350	123.32	4.908	2.11536	1.533	4.430	
ADD HYD	207.20	2& 7	8	.17670	349.54	16.965	1.80021	1.533	3.091	
COMPUTE NM HYD	207.00	-	3	.02190	66.59	2.471	2.11536	1.500	4.751	PER IMP= 100.00
ADD HYD	207.30	3& 8	2	.19860	411.98	19.436	1.83496	1.533	3.241	
ROUTE	207.40	2	7	.19860	409.27	19.436	1.83496	1.533	3.220	
COMPUTE NM HYD	209.00	-	3	.04170	112.59	4.705	2.11537	1.533	4.219	PER IMP= 100.00
ADD HYD	209.10	3& 7	2	.24030	521.86	24.140	1.88362	1.533	3.393	
ROUTE	210.20	2	9	.24030	520.89	24.140	1.88362	1.533	3.387	
COMPUTE NM HYD	216.00	-	3	.01340	40.75	1.512	2.11537	1.500	4.751	PER IMP= 100.00
ROUTE	210.20	3	5	.01340	38.61	1.512	2.11538	1.533	4.503	
COMPUTE NM HYD	215.00	-	3	.01240	37.71	1.399	2.11537	1.500	4.752	PER IMP= 100.00
ADD HYD	221.10	5& 3	2	.02580	74.90	2.911	2.11534	1.500	4.536	
ADD HYD	221.10	9& 2	10	.26610	594.86	27.051	1.90609	1.533	3.493	
COMPUTE NM HYD	220.00	-	3	.03270	97.81	3.603	2.06599	1.500	4.674	PER IMP= 95.00
ROUTE	210.20	3	9	.03270	89.80	3.603	2.06600	1.533	4.291	
ADD HYD	221.10	10& 9	11	.29880	684.67	30.654	1.92358	1.533	3.580	
ROUTE	210.20	11	9	.29880	680.08	30.654	1.92358	1.567	3.556	
COMPUTE NM HYD	221.00	-	3	.00430	12.42	.454	1.98165	1.500	4.514	PER IMP= 90.00
ROUTE	221.20	3	8	.00430	12.18	.454	1.98170	1.500	4.427	
ROUTE	213.30	8	7	.00430	11.82	.454	1.98170	1.533	4.295	
COMPUTE NM HYD	219.00	-	3	.01840	37.37	2.010	2.04851	1.633	3.216	PER IMP= 95.00
ADD HYD	219.10	3& 7	2	.02270	46.71	2.465	2.03581	1.633	3.215	
ROUTE	219.20	2	6	.02270	46.83	2.465	2.03583	1.633	3.223	
COMPUTE NM HYD	211.18	-	3	.01920	45.23	2.029	1.98165	1.567	3.681	PER IMP= 90.00
ROUTE	211.10	3	7	.01920	44.88	2.029	1.98165	1.567	3.652	
ADD HYD	218.00	6& 7	8	.04190	91.22	4.494	2.01098	1.600	3.402	
COMPUTE NM HYD	214.00	-	3	.00560	15.10	.529	1.76975	1.500	4.213	PER IMP= 65.00
ROUTE	214.40	3	7	.00560	14.85	.528	1.76978	1.500	4.172	

ADD HYD	214.11	3& 7	2	.00940	26.33	.947	1.88945	1.500	4.377		
ROUTE	214.12	2	7	.00940	26.16	.947	1.88951	1.500	4.348		
ADD HYD	214.50	8& 7	2	.05130	111.81	5.441	1.98872	1.567	3.406		
ROUTE	214.60	2	7	.05130	111.77	5.441	1.98872	1.567[1]	3.404		
ADD HYD	214.70	9& 7	2	.35010	791.85	36.095	1.93313	1.567[1]	3.534		
ROUTE	214.80	2	6	.35010	793.37	36.095	1.93313	1.567[1]	3.541		
COMPUTE NM HYD	224.00	-	3	.00790	19.42	.658	1.56192	1.500[1]	3.940	PER IMP=	50.00
ROUTE	224.10	3	7	.00790	19.38	.658	1.56195	1.500	3.833		
ADD HYD	224.20	7& 6	2	.35800	809.90	36.753	1.92494	1.567	3.535		
COMPUTE NM HYD	214.20	-	3	.01220	24.63	.720	1.10660	1.500	3.154	PER IMP=	.00
COMPUTE NM HYD	225.00	-	4	.01050	25.17	.825	1.47351	1.500	3.746	PER IMP=	35.00
ADD HYD	225.10	3& 4	5	.02270	49.80	1.545	1.27641	1.500	3.428		
ROUTE RESERVOIR	525.00	5	4	.02270	41.28	1.545	1.27641	1.567	2.842	AC-FT=	.279
ADD HYD	214.90	2& 4	5	.38070	851.18	38.299	1.88627	1.567	3.493		
ROUTE	214.91	5	9	.38070	852.91	38.299	1.88627	1.567	3.501		
COMPUTE NM HYD	217.00	-	3	.02250	63.96	2.301	1.91787	1.500	4.442	PER IMP=	80.00
ROUTE RESERVOIR	517.00	3	4	.02250	10.94	.595	.49574	2.100	.760	AC-FT=	1.955
COMPUTE NM HYD	217.11	-	2	.00520	14.88	.538	1.94129	1.500	4.470	PER IMP=	84.00
ADD HYD	217.20	2& 4	1	.02770	14.95	1.133	.76710	1.500	.843		
ADD HYD	217.14	9& 1	4	.40840	865.03	39.432	1.81036	1.567	3.310		
ROUTE	217.30	4	7	.40840	866.29	39.432	1.81034	1.567	3.314		
COMPUTE NM HYD	223.00	-	3	.02770	76.41	2.753	1.86326	1.500	4.310	PER IMP=	60.00
ADD HYD	223.10	3& 7	2	.43610	928.26	42.184	1.81371	1.567	3.326		
ROUTE	223.20	2	8	.43610	928.38	42.184	1.81370	1.567	3.326		
COMPUTE NM HYD	222.00	-	3	.02150	45.55	2.196	1.91479	1.600	3.310	PER IMP=	85.00
COMPUTE NM HYD	229.00	-	7	.00510	14.73	.539	1.98165	1.500	4.513	PER IMP=	90.00
ADD HYD	229.10	3& 7	2	.02660	55.91	2.735	1.92758	1.567	3.284		
ROUTE	229.20	2	7	.02660	55.74	2.735	1.92760	1.600	3.274		
COMPUTE NM HYD	227.00	-	3	.00870	18.38	.609	1.31307	1.500	3.300	PER IMP=	40.00
ADD HYD	227.10	3& 7	2	.03530	70.75	3.344	1.77612	1.567	3.132		
COMPUTE NM HYD	226.00	-	3	.00750	18.75	.659	1.64736	1.500	3.906	PER IMP=	65.00
ADD HYD	226.10	3& 2	4	.04280	87.14	4.003	1.75355	1.533	3.181		
ROUTE	226.20	4	7	.04280	86.56	4.003	1.75356	1.567	3.160		
COMPUTE NM HYD	228.00	-	3	.00400	11.25	.408	1.91479	1.500	4.394	PER IMP=	85.00
ADD HYD	228.10	3& 7	2	.04680	96.22	4.411	1.76733	1.533	3.212		
ADD HYD	228.20	2& 8	5	.48290	1024.03	46.596	1.80921	1.567	3.313		
ROUTE	100.10	5	7	.48290	1024.02	46.595	1.80919	1.567	3.313		
COMPUTE NM HYD	301.00	-	3	.01970	57.39	2.099	1.99749	1.500	4.552	PER IMP=	90.00
COMPUTE NM HYD	302.00	-	6	.00540	16.17	.595	2.06599	1.500	4.678	PER IMP=	95.00
ADD HYD	302.10	3& 6	2	.02510	73.55	2.694	2.01221	1.500	4.579		
COMPUTE NM HYD	303.00	-	3	.00310	7.35	.257	1.55256	1.500	3.703	PER IMP=	60.00
ROUTE	303.10	3	4	.00310	6.49	.257	1.55264	1.533	3.270		
ADD HYD	303.15	2& 4	1	.02820	79.33	2.950	1.96167	1.500	4.396		
ADD HYD	303.16	1& 7	5	.51110	1089.67	49.546	1.81761	1.567	3.331		
ROUTE	303.20	5	4	.51110	1090.53	49.545	1.81759	1.567	3.334		
COMPUTE NM HYD	307.00	-	3	.02350	64.18	2.316	1.84793	1.500	4.267	PER IMP=	80.00
COMPUTE NM HYD	308.00	-	2	.01910	52.16	1.882	1.84793	1.500	4.267	PER IMP=	80.00
ADD HYD	308.10	3& 2	7	.04260	116.34	4.198	1.84792	1.500	4.267		
ADD HYD	308.20	4& 7	2	.55370	1184.93	53.744	1.81992	1.567	3.344		
ROUTE	308.30	2	4	.55370	1184.39	53.743	1.81990	1.567	3.342		
COMPUTE NM HYD	310.00	-	3	.01070	29.81	1.073	1.87962	1.500	4.353	PER IMP=	80.00
ADD HYD	310.10	3& 4	7	.56440	1208.55	54.815	1.82103	1.567	3.346		
ROUTE	308.30	7	8	.56440	1207.35	54.815	1.82102	1.567	3.342		
COMPUTE NM HYD	304.00	-	3	.01320	31.90	1.113	1.58050	1.500	3.776	PER IMP=	60.00
ROUTE RESERVOIR	504.00	3	6	.01320	2.77	1.113	1.58038	2.166	.328	AC-FT=	.860
ROUTE	304.10	6	4	.01320	2.74	1.113	1.58033	2.333	.325		
COMPUTE NM HYD	306.00	-	3	.01630	48.31	1.781	2.04851	1.500	4.631	PER IMP=	95.00
ROUTE RESERVOIR	505.00	3	6	.01630	15.97	1.781	2.04849	1.833	1.531	AC-FT=	.798
COMPUTE NM HYD	305.00	-	7	.01590	43.50	1.592	1.87774	1.500	4.275	PER IMP=	85.00
ADD HYD	305.10	6& 7	2	.03220	53.58	3.373	1.96414	1.500	2.600		
ADD HYD	305.20	4& 2	2	.04540	54.36	4.486	1.85254	1.500	1.871		
ROUTE	305.30	2	4	.04540	55.00	4.486	1.85255	1.533	1.893		
COMPUTE NM HYD	309.00	-	3	.00730	16.75	.590	1.51515	1.500	3.586	PER IMP=	60.00
ADD HYD	309.10	3& 4	2	.05270	70.93	5.075	1.80580	1.533	2.103		
ROUTE	309.20	2	9	.05270	69.92	5.076	1.80581	1.533	2.073		

ROUTE	315.10	3	4	.01050	16.36	.620	1.10748	1.567	2.434		
COMPUTE NM HYD	314.00	-	3	.01770	17.94	.501	.53121	1.533	1.584	PER IMP=	.00
ADD HYD	314.10	3& 4	2	.02820	33.97	1.122	.74576	1.533	1.882		
ROUTE	314.20	2	4	.02820	33.83	1.122	.74578	1.567	1.874		
COMPUTE NM HYD	313.00	-	3	.00990	11.75	.328	.62056	1.533	1.855	PER IMP=	.00
ADD HYD	313.01	3& 4	2	.03810	45.10	1.449	.71323	1.533	1.849		
ADD HYD	313.02	2& 9	2	.09080	115.02	6.525	1.34735	1.533	1.979		
ROUTE	313.03	2	10	.09080	115.65	6.525	1.34736	1.567	1.990		
COMPUTE NM HYD	314.10	-	3	.02440	29.47	.697	.68963	1.500	1.887	PER IMP=	10.00
COMPUTE NM HYD	313.10	-	7	.00980	22.22	.708	1.35383	1.500	3.543	PER IMP=	25.00
ADD HYD	314.20	3& 7	2	.03420	51.69	1.605	.87994	1.500	2.362		
ADD HYD	313.12	10& 2	6	.12500	161.56	8.130	1.21947	1.533	2.020		
ROUTE	313.13	5	7	.12500	162.20	8.130	1.21947	1.567	2.028		
COMPUTE NM HYD	312.00	-	3	.02030	30.24	.930	.85908	1.500	2.328	PER IMP=	15.00
ADD HYD	312.10	3& 7	2	.14530	188.97	9.060	1.16911	1.567	2.032		
ADD HYD	312.20	2& 8	7	.70970	1396.32	63.875	1.68755	1.567	3.074		
COMPUTE NM HYD	311.00	-	3	.00470	11.23	.371	1.48196	1.500	3.732	PER IMP=	40.00
ADD HYD	311.10	3& 7	2	.71440	1405.68	64.246	1.68620	1.567	3.074		
ROUTE	311.20	2	4	.71440	1397.70	64.246	1.68618	1.567	3.057		
ROUTE	311.20	4	5	.71440	1397.62	64.246	1.68618	1.567	3.057		
COMPUTE NM HYD	316.00	-	3	.00900	19.47	.645	1.34324	1.500	3.380	PER IMP=	40.00
COMPUTE NM HYD	317.00	-	5	.02860	36.40	1.305	.85576	1.567	1.988	PER IMP=	10.00
ADD HYD	317.10	3& 5	2	.03760	54.74	1.950	.97243	1.533	2.275		
ADD HYD	317.20	2& 5	3	.75200	1450.41	66.196	1.65049	1.567	3.014		

APPENDIX C

SUMMARY PRINTOUTS
BASIN KC

AHYMO SUMMARY TABLE (AHYMO491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/17/1994

INPUT FILE = kcex10.dat USER NO.= MOLZCONM.101

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	1.570
RECALL HYD	100.00	-	1	.43040	336.15	21.167	.92214	1.567	1.220		
ROUTE	100.10	1	7	.43040	336.92	21.167	.92212	1.567	1.223		
COMPUTE NM HYD	301.00	-	3	.01970	37.04	1.304	1.24145	1.500	2.938	PER IMP=	99.00
COMPUTE NM HYD	302.00	-	6	.00540	10.55	.373	1.29582	1.500	3.052	PER IMP=	95.00
ADD HYD	302.10	3& 6	2	.02510	47.59	1.678	1.25312	1.500	2.963		
COMPUTE NM HYD	303.00	-	3	.00310	4.33	.148	.89555	1.500	2.181	PER IMP=	60.00
ROUTE	303.10	3	4	.00310	3.71	.148	.89562	1.567	1.869		
ADD HYD	303.15	2& 4	5	.02820	50.80	1.826	1.21381	1.500	2.814		
ADD HYD	303.16	5& 7	5	.45860	373.86	22.993	.94007	1.567	1.291		
ROUTE	303.20	5	4	.45860	381.17	22.992	.94005	1.567	1.299		
COMPUTE NM HYD	307.00	-	3	.02350	40.32	1.410	1.12508	1.500	2.681	PER IMP=	80.00
COMPUTE NM HYD	308.00	-	2	.01910	32.77	1.146	1.12508	1.500	2.681	PER IMP=	80.00
ADD HYD	308.10	3& 2	7	.04260	73.10	2.556	1.12506	1.500	2.681		
ADD HYD	308.20	4& 7	2	.50120	440.23	25.548	.95577	1.567	1.372		
ROUTE	308.30	2	4	.50120	444.08	25.548	.95575	1.567	1.384		
COMPUTE NM HYD	310.00	-	3	.01070	18.88	.654	1.14543	1.500	2.757	PER IMP=	80.00
ADD HYD	310.10	3& 4	7	.51190	459.35	26.202	.95972	1.567	1.402		
ROUTE	308.30	7	8	.51190	461.57	26.201	.95970	1.567	1.409		
COMPUTE NM HYD	304.00	-	3	.01320	18.92	.643	.91269	1.500	2.240	PER IMP=	60.00
ROUTE RESERVOIR	504.00	3	6	.01320	1.78	.642	.91264	2.166	.210	AC-FT=	.488
ROUTE	304.10	6	4	.01320	1.76	.642	.91251	2.333	.209		
COMPUTE NM HYD	306.00	-	3	.01630	31.40	1.117	1.28437	1.500	3.010	PER IMP=	95.00
ROUTE RESERVOIR	505.00	3	6	.01630	11.16	1.117	1.28435	1.800	1.069	AC-FT=	.494
COMPUTE NM HYD	305.00	-	7	.01590	27.54	.980	1.15536	1.500	2.706	PER IMP=	85.00
ADD HYD	305.10	6& 7	2	.03220	33.82	2.096	1.22062	1.500	1.641		
ADD HYD	305.20	4& 2	2	.04540	34.36	2.739	1.13103	1.500	1.182		
ROUTE	305.30	2	4	.04540	34.84	2.739	1.13104	1.533	1.199		
COMPUTE NM HYD	309.00	-	3	.00730	9.77	.340	.87201	1.500	2.090	PER IMP=	60.00
ADD HYD	309.10	3& 4	2	.05270	44.08	3.078	1.09514	1.533	1.307		
ROUTE	309.20	2	9	.05270	42.39	3.078	1.09515	1.567	1.257		
COMPUTE NM HYD	315.00	-	3	.01050	9.23	.314	.56141	1.500	1.374	PER IMP=	35.00
ROUTE	315.10	3	4	.01050	7.33	.314	.56144	1.567	1.090		
COMPUTE NM HYD	314.00	-	3	.01770	4.25	.117	.12345	1.533	.375	PER IMP=	.00
ADD HYD	314.10	3& 4	2	.02820	11.29	.431	.28651	1.567	.626		
ROUTE	314.20	2	4	.02820	11.33	.431	.28653	1.567	.628		
COMPUTE NM HYD	313.00	-	3	.00990	3.60	.094	.17766	1.533	.568	PER IMP=	.00
ADD HYD	313.01	3& 4	2	.03810	14.64	.525	.25823	1.567	.600		
ADD HYD	313.02	2& 9	2	.09080	57.03	3.603	.74397	1.567	.981		
ROUTE	313.03	2	10	.09080	56.56	3.603	.74397	1.567	.973		
COMPUTE NM HYD	314.10	-	3	.02440	9.98	.319	.24485	1.500	.639	PER IMP=	10.00
COMPUTE NM HYD	313.10	-	7	.00980	12.72	.365	.69847	1.500	2.028	PER IMP=	25.00
ADD HYD	314.20	3& 7	2	.03420	22.70	.684	.37482	1.500	1.037		
ADD HYD	313.12	10& 2	6	.12500	76.10	4.286	.64297	1.567	.951		
ROUTE	313.13	6	7	.12500	75.76	4.286	.64297	1.567	.947		
COMPUTE NM HYD	312.00	-	3	.02030	12.67	.391	.36088	1.500	.975	PER IMP=	15.00
ADD HYD	312.10	3& 7	2	.14530	86.79	4.677	.60356	1.567	.933		
ADD HYD	312.20	2& 8	7	.65720	548.36	30.878	.88096	1.567	1.304		
COMPUTE NM HYD	311.00	-	3	.00470	6.60	.203	.81004	1.500	2.195	PER IMP=	40.00
ADD HYD	311.10	3& 7	2	.66190	553.86	31.081	.88046	1.567	1.307		
ROUTE	311.20	2	4	.66190	554.32	31.081	.88044	1.567	1.309		
ROUTE	311.20	4	6	.66190	554.23	31.081	.88044	1.567	1.308		
COMPUTE NM HYD	316.00	-	3	.00900	10.78	.345	.71912	1.500	1.871	PER IMP=	40.00
COMPUTE NM HYD	317.00	-	5	.02860	15.23	.530	.34734	1.567	.832	PER IMP=	10.00
ADD HYD	317.10	3& 5	2	.03760	25.47	.875	.43631	1.533	1.058		
ADD HYD	317.20	2& 6	3	.69950	578.50	31.956	.85656	1.567	1.292		

AHYMO SUMMARY TABLE (AHYMO491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/17/1994

INPUT FILE = kcex100.dat USER NO.= MOLZCONM.101

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION
RAINFALL TYPE= 1										RAIN6= 2.350
RECALL HYD	228.20	-	1	.43040	527.44	36.993	1.61158	1.567	1.915	
ROUTE	100.10	1	7	.43040	528.27	36.993	1.61157	1.567	1.918	
COMPUTE NM HYD	301.00	-	3	.01970	57.39	2.099	1.99749	1.500	4.552	PER IMP= 90.00
COMPUTE NM HYD	302.00	-	6	.00540	16.17	.595	2.06599	1.500	4.678	PER IMP= 95.00
ADD HYD	302.10	3& 6	2	.02510	73.55	2.694	2.01221	1.500	4.579	
COMPUTE NM HYD	303.00	-	3	.00310	7.35	.257	1.55256	1.500	3.703	PER IMP= 60.00
ROUTE	303.10	3	4	.00310	6.49	.257	1.55264	1.533	3.270	
ADD HYD	303.15	2& 4	5	.02820	79.33	2.950	1.96167	1.500	4.396	
ADD HYD	303.16	5& 7	5	.45860	593.93	39.943	1.63309	1.567	2.024	
ROUTE	303.20	5	4	.45860	596.92	39.943	1.63307	1.567	2.034	
COMPUTE NM HYD	307.00	-	3	.02350	64.18	2.316	1.84793	1.500	4.267	PER IMP= 80.00
COMPUTE NM HYD	308.00	-	2	.01910	52.16	1.882	1.84793	1.500	4.267	PER IMP= 80.00
ADD HYD	308.10	3& 2	7	.04260	116.34	4.198	1.84792	1.500	4.267	
ADD HYD	308.20	4& 7	2	.50120	691.32	44.141	1.65134	1.567	2.155	
ROUTE	308.30	2	4	.50120	696.09	44.141	1.65131	1.567	2.170	
COMPUTE NM HYD	310.00	-	3	.01070	29.81	1.073	1.87962	1.500	4.353	PER IMP= 80.00
ADD HYD	310.10	3& 4	7	.51190	720.25	45.213	1.65609	1.567	2.198	
ROUTE	308.30	7	8	.51190	723.22	45.213	1.65607	1.567	2.208	
COMPUTE NM HYD	304.00	-	3	.01320	31.90	1.113	1.58050	1.500	3.776	PER IMP= 60.00
ROUTE RESERVOIR	504.00	3	6	.01320	2.77	1.113	1.58038	2.166	.328	AC-FT= .860
ROUTE	304.10	6	4	.01320	2.74	1.113	1.58033	2.333	.325	
COMPUTE NM HYD	306.00	-	3	.01630	48.31	1.781	2.04851	1.500	4.631	PER IMP= 95.00
ROUTE RESERVOIR	505.00	3	6	.01630	15.97	1.781	2.04849	1.833	1.531	AC-FT= .798
COMPUTE NM HYD	305.00	-	7	.01590	43.50	1.592	1.87774	1.500	4.275	PER IMP= 85.00
ADD HYD	305.10	6& 7	2	.03220	53.58	3.373	1.96414	1.500	2.600	
ADD HYD	305.20	4& 2	2	.04540	54.36	4.486	1.85254	1.500	1.871	
ROUTE	305.30	2	4	.04540	55.00	4.486	1.85255	1.533	1.893	
COMPUTE NM HYD	309.00	-	3	.00730	16.75	.590	1.51515	1.500	3.586	PER IMP= 60.00
ADD HYD	309.10	3& 4	2	.05270	70.93	5.075	1.80580	1.533	2.103	
ROUTE	309.20	2	9	.05270	69.92	5.076	1.80581	1.533	2.073	
COMPUTE NM HYD	315.00	-	3	.01050	18.51	.620	1.10746	1.500	2.755	PER IMP= 35.00
ROUTE	315.10	3	4	.01050	16.36	.620	1.10748	1.567	2.434	
COMPUTE NM HYD	314.00	-	3	.01770	17.94	.501	.53121	1.533	1.584	PER IMP= .00
ADD HYD	314.10	3& 4	2	.02820	33.97	1.122	.74576	1.533	1.882	
ROUTE	314.20	2	4	.02820	33.83	1.122	.74578	1.567	1.874	
COMPUTE NM HYD	313.00	-	3	.00990	11.75	.328	.62056	1.533	1.855	PER IMP= .00
ADD HYD	313.01	3& 4	2	.03810	45.10	1.449	.71323	1.533	1.849	
ADD HYD	313.02	2& 9	2	.09080	115.02	6.525	1.34735	1.533	1.979	
ROUTE	313.03	2	10	.09080	115.65	6.525	1.34736	1.567	1.990	
COMPUTE NM HYD	314.10	-	3	.02440	29.47	.897	.68963	1.500	1.887	PER IMP= 10.00
COMPUTE NM HYD	313.10	-	7	.00980	22.22	.708	1.35383	1.500	3.543	PER IMP= 25.00
ADD HYD	314.20	3& 7	2	.03420	51.69	1.605	.87994	1.500	2.362	
ADD HYD	313.12	10& 2	6	.12500	161.56	8.130	1.21947	1.533	2.020	
ROUTE	313.13	6	7	.12500	162.20	8.130	1.21947	1.567	2.028	
COMPUTE NM HYD	312.00	-	3	.02030	30.24	.930	.85908	1.500	2.328	PER IMP= 15.00
ADD HYD	312.10	3& 7	2	.14530	188.97	9.060	1.16911	1.567	2.032	
ADD HYD	312.20	2& 8	7	.65720	912.19	54.273	1.54841	1.567	2.169	
COMPUTE NM HYD	311.00	-	3	.00470	11.23	.371	1.48196	1.500	3.732	PER IMP= 40.00
ADD HYD	311.10	3& 7	2	.66190	921.55	54.644	1.54793	1.567	2.175	
ROUTE	311.20	2	4	.66190	924.45	54.643	1.54791	1.567	2.192	
ROUTE	311.20	4	6	.66190	924.32	54.643	1.54791	1.567	2.182	

COMPUTE NM HYD	317.00	-	5	.02860	36.40	1.305	.85576	1.567	1.988	PER IMP= 10.00
ADD HYD	317.10	3& 5	2	.03760	54.74	1.950	.97243	1.533	2.275	
ADD HYD	317.20	2& 6	3	.69950	977.11	56.593	1.51698	1.567	2.183	

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APPENDIX C

SUMMARY PRINTOUTS
BASIN SP

AHYMO SUMMARY TABLE (AHYMO491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/17/1994
INPUT FILE = spex10.dat USER NO.= MOLZCONM.I01

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	1.570
COMPUTE NM HYD	401.00	-	5	.02800	51.96	1.839	1.23127	1.500	2.899	PER IMP=	90.00
ROUTE	200.00	5	2	.02800	44.36	1.839	1.23128	1.567	2.476		
COMPUTE NM HYD	402.00	-	5	.08700	56.37	1.612	.34734	1.500	1.012	PER IMP=	10.00
ADD HYD	113.30	2& 5	3	.11500	99.16	3.450	.56255	1.533	1.347		
COMPUTE NM HYD	407.00	-	5	.01500	7.47	.209	.26160	1.533	.779	PER IMP=	5.26
ROUTE	202.00	5	2	.01500	7.09	.209	.26162	1.567	.738		
ADD HYD	113.30	2& 3	4	.13000	106.19	3.660	.52783	1.533	1.276		
ROUTE RESERVOIR	201.10	4	1	.13000	58.78	2.994	.43183	1.700	.707	AC-FT=	1.490
ROUTE	201.20	1	3	.13000	60.10	2.994	.43182	1.667	.722		
ROUTE	202.00	3	2	.13000	56.59	2.993	.43171	1.733	.680		
COMPUTE NM HYD	403.00	-	3	.02600	10.62	.287	.20670	1.533	.638	PER IMP=	2.00
ADD HYD	202.30	2& 3	4	.15600	60.30	3.280	.39421	1.733	.604		
ROUTE	202.00	4	2	.15600	59.15	3.279	.39413	1.766	.592		
COMPUTE NM HYD	404.00	-	3	.02500	10.21	.276	.20670	1.533	.638	PER IMP=	2.00
ADD HYD	202.30	2& 3	4	.18100	62.09	3.555	.36824	1.766	.536		
ROUTE	202.20	4	3	.18100	62.12	3.555	.36823	1.766	.536		
COMPUTE NM HYD	405.00	-	2	.00300	3.80	.105	.65442	1.500	1.980	PER IMP=	18.00
ADD HYD	202.30	2& 3	4	.18400	63.18	3.659	.37290	1.766	.537		
ROUTE	202.30	4	2	.18400	58.81	3.658	.37278	1.833	.499		
COMPUTE NM HYD	406.00	-	3	.01700	6.94	.187	.20670	1.533	.638	PER IMP=	2.00
ADD HYD	202.40	2& 3	19	.20100	60.19	3.846	.35873	1.800	.468		

AHYMO SUMMARY TABLE (AHYMO491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/17/1994

INPUT FILE = spex100.dat USER NO.= MOLZCONM.I01

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	2.350
COMPUTE NM HYD	401.00	-	5	.02800	30.77	2.959	1.98165	1.500	4.507	PER IMP=	90.00
ROUTE	200.00	5	2	.02800	71.48	2.959	1.98166	1.533	3.989		
COMPUTE NM HYD	402.00	-	5	.08700	134.17	3.971	.85576	1.500	2.410	PER IMP=	10.00
ADD HYD	113.30	2& 5	3	.11500	203.50	6.930	1.12988	1.533	2.765		
COMPUTE NM HYD	407.00	-	5	.01500	20.25	.588	.73505	1.500	2.109	PER IMP=	5.26
ROUTE	202.00	5	2	.01500	19.88	.588	.73506	1.533	2.070		
ADD HYD	113.30	2& 3	4	.13000	223.38	7.518	1.08433	1.533	2.685		
ROUTE RESERVOIR	201.10	4	1	.13000	114.63	6.848	.98765	1.700	1.378	AC-FT=	1.971
ROUTE	201.20	1	3	.13000	116.63	6.848	.98765	1.667	1.402		
ROUTE	202.00	3	2	.13000	114.19	6.847	.98754	1.733	1.372		
COMPUTE NM HYD	403.00	-	3	.02600	32.22	.914	.65929	1.500	1.936	PER IMP=	2.00
ADD HYD	202.30	2& 3	4	.15600	128.40	7.761	.93283	1.700	1.286		
ROUTE	202.00	4	2	.15600	127.49	7.760	.93275	1.733	1.277		
COMPUTE NM HYD	404.00	-	3	.02500	30.98	.879	.65929	1.500	1.936	PER IMP=	2.00
ADD HYD	202.30	2& 3	4	.18100	141.56	8.639	.89497	1.700	1.222		
ROUTE	202.20	4	3	.18100	141.63	8.639	.89497	1.700	1.223		
COMPUTE NM HYD	405.00	-	2	.00300	6.70	.209	1.30565	1.500	3.489	PER IMP=	18.00
ADD HYD	202.30	2& 3	4	.18400	144.59	8.848	.90166	1.700	1.228		
ROUTE	202.30	4	2	.18400	142.32	8.847	.90154	1.733	1.209		
COMPUTE NM HYD	406.00	-	3	.01700	21.07	.598	.65929	1.500	1.937	PER IMP=	2.00
ADD HYD	202.40	2& 3	19	.20100	150.64	9.445	.88105	1.700	1.171		

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 2										RAIN24=	2.750
COMPUTE NM HYD	200.00	-	3	.05000	133.22	5.598	2.01846	1.500	4.003	PER IMP=	90.00
ROUTE	200.01	3	9	.05200	133.44	5.593	2.01848	1.500	4.010		
COMPUTE NM HYD	301.00	-	4	.01870	50.69	2.195	2.20193	1.500	4.235	PER IMP=	80.00
ROUTE	301.01	4	10	.01870	50.37	2.195	2.20096	1.500	4.224		
ADD HYD	301.11	9&10	2	.07070	184.02	7.793	2.06673	1.500	4.027		
ROUTE	301.12	2	3	.07070	182.16	7.793	2.06674	1.500	4.025		
COMPUTE NM HYD	201.00	-	4	.04570	125.37	5.425	2.22578	1.500	4.286	PER IMP=	80.00
ADD HYD	201.10	3& 4	20	.11640	307.53	13.218	2.12917	1.500	4.128		
ROUTE RESERVOIR	505.00	20	4	.11640	82.34	13.218	2.12917	1.950	1.109	AC-FT=	5.892
ROUTE	201.11	4	5	.11640	82.66	13.218	2.12917	1.950	1.110		
COMPUTE NM HYD	500.00	-	7	.02100	53.67	2.187	1.95275	1.500	3.993	PER IMP=	60.50
ADD HYD	500.10	6& 7	8	.13740	109.61	15.405	2.10220	1.550	1.246		
COMPUTE NM HYD	400.00	-	3	.02400	59.98	2.499	1.95264	1.500	3.905	PER IMP=	65.70
ADD HYD	400.10	8& 3	6	.16140	162.08	17.904	2.07996	1.550	1.575		
ROUTE	400.11	6	7	.16140	163.39	17.904	2.07996	1.550	1.587		
COMPUTE NM HYD	103.00	-	3	.01060	30.05	1.334	2.35904	1.500	4.430	PER IMP=	90.00
ADD HYD	103.10	3& 7	8	.17200	190.69	19.238	2.09716	1.500	1.734		
ROUTE	103.11	8	9	.17200	191.37	19.238	2.09716	1.550	1.738		
COMPUTE NM HYD	104.00	-	3	.00263	8.02	.243	1.73442	1.500	3.575	PER IMP=	55.00
ADD HYD	104.10	9& 3	10	.17463	196.74	19.481	2.09169	1.550	1.760		
COMPUTE NM HYD	101.00	-	3	.00242	6.35	.213	1.64745	1.500	3.457	PER IMP=	50.00
ADD HYD	101.10	10& 3	11	.17705	201.53	19.694	2.08562	1.550	1.779		
COMPUTE NM HYD	102.00	-	3	.00087	2.46	.111	2.51715	1.500	4.636	PER IMP=	100.00
ADD HYD	102.10	11& 3	12	.17788	203.90	19.805	2.08763	1.500	1.791		
ROUTE	102.11	12	13	.17788	204.45	19.805	2.08763	1.550	1.796		
COMPUTE NM HYD	109.00	-	3	.01020	28.92	1.287	2.35904	1.500	4.430	PER IMP=	90.00
ADD HYD	109.10	13& 3	14	.18808	232.29	21.089	2.10235	1.500	1.930		
ROUTE	109.11	14	15	.18808	231.57	21.089	2.10235	1.500	1.924		
ROUTE	109.11	15	1	.18808	232.20	21.089	2.10235	1.550	1.929		
COMPUTE NM HYD	106.00	-	3	.00165	4.89	.222	2.51715	1.500	4.629	PER IMP=	100.00
ADD HYD	106.10	1& 3	16	.18973	236.46	21.310	2.10595	1.550	1.947		
COMPUTE NM HYD	105.00	-	3	.00067	1.84	.031	.86471	1.500	2.431	PER IMP=	5.00
ADD HYD	105.10	16& 3	17	.19040	237.43	21.340	2.10150	1.550	1.948		
COMPUTE NM HYD	107.00	-	3	.00075	1.17	.035	.86471	1.500	2.428	PER IMP=	5.00
ADD HYD	107.10	17& 3	18	.19115	238.53	21.374	2.09655	1.550	1.950		
COMPUTE NM HYD	108.00	-	3	.00100	2.97	.134	2.51715	1.500	4.633	PER IMP=	100.00
ADD HYD	108.10	18& 3	19	.19215	241.11	21.508	2.09874	1.550	1.961		
COMPUTE NM HYD	110.00	-	3	.00950	26.94	1.195	2.35904	1.500	4.431	PER IMP=	90.00
ADD HYD	110.10	19& 3	5	.20165	267.84	22.703	2.11100	1.500	2.075		
ROUTE	110.11	5	1	.20165	267.05	22.703	2.11100	1.500	2.069		
COMPUTE NM HYD	111.00	-	3	.00100	2.97	.134	2.51715	1.500	4.633	PER IMP=	100.00
ADD HYD	111.10	1& 3	4	.20265	270.02	22.837	2.11300	1.500	2.082		
COMPUTE NM HYD	112.00	-	3	.00059	.92	.027	.86471	1.500	2.439	PER IMP=	5.00
ADD HYD	112.10	4& 3	5	.20324	270.94	22.864	2.10931	1.500	2.083		
COMPUTE NM HYD	113.00	-	3	.00092	2.73	.124	2.51715	1.500	4.636	PER IMP=	100.00
ADD HYD	113.10	5& 3	6	.20416	273.67	22.987	2.11115	1.500	2.094		
COMPUTE NM HYD	114.00	-	3	.00620	17.28	.757	2.28884	1.500	4.356	PER IMP=	85.00
ADD HYD	114.10	6& 3	7	.21036	290.95	23.744	2.11638	1.500	2.161		
ROUTE	114.11	7	8	.21036	290.24	23.744	2.11638	1.500	2.156		
ROUTE	114.12	8	9	.21036	290.66	23.744	2.11639	1.550	2.159		
ROUTE	114.13	9	10	.21036	291.35	23.744	2.11639	1.550	2.164		
ROUTE	114.14	10	11	.21036	292.74	23.744	2.11638	1.550	2.174		

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	1.870
COMPUTE NM HYD	101.00	-	3	.04300	77.56	2.579	1.16824	1.500	2.818	PER IMP=	61.00
ROUTE RESERVOIR	105.01	3	4	.04300	77.51	2.579	1.16824	1.500	2.817	AC-FT=	.009
ROUTE	105.10	4	7	.04300	69.87	2.579	1.16825	1.533	2.537		
COMPUTE NM HYD	102.00	-	1	.03200	51.20	1.184	1.06275	1.467	2.005	PER IMP=	59.00
ADD HYD	234.50	3& 7	2	.06100	118.31	4.333	1.11874	1.533	2.262		
ROUTE RESERVOIR	105.01	2	4	.06100	118.16	4.333	1.11875	1.533	2.262	AC-FT=	.010
ROUTE	105.10	4	7	.06100	116.58	4.333	1.11875	1.567	2.249		
COMPUTE NM HYD	102.10	-	3	.02100	41.56	1.479	1.32081	1.500	3.092	PER IMP=	98.00
ROUTE RESERVOIR	105.01	3	4	.02100	27.88	1.479	1.32079	1.600	2.074	AC-FT=	.013
ROUTE	105.10	4	7	.02100	27.85	1.479	1.32082	1.733	2.074		
ADD HYD	234.50	3& 7	2	.10200	144.31	6.312	1.16034	1.567	2.211		
ROUTE	105.10	2	3	.10200	142.75	6.312	1.16035	1.567	2.187		
COMPUTE NM HYD	115.00	-	1	.02400	45.50	1.598	1.24868	1.500	2.962	PER IMP=	90.00
ROUTE RESERVOIR	105.01	1	4	.02400	24.28	1.598	1.24867	1.633	1.581	AC-FT=	.250
ROUTE	105.10	4	5	.02400	24.60	1.598	1.24869	1.433	1.601		
ADD HYD	234.50	3& 5	2	.12600	167.05	7.911	1.17717	1.567	2.072		
ROUTE	105.10	2	5	.12600	164.92	7.911	1.17717	1.600	2.045		
COMPUTE NM HYD	106.00	-	12	.02200	43.33	1.550	1.32081	1.500	3.092	PER IMP=	98.00
ROUTE RESERVOIR	105.01	12	8	.02200	43.47	1.550	1.32079	1.500	3.088	AC-FT=	.010
ROUTE	105.10	8	7	.02200	39.22	1.550	1.32081	1.533	2.786		
COMPUTE NM HYD	106.00	-	7	.01500	20.00	.928	1.15989	1.600	2.146	PER IMP=	50.00
ADD HYD	234.50	7& 9	9	.03700	58.22	2.478	1.25555	1.567	2.458		
ROUTE RESERVOIR	105.01	9	4	.03700	58.31	2.478	1.25555	1.567	2.463	AC-FT=	.010
ROUTE	105.10	4	2	.03700	57.55	2.478	1.25557	1.567	2.430		
ADD HYD	234.50	2& 5	1	.16300	222.23	10.388	1.19496	1.567	2.130		
ROUTE	105.10	1	10	.16300	221.25	10.388	1.19496	1.600	2.121		
COMPUTE NM HYD	103.00	-	4	.07700	83.90	3.235	.78703	1.667	1.297	PER IMP=	42.00
ROUTE RESERVOIR	105.01	4	1	.07700	83.91	3.234	.78762	1.667	1.297	AC-FT=	.009
ROUTE	105.10	1	3	.07700	81.69	3.235	.78762	1.700	1.252		
COMPUTE NM HYD	104.00	-	2	.01220	21.71	.742	1.14019	1.500	2.781	PER IMP=	77.00
ADD HYD	234.50	2& 3	1	.08920	72.09	3.976	.83584	1.667	1.263		
ROUTE RESERVOIR	105.01	1	2	.08920	72.09	3.976	.83584	1.667	1.263	AC-FT=	.009
ROUTE	105.10	2	3	.08920	72.07	3.976	.83584	1.667	1.262		
ADD HYD	234.50	10& 3	1	.25220	287.91	14.365	1.06794	1.600	1.784		
ROUTE	105.10	1	2	.25220	287.62	14.365	1.06794	1.600	1.782		
ROUTE	105.10	2	11	.25220	287.36	14.365	1.06794	1.600	1.780		
COMPUTE NM HYD	107.00	-	5	.03200	28.23	.960	.36263	1.567	1.379	PER IMP=	15.00
ROUTE	105.10	5	2	.03200	28.28	.960	.36263	1.567	1.381		
ADD HYD	234.50	11& 2	10	.28420	314.55	15.325	1.01104	1.600	1.729		
COMPUTE NM HYD	108.00	-	4	.02400	17.81	.458	.35791	1.533	1.160	PER IMP=	4.00
ADD HYD	234.50	10& 4	1	.30820	327.98	15.783	.96018	1.600	1.663		
COMPUTE NM HYD	110.10	-	15	.01000	6.46	.163	.30362	1.533	1.010	PER IMP=	2.00
ADD HYD	234.50	15& 1	2	.31820	332.87	15.946	.93961	1.600	1.635		
ROUTE	110.30	2	4	.31820	334.62	15.946	.93961	1.600	1.643		
COMPUTE NM HYD	109.00	-	2	.02700	12.58	.340	.23608	1.533	.728	PER IMP=	3.00
ADD HYD	234.50	4& 2	6	.34520	344.71	16.286	.88458	1.600	1.558		
COMPUTE NM HYD	110.00	-	15	.03000	15.80	.425	.26583	1.533	.823	PER IMP=	4.00
ADD HYD	111.00	15& 6	20	.37520	356.36	16.711	.83511	1.600	1.484		
ROUTE RESERVOIR	111.10	20	1	.37520	348.25	16.711	.83511	1.633	1.451	AC-FT=	1.421

APPENDIX C

SUMMARY PRINTOUTS
BASIN WE

AHYMO SUMMARY TABLE (AHYMO491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =08/21/1994

INPUT FILE = [redacted] USER NO.= MOLZCONM.I01

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	1.870
COMPUTE NM HYD	501.00	-	3	.04300	77.56	2.679	1.16824	1.500	2.813	PER IMP=	81.00
ROUTE RESERVOIR	105.01	3	4	.04300	13.21	2.679	1.16823	2.100	.480	AC-FT=	1.656
ROUTE	105.10	4	7	.04300	13.21	2.679	1.16822	2.766	.480		
COMPUTE NM HYD	502.00	-	3	.03800	51.20	2.154	1.06275	1.567	2.105	PER IMP=	69.00
ADD HYD	234.50	3& 7	2	.08100	61.20	4.833	1.11873	1.600	1.181		
ROUTE	105.10	2	7	.08100	61.42	4.833	1.11874	1.600	1.185		
COMPUTE NM HYD	503.00	-	5	.02100	41.56	1.479	1.32081	1.500	3.092	PER IMP=	98.00
ROUTE RESERVOIR	105.01	5	4	.02100	17.87	1.479	1.32079	1.700	1.329	AC-FT=	.552
ROUTE	105.10	4	3	.02100	17.86	1.479	1.32079	1.733	1.329		
ADD HYD	234.50	3& 7	2	.10200	78.32	6.312	1.16033	1.600	1.200		
ROUTE	105.10	2	3	.10200	77.92	6.312	1.16033	1.600	1.194		
COMPUTE NM HYD	504.00	-	1	.02200	43.53	1.550	1.32081	1.500	3.092	PER IMP=	98.00
ROUTE	110.30	1	4	.02200	31.50	1.550	1.32081	1.567	2.237		
COMPUTE NM HYD	505.00	-	1	.02400	45.50	1.598	1.24868	1.500	2.962	PER IMP=	90.00
ADD HYD	234.50	1& 4	1	.04600	72.60	3.148	1.28315	1.533	2.466		
ROUTE RESERVOIR	105.01	1	4	.04600	24.08	3.148	1.28315	2.000	.818	AC-FT=	1.408
ROUTE	105.10	4	5	.04600	24.08	3.148	1.28315	2.033	.818		
ADD HYD	234.50	3& 5	2	.14800	100.38	9.460	1.19850	1.633	1.060		
ROUTE	105.10	2	5	.14800	100.05	9.460	1.19850	1.633	1.056		
ROUTE	105.10	5	10	.14800	99.80	9.460	1.19850	1.633	1.054		
COMPUTE NM HYD	507.00	-	4	.02500	28.65	1.005	.75396	1.533	1.791	PER IMP=	32.00
ROUTE RESERVOIR	105.01	4	1	.02500	6.62	1.005	.75394	1.933	.414	AC-FT=	.593
ROUTE	105.10	1	3	.02500	6.61	1.005	.75390	2.000	.413		
COMPUTE NM HYD	508.00	-	4	.05200	47.19	2.330	.84010	1.667	1.418	PER IMP=	42.00
ROUTE RESERVOIR	105.01	4	1	.05200	37.12	2.330	.84009	1.800	1.115	AC-FT=	.476
ADD HYD	234.50	3& 1	1	.07700	43.41	3.335	.81210	1.800	.881		
ROUTE	105.10	1	3	.07700	42.76	3.335	.81210	1.833	.868		
COMPUTE NM HYD	509.00	-	2	.01220	21.71	.742	1.14019	1.500	2.781	PER IMP=	77.00
ADD HYD	234.50	2& 3	1	.08920	49.39	4.077	.85697	1.800	.865		
ROUTE RESERVOIR	105.01	1	2	.08920	49.38	4.077	.85697	1.800	.865	AC-FT=	.003
ROUTE	105.10	2	3	.08920	49.43	4.077	.85697	1.833	.866		
ADD HYD	234.50	10& 3	1	.23720	139.91	13.537	1.07006	1.700	.922		
ROUTE	105.10	1	2	.23720	139.91	13.537	1.07006	1.700	.922		
ROUTE	105.10	2	11	.23720	139.92	13.537	1.07006	1.700	.922		
COMPUTE NM HYD	506.00	-	5	.03200	28.23	.960	.56263	1.567	1.379	PER IMP=	15.00
ROUTE	105.10	5	2	.03200	28.28	.960	.56263	1.567	1.381		
ADD HYD	234.50	11& 2	10	.26920	159.48	14.497	1.00974	1.667	.926		
COMPUTE NM HYD	510.00	-	4	.02400	17.81	.458	.35791	1.533	1.160	PER IMP=	4.00
ADD HYD	234.50	10& 4	1	.29320	168.12	14.955	.95638	1.667	.896		
COMPUTE NM HYD	511.00	-	15	.01000	6.46	.163	.30562	1.533	1.010	PER IMP=	2.00
ADD HYD	234.50	15& 1	2	.30320	171.55	15.118	.93492	1.633	.884		
ROUTE	110.30	2	4	.30320	172.21	15.118	.93492	1.633	.887		
COMPUTE NM HYD	513.00	-	2	.02700	12.58	.340	.23608	1.533	.728	PER IMP=	3.00
ADD HYD	234.50	4& 2	6	.33020	180.16	15.458	.87778	1.633	.852		
COMPUTE NM HYD	512.00	-	15	.03000	15.80	.425	.26583	1.533	.823	PER IMP=	4.00
ADD HYD	111.00	15& 6	20	.36020	190.21	15.884	.82681	1.600	.825		
ROUTE RESERVOIR	111.10	20	1	.36020	149.30	15.884	.82681	1.866	.648	AC-FT=	2.206

AHYMO SUMMARY TABLE (AHYMO491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/21/1994

INPUT FILE = WEEX100.DA1 USER NO.= MOL2CONM.101

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	2.350
COMPUTE NM HYD	501.00	-	3	.04300	121.53	4.364	1.91155	1.500	4.418	PER IMP=	61.00
ROUTE RESERVOIR	105.01	3	4	.04300	13.22	4.364	1.91154	2.166	.480	AC-FT=	3.061
ROUTE	105.10	4	7	.04300	13.21	4.364	1.91153	2.700	.480		
COMPUTE NM HYD	502.00	-	3	.03600	82.37	3.617	1.78487	1.567	3.387	PER IMP=	69.00
ADD HYD	234.50	3& 7	2	.08100	94.09	8.001	1.85210	1.567	1.815		
ROUTE	105.10	2	7	.08100	93.44	8.001	1.85210	1.600	1.802		
COMPUTE NM HYD	503.00	-	5	.02100	63.45	2.347	2.09562	1.500	4.721	PER IMP=	98.00
ROUTE RESERVOIR	105.01	5	4	.02100	24.11	2.347	2.09560	1.766	1.794	AC-FT=	.926
ROUTE	105.10	4	3	.02100	24.09	2.347	2.09560	1.766	1.792		
ADD HYD	234.50	3& 7	2	.10200	115.51	10.348	1.90223	1.600	1.769		
ROUTE	105.10	2	3	.10200	115.46	10.348	1.90223	1.600	1.769		
COMPUTE NM HYD	504.00	-	1	.02200	66.47	2.459	2.09562	1.500	4.721	PER IMP=	98.00
ROUTE	110.30	1	4	.02200	51.28	2.459	2.09563	1.633	3.642		
COMPUTE NM HYD	505.00	-	1	.02400	70.31	2.571	2.00850	1.500	4.577	PER IMP=	90.00
ADD HYD	234.50	1& 4	1	.04600	107.77	5.030	2.05014	1.533	3.661		
ROUTE RESERVOIR	105.01	1	4	.04600	29.76	5.030	2.05014	2.100	1.011	AC-FT=	22.225
ROUTE	105.10	4	5	.04600	29.76	5.030	2.05014	2.100	1.011		
ADD HYD	234.50	3& 5	2	.14800	139.44	15.378	1.94819	1.600	1.472		
ROUTE	105.10	2	5	.14800	138.34	15.378	1.94820	1.600	1.460		
ROUTE	105.10	5	10	.14800	138.47	15.378	1.94820	1.633	1.462		
COMPUTE NM HYD	507.00	-	4	.02500	49.86	1.892	1.41895	1.533	3.116	PER IMP=	32.00
ROUTE RESERVOIR	105.01	4	1	.02500	10.01	1.892	1.41893	2.066	.625	AC-FT=	1.176
ROUTE	105.10	1	3	.02500	10.00	1.692	1.41889	2.233	.625		
COMPUTE NM HYD	508.00	-	4	.05200	60.74	4.221	1.52184	1.667	2.425	PER IMP=	42.00
ROUTE RESERVOIR	105.01	4	1	.05200	59.36	4.221	1.52183	1.833	1.784	AC-FT=	.907
ADD HYD	234.50	3& 1	1	.07700	69.07	6.112	1.48840	1.833	1.402		
ROUTE	105.10	1	3	.07700	68.12	6.112	1.48840	1.866	1.382		
COMPUTE NM HYD	509.00	-	2	.01220	34.17	1.223	1.87993	1.500	4.376	PER IMP=	77.00
ADD HYD	234.50	2& 3	1	.08920	78.70	7.336	1.54194	1.833	1.379		
ROUTE RESERVOIR	105.01	1	2	.08920	78.69	7.336	1.54194	1.833	1.378	AC-FT=	.005
ROUTE	105.10	2	3	.08920	78.61	7.336	1.54195	1.833	1.377		
ADD HYD	234.50	10& 3	1	.23720	202.20	22.713	1.79542	1.667	1.332		
ROUTE	105.10	1	2	.23720	202.31	22.713	1.79542	1.667	1.333		
ROUTE	105.10	2	11	.23720	202.29	22.713	1.79542	1.667	1.333		
COMPUTE NM HYD	506.00	-	5	.03200	53.36	2.008	1.17637	1.567	2.605	PER IMP=	15.00
ROUTE	105.10	5	2	.03200	53.36	2.008	1.17637	1.567	2.606		
ADD HYD	234.50	11& 2	10	.26920	246.62	24.721	1.72183	1.633	1.431		
COMPUTE NM HYD	510.00	-	4	.02400	39.89	1.147	.89572	1.500	2.597	PER IMP=	4.00
ADD HYD	234.50	10& 4	1	.29320	272.60	25.867	1.65421	1.600	1.453		
COMPUTE NM HYD	511.00	-	15	.01000	15.42	.437	.81964	1.500	2.410	PER IMP=	2.00
ADD HYD	234.50	15& 1	2	.30320	284.60	26.305	1.62668	1.600	1.467		
ROUTE	110.30	2	4	.30320	284.81	26.305	1.62669	1.600	1.468		
COMPUTE NM HYD	513.00	-	2	.02700	35.38	1.011	.70235	1.500	2.047	PER IMP=	3.00
ADD HYD	234.50	4& 2	6	.33020	312.85	27.316	1.55110	1.600	1.480		
COMPUTE NM HYD	512.00	-	15	.03000	41.55	1.194	.74599	1.500	2.164	PER IMP=	4.00
ADD HYD	111.00	15& 6	20	.36020	345.52	28.510	1.48405	1.600	1.499		
ROUTE RESERVOIR	111.10	20	1	.36020	256.34	28.510	1.48405	1.800	1.112	AC-FT=	3.817

AHYMO SUMMARY TABLE (AHYMO491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/21/1994

INPUT FILE = WEFU710.DAT USER NO.= MOLZOONM.101

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	1.570
COMPUTE NM HYD	101.00	-	3	.04300	77.56	2.679	1.16824	1.500	2.818	PER IMP=	81.00
ROUTE RESERVOIR	105.01	3	4	.04300	77.51	2.679	1.16824	1.500	2.817	AC-FT=	.009
ROUTE	105.10	4	7	.04300	69.87	2.679	1.16825	1.533	2.539		
COMPUTE NM HYD	102.00	-	3	.03800	51.20	2.154	1.06275	1.567	2.105	PER IMP=	69.00
ADD HYD	234.50	3& 7	2	.08100	118.31	4.833	1.11874	1.533	2.282		
ROUTE RESERVOIR	105.01	2	4	.08100	118.16	4.833	1.11875	1.533	2.279	AC-FT=	.010
ROUTE	105.10	4	7	.08100	116.53	4.833	1.11875	1.567	2.249		
COMPUTE NM HYD	102.10	-	5	.02100	41.56	1.479	1.32081	1.500	3.092	PER IMP=	99.00
ROUTE RESERVOIR	105.01	5	4	.02100	27.88	1.479	1.32079	1.600	2.074	AC-FT=	.131
ROUTE	105.10	4	3	.02100	27.88	1.479	1.32082	1.733	2.074		
ADD HYD	234.50	3& 7	2	.10200	144.31	6.312	1.16034	1.567	2.211		
ROUTE	105.10	2	3	.10200	142.75	6.312	1.16035	1.567	2.187		
COMPUTE NM HYD	105.00	-	1	.02400	45.50	1.598	1.24868	1.500	2.962	PER IMP=	90.00
ROUTE RESERVOIR	105.01	1	4	.02400	24.28	1.598	1.24867	1.633	1.581	AC-FT=	.250
ROUTE	105.10	4	5	.02400	24.60	1.598	1.24869	1.433	1.601		
ADD HYD	234.50	3& 5	2	.12600	167.05	7.911	1.17717	1.567	2.072		
ROUTE	105.10	2	5	.12600	164.92	7.911	1.17717	1.600	2.045		
COMPUTE NM HYD	106.00	-	12	.02200	43.53	1.550	1.32081	1.500	3.092	PER IMP=	98.00
ROUTE RESERVOIR	105.01	12	8	.02200	43.47	1.550	1.32079	1.500	3.088	AC-FT=	.010
ROUTE	105.10	8	7	.02200	39.22	1.550	1.32081	1.533	2.786		
COMPUTE NM HYD	106.00	-	3	.01500	20.60	.928	1.15989	1.600	2.146	PER IMP=	80.00
ADD HYD	234.50	7& 3	9	.03700	58.22	2.478	1.25555	1.567	2.458		
ROUTE RESERVOIR	105.01	9	4	.03700	58.31	2.478	1.25555	1.567	2.463	AC-FT=	.010
ROUTE	105.10	4	2	.03700	57.55	2.478	1.25557	1.567	2.430		
ADD HYD	234.50	2& 5	1	.16300	222.23	10.388	1.19496	1.567	2.130		
ROUTE	105.10	1	10	.16300	221.25	10.388	1.19496	1.600	2.121		
COMPUTE NM HYD	103.00	-	4	.07700	63.90	3.235	.78763	1.667	1.297	PER IMP=	42.00
ROUTE RESERVOIR	105.01	4	1	.07700	63.91	3.234	.78762	1.667	1.297	AC-FT=	.009
ROUTE	105.10	1	3	.07700	61.69	3.235	.78762	1.700	1.252		
COMPUTE NM HYD	104.00	-	2	.01220	21.71	.742	1.14019	1.500	2.781	PER IMP=	77.00
ADD HYD	234.50	2& 3	1	.08920	72.09	3.976	.83584	1.667	1.263		
ROUTE RESERVOIR	105.01	1	2	.08920	72.09	3.976	.83584	1.667	1.263	AC-FT=	.005
ROUTE	105.10	2	3	.08920	72.07	3.976	.83584	1.667	1.262		
ADD HYD	234.50	10& 3	1	.25220	287.91	14.365	1.06794	1.600	1.784		
ROUTE	105.10	1	2	.25220	287.62	14.365	1.06794	1.600	1.782		
ROUTE	105.10	2	11	.25220	287.36	14.365	1.06794	1.600	1.780		
COMPUTE NM HYD	107.00	-	5	.03200	28.23	.960	.56263	1.567	1.379	PER IMP=	15.00
ROUTE	105.10	5	2	.03200	28.28	.960	.56263	1.567	1.381		
ADD HYD	234.50	11& 2	10	.28420	314.55	15.325	1.01104	1.600	1.729		
COMPUTE NM HYD	108.00	-	4	.02400	17.81	.458	.35791	1.533	1.160	PER IMP=	4.00
ADD HYD	234.50	10& 4	1	.30820	327.98	15.783	.96018	1.600	1.663		
COMPUTE NM HYD	110.10	-	15	.01000	6.46	.163	.30562	1.533	1.010	PER IMP=	2.00
ADD HYD	234.50	15& 1	2	.31820	332.87	15.946	.93961	1.600	1.635		
ROUTE	110.30	2	4	.31820	334.62	15.946	.93961	1.600	1.643		
COMPUTE NM HYD	109.00	-	2	.02700	12.58	.340	.23608	1.533	.728	PER IMP=	3.00
ADD HYD	234.50	4& 2	6	.34520	344.31	16.286	.88458	1.600	1.558		
COMPUTE NM HYD	110.00	-	15	.03000	15.80	.425	.26583	1.533	.823	PER IMP=	4.00
ADD HYD	111.00	15& 6	20	.37520	356.36	16.711	.83511	1.600	1.484		
ROUTE RESERVOIR	111.10	20	1	.37520	348.25	16.711	.83511	1.633	1.450	AC-FT=	1.421

AHYMO SUMMARY TABLE (AHYMO491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/21/1994

INPUT FILE = WEFUTI00.DAT USER NO.= MOL,CDNM.I01

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	2.350
COMPUTE NM HYD	101.00	-	3	.04300	121.38	4.334	1.91155	1.500	4.418	PER IMP=	81.00
ROUTE RESERVOIR	105.01	3	4	.04300	91.73	4.384	1.91155	1.600	2.970	AC-FT=	.368
ROUTE	105.10	4	7	.04300	81.13	4.384	1.91156	1.766	2.948		
COMPUTE NM HYD	102.00	-	3	.03800	82.37	3.617	1.78487	1.567	3.387	PER IMP=	69.00
ADD HYD	234.50	3& 7	2	.08100	161.69	8.001	1.85211	1.567	3.119		
ROUTE RESERVOIR	105.01	2	4	.08100	119.64	8.001	1.85211	1.766	2.308	AC-FT=	.720
ROUTE	105.10	4	7	.08100	119.59	8.001	1.85212	2.000	2.307		
COMPUTE NM HYD	102.10	-	5	.02100	63.45	2.347	2.09562	1.500	4.721	PER IMP=	98.00
ROUTE RESERVOIR	105.01	5	4	.02100	27.88	2.347	2.09560	1.700	2.075	AC-FT=	.496
ROUTE	105.10	4	3	.02100	27.88	2.347	2.09563	1.866	2.075		
ADD HYD	234.50	3& 7	2	.10200	147.48	10.348	1.90224	2.000	2.259		
ROUTE	105.10	2	3	.10200	147.48	10.348	1.90225	2.000	2.259		
COMPUTE NM HYD	105.00	-	1	.02400	70.31	2.571	2.00850	1.500	4.577	PER IMP=	90.00
ROUTE RESERVOIR	105.01	1	4	.02400	24.29	2.571	2.00849	1.800	1.581	AC-FT=	.765
ROUTE	105.10	4	5	.02400	24.30	2.571	2.00851	1.400	1.582		
ADD HYD	234.50	3& 5	2	.12600	171.76	12.919	1.92248	2.000	2.130		
ROUTE	105.10	2	5	.12600	171.77	12.919	1.92248	2.000	2.130		
COMPUTE NM HYD	106.00	-	12	.02200	66.47	2.459	2.09562	1.500	4.721	PER IMP=	98.00
ROUTE RESERVOIR	105.01	12	8	.02200	43.48	2.459	2.09560	1.600	3.088	AC-FT=	.223
ROUTE	105.10	8	7	.02200	43.43	2.459	2.09563	1.800	3.084		
COMPUTE NM HYD	106.00	-	3	.01500	32.50	1.521	1.90164	1.600	3.385	PER IMP=	80.00
ADD HYD	234.50	7& 3	9	.03700	75.41	3.980	2.01696	1.600	3.184		
ROUTE RESERVOIR	105.01	9	4	.03700	58.97	3.980	2.01696	1.800	2.490	AC-FT=	.299
ROUTE	105.10	4	2	.03700	58.97	3.980	2.01697	1.800	2.490		
ADD HYD	234.50	2& 5	1	.16300	230.71	16.899	1.94393	1.966	2.212		
ROUTE	105.10	1	10	.16300	230.70	16.899	1.94393	1.966	2.211		
COMPUTE NM HYD	103.00	-	4	.07700	112.71	5.923	1.44222	1.667	2.287	PER IMP=	42.00
ROUTE RESERVOIR	105.01	4	1	.07700	69.09	5.923	1.44221	1.900	1.402	AC-FT=	.856
ROUTE	105.10	1	3	.07700	69.02	5.923	1.44222	2.266	1.401		
COMPUTE NM HYD	104.00	-	2	.01220	34.17	1.223	1.87993	1.500	4.376	PER IMP=	77.00
ADD HYD	234.50	2& 3	1	.08920	94.65	7.146	1.50208	1.533	1.658		
ROUTE RESERVOIR	105.01	1	2	.08920	94.50	7.146	1.50207	1.533	1.655	AC-FT=	.006
ROUTE	105.10	2	3	.08920	95.23	7.146	1.50208	1.567	1.668		
ADD HYD	234.50	10& 3	1	.25220	323.49	24.045	1.78765	1.567	2.004		
ROUTE	105.10	1	2	.25220	323.84	24.045	1.78765	1.567	2.006		
ROUTE	105.10	2	11	.25220	323.97	24.045	1.78765	1.567	2.007		
COMPUTE NM HYD	107.00	-	5	.03200	53.36	2.008	1.17637	1.567	2.605	PER IMP=	15.00
ROUTE	105.10	5	2	.03200	53.36	2.008	1.17637	1.567	2.606		
ADD HYD	234.50	11& 2	10	.28420	377.33	26.053	1.71882	1.567	2.075		
COMPUTE NM HYD	108.00	-	4	.02400	39.89	1.147	.89572	1.500	2.597	PER IMP=	4.00
ADD HYD	234.50	10& 4	1	.30820	412.70	27.199	1.65472	1.567	2.092		
COMPUTE NM HYD	110.10	-	15	.01000	15.42	.437	.81964	1.500	2.410	PER IMP=	2.00
ADD HYD	234.50	15& 1	2	.31820	426.54	27.636	1.62848	1.567	2.095		
ROUTE	110.30	2	4	.31820	428.56	27.636	1.62848	1.567	2.104		
COMPUTE NM HYD	109.00	-	2	.02700	35.38	1.011	.70235	1.500	2.047	PER IMP=	3.00
ADD HYD	234.50	4& 2	6	.34520	460.81	28.648	1.55604	1.567	2.086		
COMPUTE NM HYD	110.00	-	15	.03000	41.55	1.194	.74599	1.500	2.164	PER IMP=	4.00
ADD HYD	111.00	15& 6	20	.37520	498.45	29.841	1.49127	1.567	2.076		
ROUTE RESERVOIR	111.10	20	1	.37520	408.22	29.841	1.49127	1.700	1.700	AC-FT=	3.663
ROUTE	111.10	1	4	.37520	400.82	29.841	1.49127	1.766	1.669		
COMPUTE NM HYD	111.00	-	5	.04700	58.22	1.653	.65929	1.500	1.936	PER IMP=	2.00
ADD HYD	111.20	4& 5	1	.42220	418.25	31.494	1.39865	1.766	1.548		
ROUTE	111.10	1	4	.42220	416.66	31.494	1.39865	1.800	1.542		
COMPUTE NM HYD	111.30	-	5	.03000	39.62	1.157	.72303	1.500	2.064	PER IMP=	6.00
ADD HYD	111.40	4& 5	1	.45220	428.01	32.651	1.35383	1.766	1.479		

COMPUTE NM HYD	112.10	-	5	.00300	6.70	.209	1.30565	1.500	3.489 PER IMP=	10.00
ADD HYD	112.20	4& 5	1	.45520	430.35	32.960	1.35351	1.766	1.477	
ROUTE	113.00	1	4	.45520	430.11	32.860	1.35351	1.766	1.476	
COMPUTE NM HYD	113.10	-	5	.00700	9.98	.298	.79911	1.500	2.229 PER IMP=	10.00
ADD HYD	113.30	4& 5	1	.46220	433.25	33.158	1.34511	1.766	1.465	
ROUTE	113.00	1	4	.46220	433.08	33.159	1.34511	1.766	1.464	
ROUTE	114.00	4	2	.46220	433.45	33.158	1.34511	1.800	1.465	
COMPUTE NM HYD	114.10	-	5	.01200	15.98	.460	.71830	1.500	2.080 PER IMP=	4.00
ADD HYD	113.30	2& 5	1	.47420	437.39	33.618	1.32925	1.800	1.441	

AHYMO SUMMARY TABLE (AHYMO491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/20/1994

INPUT FILE = SESEX10.DAT USER NO.= MOLZCONM.101

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION
RAINFALL TYPE= 1									RAIN6=	1.570
COMPUTE NM HYD	201.00	-	2	.01590	7.12	.105	.12345	1.467	.700 PER IMP=	.00
ROUTE	201.10	2	7	.01590	7.63	.105	.12349	1.467	.750	
COMPUTE NM HYD	202.00	-	3	.02520	12.30	.166	.12345	1.467	.763 PER IMP=	.00
ROUTE	202.10	3	6	.02520	13.32	.166	.12347	1.467	.826	
RECALL HYD	102.00	-	2	.25250	68.40	8.807	.65400	1.866	.423	
COMPUTE NM HYD	203.00	-	3	.06310	18.88	.415	.12345	1.500	.467 PER IMP=	.00
ADD HYD	1803.10	2& 3	1	.31560	70.31	9.223	.54793	1.833	.348	
ROUTE	203.10	1	8	.31560	70.30	9.223	.54794	1.833	.348	
COMPUTE NM HYD	204.00	-	3	.05210	17.30	.343	.12345	1.500	.519 PER IMP=	.00
ROUTE	204.10	3	9	.05210	17.51	.343	.12346	1.467	.525	
COMPUTE NM HYD	205.00	-	2	.02190	15.83	.144	.12345	1.433	1.129 PER IMP=	.00
ROUTE	205.10	2	10	.02190	14.75	.144	.12348	1.467	1.052	

APPENDIX C

SUMMARY PRINTOUTS
BASIN SE35

HYDROLOGY SUMMARY TABLE (AH/MOD91) - HMHFSH VERSION OF H/MD - APRIL, 1991 RUN DATE (MON/DAY/YR) = 6/21/1994
INPUT FILE = G35EX10.DAT USER NO.= HSEIECNM.IXX

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	1.87
COMPUTE NM HYD	s03.00	-	3	.00970	14.78	.464	.89595	1.500	2.381	PER IMP=	47.00
ROUTE RESERVOIR	503.00	3	4	.00970	5.28	.464	.89595	1.733	.850	AC-FT=	.148
ROUTE	s03.10	4	7	.00970	5.27	.464	.89600	1.800	.849		
COMPUTE NM HYD	s02.00	-	3	.01240	19.00	.615	.92930	1.500	2.436	PER IMP=	31.00
ROUTE RESERVOIR	502.00	3	4	.01240	15.48	.615	.92928	1.567	1.951	AC-FT=	.130
ADD HYD	s02.10	4& 7	2	.02210	20.68	1.078	.91465	1.567	1.462		
ROUTE RESERVOIR	501.00	2	4	.02210	13.80	1.078	.91465	1.800	.976	AC-FT=	.277
ROUTE	s02.20	4	7	.02210	13.79	1.078	.91466	1.900	.974		
COMPUTE NM HYD	s01.00	-	3	.01020	16.45	.533	.97929	1.500	2.523	PER IMP=	37.00
ROUTE RESERVOIR	501.00	3	4	.01020	10.95	.533	.97925	1.600	1.682	AC-FT=	.114
ADD HYD	s01.10	4& 7	2	.03230	33.39	1.611	.93504	1.667	1.612		
ROUTE	s01.20	2	7	.03230	33.33	1.611	.93505	1.700	1.126		
COMPUTE NM HYD	s04.00	-	3	.02290	21.04	.915	.71622	1.633	1.376	PER IMP=	29.00
ROUTE RESERVOIR	504.00	3	4	.02290	12.81	.915	.71620	1.833	.838	AC-FT=	.226
ADD HYD	s04.10	4& 7	2	.05620	35.81	2.526	.84282	1.700	.996		
ROUTE	601.21	2	5	.05620	35.46	2.526	.84283	1.732	.986		
COMPUTE NM HYD	s12.00	-	3	.03020	26.14	.746	.46311	1.500	1.352	PER IMP=	15.00
ROUTE RESERVOIR	512.00	3	4	.03020	24.32	.746	.46310	1.567	1.258	AC-FT=	.044
ADD HYD	s13.10	5& 2	6	.11240	71.16	5.052	.84282	1.732	.989		
COMPUTE NM HYD	s05.00	-	3	.02440	31.76	1.090	.83767	1.533	2.034	PER IMP=	40.00
ROUTE RESERVOIR	505.00	3	4	.02440	10.62	1.090	.83766	1.833	.680	AC-FT=	.524
ROUTE	s05.20	4	5	.02440	10.59	1.090	.83765	1.865	.678		
ADD HYD	s05.10	5& 6	2	.13680	81.25	6.142	.84189	1.732	.929		
ROUTE RESERVOIR	501.00	2	4	.13680	51.23	6.142	.84189	2.198	.585	AC-FT=	1.471
ROUTE	s05.20	4	7	.13680	51.21	6.142	.84190	2.300	.585		
COMPUTE NM HYD	s06.00	-	3	.04440	12.80	.806	.34034	1.733	.450	PER IMP=	11.00
ROUTE RESERVOIR	506.10	3	4	.04440	12.30	.806	.34034	1.800	.433	AC-FT=	.007
ROUTE	s06.10	4	5	.04440	12.23	.806	.34036	1.833	.430		
ADD HYD	s06.20	5& 7	2	.18120	s3.00	6.948	.71900	1.866	.543		
ROUTE RESERVOIR	501.00	2	4	.18120	51.12	6.948	.71900	2.733	.441	AC-FT=	.958
ROUTE	s05.30	4	7	.18120	51.12	6.948	.71900	2.766	.441		
COMPUTE NM HYD	s08.00	-	3	.00550	8.43	.265	.90431	1.500	2.396	PER IMP=	48.00
ROUTE RESERVOIR	508.00	3	4	.00550	7.22	.265	.90426	1.567	2.052	AC-FT=	.027
ADD HYD	s08.10	4& 7	2	.16670	53.17	7.214	.72445	1.833	.445		
COMPUTE NM HYD	s10.00	-	3	.02630	9.40	.684	.48786	1.866	.558	PER IMP=	12.00
ROUTE RESERVOIR	510.00	3	4	.02630	9.40	.684	.48786	1.866	.559	AC-FT=	.006
ADD HYD	610.10	2& 4	5	.21300	s2.52	7.898	.69524	1.833	.459		
ROUTE	610.20	5	10	.21300	61.69	7.898	.69524	1.933	.452		
COMPUTE NM HYD	607.00	-	3	.00750	11.63	.368	.92097	1.500	2.423	PER IMP=	50.00
ROUTE RESERVOIR	507.00	3	4	.00750	11.48	.368	.92094	1.500	2.391	AC-FT=	.306
ROUTE	s07.10	4	7	.00750	11.29	.368	.92100	1.533	2.352		
COMPUTE NM HYD	s09.00	-	3	.02080	9.66	.289	.26045	1.500	.726	PER IMP=	8.00
ROUTE RESERVOIR	509.00	3	4	.02080	8.88	.289	.26043	1.567	.667	AC-FT=	.025
ADD HYD	s09.10	4& 7	2	.02930	20.01	.657	.43548	1.533	1.105		
ROUTE	609.20	2	7	.02930	19.93	.657	.43549	1.533	1.100		
ADD HYD	s09.30	10& 7	9	.24130	66.50	8.535	.66477	1.866	.431		
COMPUTE NM HYD	611.00	-	3	.01120	7.99	.252	.42199	1.500	1.115	PER IMP=	20.00
ADD HYD	611.10	3& 9	2	.25250	68.40	8.307	.65400	1.866	.423		

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME AC-FT.	RUNOFF INCHES	TIME TO PEAK HOURS	CFS PER SQMI	NOTATION	
RAINFALL TYPE= 1										RAIN=	2.331
COMPUTE NM HYD	100.00	-	1	.00970	24.41	.324	1.59201	1.510	3.732	PER IMP=	47.00
ROUTE RESERVOIR	100.01	1	4	.00970	5.24	.324	1.59198	1.750	.845	AC-FT=	.441
ROUTE	100.11	4	7	.00970	5.24	.324	1.59200	2.133	.345		
COMPUTE NM HYD	200.00	-	1	.01240	31.64	1.173	1.62151	1.510	3.732	PER IMP=	81.00
ROUTE RESERVOIR	200.01	1	4	.01240	22.39	1.173	1.62147	1.500	2.321	AC-FT=	.171
ADD HYD	200.11	4& 7	2	.02221	27.82	1.497	1.62141	1.500	1.946		
ROUTE RESERVOIR	201.01	2	4	.02221	17.84	1.497	1.62141	2.000	.973	AC-FT=	.324
ROUTE	200.21	4	7	.02221	17.84	1.497	1.62141	2.100	.973		
COMPUTE NM HYD	301.00	-	1	.01020	25.83	.920	1.59076	1.500	4.037	PER IMP=	67.00
ROUTE RESERVOIR	301.10	1	4	.01020	13.14	.920	1.59077	1.667	2.012	AC-FT=	.268
ADD HYD	300.10	4& 7	2	.03220	25.83	2.322	1.62330	1.733	1.298		
ROUTE	300.20	2	7	.03220	26.78	2.322	1.62332	1.766	1.295		
COMPUTE NM HYD	304.00	-	1	.02290	37.42	1.750	1.57310	1.633	2.447	PER IMP=	29.00
ROUTE RESERVOIR	304.10	1	4	.02290	17.64	1.750	1.57308	2.000	.892	AC-FT=	.742
ADD HYD	304.10	4& 7	2	.05520	40.07	4.572	1.62551	1.833	1.114		
ROUTE	301.21	2	5	.05520	40.04	4.572	1.62551	1.865	1.113		
COMPUTE NM HYD	313.00	-	1	.02120	24.25	1.649	1.62408	1.500	2.307	PER IMP=	15.00
ROUTE RESERVOIR	313.00	1	4	.02020	18.67	1.649	1.62407	1.600	2.331	AC-FT=	.215
ADD HYD	313.10	5& 2	6	.11240	80.10	9.145	1.62550	1.832	1.114		
COMPUTE NM HYD	303.00	-	1	.02440	32.45	1.982	1.62289	1.533	2.402	PER IMP=	40.00
ROUTE RESERVOIR	315.00	1	4	.02440	12.97	1.982	1.62287	2.000	.831	AC-FT=	1.070
ROUTE	605.20	4	5	.02440	12.96	1.982	1.62287	2.031	.830		
ADD HYD	305.10	5& 6	2	.13680	92.96	11.127	1.62503	1.865	1.062		
ROUTE RESERVOIR	301.00	2	4	.13680	51.22	11.127	1.62503	2.831	.585	AC-FT=	1.337
ROUTE	605.20	4	7	.13680	51.22	11.127	1.62503	2.866	.585		
COMPUTE NM HYD	310.00	-	1	.04440	31.50	1.939	.83998	1.733	1.102	PER IMP=	11.00
ROUTE RESERVOIR	306.00	1	4	.04440	13.01	1.989	.83997	2.200	.458	AC-FT=	.220
ROUTE	306.10	4	5	.04440	13.01	1.989	.83999	2.266	.458		
ADD HYD	306.20	5& 7	2	.18120	84.19	13.116	1.35716	2.000	.554		
ROUTE RESERVOIR	301.00	2	4	.18120	51.29	13.116	1.35716	2.733	.442	AC-FT=	1.932
ROUTE	606.30	4	7	.18120	51.29	13.116	1.35717	2.700	.442		
COMPUTE NM HYD	608.00	-	1	.00550	13.90	.470	1.60189	1.500	3.950	PER IMP=	48.00
ROUTE RESERVOIR	508.20	1	4	.00550	11.85	.470	1.60184	1.567	3.367	AC-FT=	.032
ADD HYD	606.10	4& 7	2	.16670	54.75	13.585	1.36437	1.900	.458		
COMPUTE NM HYD	610.00	-	1	.02630	20.05	1.502	1.07117	1.366	1.191	PER IMP=	12.00
ROUTE RESERVOIR	510.00	1	4	.02630	13.30	1.502	1.07117	2.000	1.087	AC-FT=	.059
ADD HYD	610.10	2& 4	5	.21300	72.70	15.088	1.32817	1.933	.533		
ROUTE	610.20	5	10	.21300	71.35	15.088	1.32917	2.033	.525		
COMPUTE NM HYD	607.00	-	1	.00750	19.10	.649	1.62163	1.500	3.979	PER IMP=	50.00
ROUTE RESERVOIR	507.00	1	4	.00750	14.23	.649	1.62160	1.600	2.964	AC-FT=	.054
ROUTE	607.10	4	7	.00750	14.22	.649	1.62166	1.600	2.963		
COMPUTE NM HYD	609.00	-	1	.02080	27.07	.804	.72458	1.500	2.033	PER IMP=	3.00
ROUTE RESERVOIR	509.00	1	4	.02080	24.01	.804	.72457	1.567	1.803	AC-FT=	.099
ADD HYD	609.10	4& 7	2	.02830	38.11	1.452	.96230	1.567	2.104		
ROUTE	609.20	2	7	.02830	37.95	1.452	.96231	1.600	2.095		
ADD HYD	609.30	10& 7	9	.24130	79.75	16.540	1.23526	1.933	.516		
COMPUTE NM HYD	611.00	-	1	.01120	17.83	.561	.93877	1.500	2.487	PER IMP=	20.00
ADD HYD	611.10	3& 9	2	.25250	82.87	17.101	1.26989	1.933	.513		

APPENDIX C

SUMMARY PRINTOUTS
BASIN GA

AHYMO SUMMARY TABLE (AHYM0491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/17/1994

INPUT FILE = [redacted] USER NO.= MOLICONM.I01

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	1.570
COMPUTE NM HYD	701.00	-	3	.00420	6.78	.219	.97928	1.500	2.522	PER IMP=	57.00
ROUTE RESERVOIR	501.00	3	4	.00420	5.56	.219	.97922	1.567	2.070	AC-FT=	.029
ROUTE	701.10	4	7	.00420	5.46	.219	.97934	1.600	2.032		
COMPUTE NM HYD	702.00	-	3	.00450	7.59	.251	1.04592	1.500	2.634	PER IMP=	65.00
ROUTE RESERVOIR	502.00	3	4	.00450	5.45	.251	1.04586	1.600	1.892	AC-FT=	.032
ADD HYD	702.10	4& 7	2	.00870	10.91	.470	1.01368	1.600	1.960		
ROUTE	702.20	2	7	.00870	10.87	.470	1.01374	1.600	1.952		
COMPUTE NM HYD	708.00	-	3	.00540	8.38	.265	.92097	1.500	2.424	PER IMP=	50.00
ROUTE RESERVOIR	508.00	3	4	.00540	4.11	.265	.92091	1.667	1.189	AC-FT=	.095
ADD HYD	708.10	4& 7	2	.01410	14.85	.736	.97813	1.633	1.645		
ROUTE	708.20	2	7	.01410	14.85	.736	.97816	1.633	1.646		
COMPUTE NM HYD	707.00	-	3	.00850	11.66	.342	.75437	1.500	2.144	PER IMP=	30.00
ROUTE RESERVOIR	507.00	3	4	.00850	4.35	.342	.75434	1.733	.800	AC-FT=	.156
ADD HYD	707.10	4& 7	2	.02260	19.07	1.077	.89393	1.633	1.318		
ROUTE	707.20	2	7	.02260	18.59	1.078	.89395	1.700	1.286		
COMPUTE NM HYD	711.00	-	3	.02870	57.30	2.047	1.33747	1.500	3.119	PER IMP=	100.00
ROUTE RESERVOIR	511.00	3	4	.02870	37.09	2.047	1.33746	1.600	2.019	AC-FT=	.393
ADD HYD	711.10	4& 7	2	.05130	55.35	3.125	1.14206	1.667	1.686		
ROUTE	711.20	2	7	.05130	54.79	3.125	1.14207	1.733	1.669		
COMPUTE NM HYD	709.00	-	3	.01400	12.13	.688	.92097	1.733	1.354	PER IMP=	50.00
ROUTE RESERVOIR	509.00	3	4	.01400	9.55	.688	.92092	1.900	1.066	AC-FT=	.118
ROUTE	709.10	4	6	.01400	9.55	.688	.92095	1.900	1.065		
ADD HYD	709.20	6& 7	2	.06530	63.21	3.812	1.09465	1.766	1.512		
COMPUTE NM HYD	713.00	-	3	.00460	6.52	.195	.79602	1.500	2.215	PER IMP=	35.00
ROUTE RESERVOIR	513.00	3	4	.00460	5.28	.195	.79596	1.567	1.793	AC-FT=	.021
ADD HYD	713.10	2& 4	5	.06990	66.46	4.008	1.07499	1.700	1.486		
ROUTE	713.20	5	7	.06990	65.71	4.008	1.07500	1.766	1.469		
COMPUTE NM HYD	730.00	-	3	.01120	14.79	.627	1.04942	1.567	2.063	PER IMP=	70.00
COMPUTE NM HYD	714.00	-	6	.02320	36.99	1.191	.96262	1.500	2.492	PER IMP=	55.00
ADD HYD	730.10	3& 6	4	.03440	49.18	1.818	.99086	1.533	2.234		
ROUTE RESERVOIR	530.00	4	2	.03440	17.88	1.818	.99086	1.800	.812	AC-FT=	.903
ADD HYD	730.20	2& 7	5	.10430	83.47	5.825	1.04724	1.766	1.250		
ROUTE	730.30	5	7	.10430	83.25	5.825	1.04725	1.766	1.247		
COMPUTE NM HYD	712.00	-	3	.02160	42.22	1.504	1.30561	1.500	3.054	PER IMP=	97.00
ROUTE RESERVOIR	512.00	3	4	.02160	30.66	1.504	1.30559	1.600	2.218	AC-FT=	.231
ROUTE	712.10	4	6	.02160	30.62	1.504	1.30562	1.600	2.215		
ADD HYD	712.20	6& 7	2	.12590	107.56	7.330	1.09157	1.733	1.335		
ROUTE	712.30	2	7	.12590	107.30	7.330	1.09157	1.766	1.332		
COMPUTE NM HYD	716.00	-	3	.00940	14.99	.483	.96262	1.500	2.492	PER IMP=	55.00
ADD HYD	716.10	3& 7	6	.13530	112.63	7.812	1.08261	1.733	1.301		
ROUTE	716.20	6	8	.13530	112.43	7.812	1.08261	1.766	1.298		
COMPUTE NM HYD	704.00	-	3	.01070	15.33	.549	.96262	1.533	2.238	PER IMP=	55.00
ROUTE RESERVOIR	504.00	3	4	.01070	8.66	.549	.96259	1.700	1.264	AC-FT=	.151
ROUTE	704.10	4	7	.01070	8.54	.549	.96263	1.733	1.248		
COMPUTE NM HYD	703.00	-	3	.00790	12.60	.406	.96262	1.500	2.493	PER IMP=	55.00
ROUTE RESERVOIR	503.00	3	4	.00790	4.29	.406	.96258	1.766	.849	AC-FT=	.191
ADD HYD	703.10	4& 7	2	.01860	12.83	.955	.96254	1.733	1.078		
ROUTE	703.20	2	7	.01860	12.80	.955	.96257	1.766	1.075		
COMPUTE NM HYD	729.00	-	3	.01400	21.70	.688	.92097	1.500	2.422	PER IMP=	50.00
ROUTE RESERVOIR	509.00	3	4	.01400	5.19	.688	.92095	1.866	.579	AC-FT=	.365
ADD HYD	709.10	4& 7	2	.03260	17.93	1.642	.94466	1.766	.859		
ROUTE	709.20	2	7	.03260	17.77	1.642	.94468	1.800	.852		
COMPUTE NM HYD	710.00	-	3	.03120	52.03	2.173	1.30561	1.533	2.606	PER IMP=	97.00
ADD HYD	710.10	3& 7	2	.06380	63.43	3.815	1.12117	1.567	1.553		
COMPUTE NM HYD	705.00	-	3	.01620	20.48	.565	.65442	1.500	1.975	PER IMP=	18.00

ROUTE	705.10	4	7	.01620	6.12	.565	.65441	1.766	.591		
COMPUTE NM HYD	706.00	-	3	.02250	29.44	.835	.69606	1.500	2.045	PER IMP=	23.00
ROUTE RESERVOIR	506.00	3	4	.02250	6.31	.833	.69605	1.833	.439	AC-FT=	.474
ADD HYD	706.10	4& 7	5	.03870	12.40	1.401	.67860	1.766	.501		
ROUTE	706.20	5	7	.03870	12.39	1.401	.67861	1.800	.500		
ADD HYD	710.20	2& 7	6	.10250	73.10	5.216	.95407	1.567	1.114		
ROUTE	710.30	6	7	.10250	71.81	5.216	.95407	1.600	1.095		
COMPUTE NM HYD	715.00	-	3	.00940	13.32	.399	.79602	1.500	2.213	PER IMP=	35.00
ROUTE RESERVOIR	515.00	3	4	.00940	3.71	.399	.79599	1.800	.616	AC-FT=	.230
ADD HYD	715.10	4& 7	2	.11190	75.01	5.615	.94079	1.600	1.047		
ROUTE	715.20	2	7	.11190	74.12	5.615	.94079	1.600	1.035		
COMPUTE NM HYD	713.00	-	3	.01520	23.56	.747	.92097	1.500	2.422	PER IMP=	50.00
ADD HYD	718.20	3& 7	2	.12710	93.23	6.361	.93842	1.600	1.107		
ROUTE RESERVOIR	518.00	2	4	.12710	54.70	6.361	.93842	2.000	.672	AC-FT=	1.617
ADD HYD	721.10	4& 8	9	.26240	162.97	14.173	1.01276	1.800	.970		
COMPUTE NM HYD	719.00	-	3	.02090	18.26	.468	.41948	1.500	1.365	PER IMP=	5.00
ROUTE RESERVOIR	519.00	3	4	.02090	8.19	.468	.41947	1.667	.606	AC-FT=	.247
ROUTE	719.10	4	7	.02090	6.40	.468	.41947	1.833	.479		
COMPUTE NM HYD	722.00	-	3	.02100	8.35	.458	.40892	1.667	.622	PER IMP=	10.00
ADD HYD	722.10	3& 7	2	.04190	14.00	.925	.41415	1.733	.522		
COMPUTE NM HYD	723.00	-	3	.01630	6.69	.355	.40892	1.667	.641	PER IMP=	10.00
ADD HYD	723.10	2& 3	5	.05820	20.38	1.281	.41267	1.733	.547		
ROUTE	723.20	5	7	.05820	20.39	1.281	.41266	1.733	.547		
ADD HYD	723.30	9& 7	2	.32060	182.17	15.454	.90383	1.800	.888		
ROUTE	723.40	2	6	.32060	179.05	15.454	.90383	1.833	.873		
COMPUTE NM HYD	721.00	-	3	.03580	13.61	.402	.21042	1.533	.594	PER IMP=	5.00
ADD HYD	721.20	3& 6	6	.35640	182.26	15.856	.83417	1.833	.799		
COMPUTE NM HYD	717.00	-	3	.01590	19.10	.631	.74469	1.500	1.877	PER IMP=	45.00
ROUTE	717.10	3	4	.01590	14.95	.632	.74470	1.567	1.469		
COMPUTE NM HYD	720.00	-	3	.03540	13.46	.397	.21042	1.533	.594	PER IMP=	5.00
ADD HYD	720.10	3& 4	2	.05130	27.82	1.029	.37600	1.533	.833		
ADD HYD	720.20	2& 6	5	.40770	173.01	16.869	.77452	1.833	.740		
COMPUTE NM HYD	724.00	-	3	.04650	20.75	.592	.23872	1.533	.697	PER IMP=	5.00
ADD HYD	721.30	3& 5	2	.45420	198.09	17.477	.72146	1.800	.681		
ROUTE RESERVOIR	521.00	2	4	.45420	175.70	17.477	.72145	1.966	.604	AC-FT=	1.282
ROUTE	721.40	4	7	.45420	175.55	17.477	.72146	2.000	.604		
COMPUTE NM HYD	726.10	-	3	.02420	12.67	.755	.27536	1.500	.779	PER IMP=	5.00
ADD HYD	726.11	3& 7	2	.47840	177.42	17.832	.67870	1.766	.679		
ROUTE RESERVOIR	521.10	2	6	.47840	175.17	17.832	.69890	2.065	.872	AC-FT=	.681
COMPUTE NM HYD	725.00	-	3	.02380	35.27	1.215	.95712	1.500	2.317	PER IMP=	65.00
ROUTE RESERVOIR	525.00	3	4	.02380	16.67	1.215	.95711	1.567	1.098	AC-FT=	.347
ROUTE	725.10	4	7	.02380	16.31	1.215	.95713	1.766	1.071		
ADD HYD	725.20	5& 7	2	.50220	189.34	19.047	.71113	2.033	.559		
ROUTE	725.30	2	7	.50220	189.20	19.047	.71113	2.066	.589		
COMPUTE NM HYD	726.00	-	3	.15530	85.80	2.355	.28437	1.533	.863	PER IMP=	.00
ADD HYD	726.10	3& 7	2	.65750	196.46	21.402	.61033	2.033	.467		
ROUTE RESERVOIR	526.00	2	4	.65750	196.18	21.402	.61033	2.066	.466	AC-FT=	.447
ROUTE	726.20	4	5	.65750	195.92	21.402	.61033	2.100	.466		
COMPUTE NM HYD	728.00	-	3	.03440	20.24	.634	.34554	1.500	.917	PER IMP=	15.00
ADD HYD	728.10	3& 5	6	.69190	198.79	22.036	.59717	2.066	.449		
COMPUTE NM HYD	727.00	-	3	.05270	24.88	.971	.34554	1.567	.738	PER IMP=	15.00
ROUTE RESERVOIR	527.00	3	4	.05270	24.22	.971	.34553	1.600	.718	AC-FT=	.051
ADD HYD	727.10	4& 6	2	.74460	207.03	23.007	.57936	1.733	.434		

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAINo=	2.331
COMPUTE NM HYD	701.10	-	3	.00420	11.10	.279	1.89703	1.500	4.370	PER IMP=	97.00
ROUTE RESERVOIR	501.00	3	4	.00421	8.99	.279	1.89069	1.867	3.343	AC-FT=	.185
ROUTE	701.11	4	7	.00420	8.77	.279	1.89079	1.500	3.284		
COMPUTE NM HYD	702.10	-	3	.00450	12.14	.425	1.76975	1.500	4.214	PER IMP=	68.00
ROUTE RESERVOIR	502.11	3	4	.00450	9.89	.425	1.75959	1.833	3.731	AC-FT=	.197
ADD HYD	702.10	4& 7	2	.00870	18.52	.803	1.77154	1.500	3.797		
ROUTE	702.11	2	7	.00870	15.33	.803	1.77150	1.500	2.754		
COMPUTE NM HYD	703.00	-	3	.00540	13.76	.467	1.82163	1.500	3.931	PER IMP=	50.00
ROUTE RESERVOIR	503.10	3	4	.00540	9.33	.467	1.82153	1.700	1.811	AC-FT=	.171
ADD HYD	703.10	4& 7	2	.01410	21.83	1.270	1.68941	1.633	2.385		
ROUTE	703.20	2	7	.01410	21.51	1.270	1.68944	1.633	2.334		
COMPUTE NM HYD	707.00	-	3	.00850	19.96	.846	1.42414	1.500	3.667	PER IMP=	70.00
ROUTE RESERVOIR	507.00	3	4	.00850	9.93	.846	1.42411	1.766	1.274	AC-FT=	.237
ADD HYD	707.10	4& 7	2	.02280	28.03	1.916	1.58960	1.667	1.938		
ROUTE	707.20	2	7	.02280	27.48	1.916	1.58962	1.700	1.909		
COMPUTE NM HYD	711.00	-	3	.02870	37.26	2.238	2.11537	1.500	4.751	PER IMP=	100.00
ROUTE RESERVOIR	511.00	3	4	.02870	27.35	2.238	2.11536	1.866	1.489	AC-FT=	1.466
ADD HYD	711.10	4& 7	2	.05130	54.20	5.154	1.88372	1.733	1.651		
ROUTE	711.20	2	7	.05130	53.71	5.154	1.88373	1.800	1.636		
COMPUTE NM HYD	709.00	-	3	.01400	20.33	1.211	1.62164	1.733	2.269	PER IMP=	50.00
ROUTE RESERVOIR	509.00	3	4	.01400	14.88	1.211	1.62159	1.933	1.651	AC-FT=	.248
ROUTE	709.10	4	6	.01400	14.85	1.211	1.62162	1.966	1.638		
ADD HYD	709.20	6& 7	2	.06530	67.42	6.365	1.82752	1.833	1.613		
COMPUTE NM HYD	710.00	-	3	.00460	11.04	.362	1.47351	1.500	3.749	PER IMP=	55.00
ROUTE RESERVOIR	510.00	3	4	.00460	8.37	.361	1.47345	1.500	1.843	AC-FT=	.052
ADD HYD	710.10	2& 4	5	.06990	72.42	6.726	1.80422	1.800	1.619		
ROUTE	710.20	5	7	.06990	72.01	6.726	1.80423	1.866	1.610		
COMPUTE NM HYD	720.00	-	3	.01120	23.92	1.352	1.76175	1.567	3.338	PER IMP=	70.00
COMPUTE NM HYD	714.00	-	6	.02320	50.20	2.068	1.67101	1.500	4.054	PER IMP=	95.00
ADD HYD	720.10	3& 6	4	.03440	80.65	3.120	1.70053	1.500	3.636		
ROUTE RESERVOIR	520.00	4	2	.03440	31.44	3.120	1.70053	1.800	1.428	AC-FT=	1.645
ADD HYD	720.20	2& 7	5	.10430	103.38	9.846	1.77002	1.833	1.549		
ROUTE	720.30	5	7	.10430	103.31	9.846	1.77002	1.866	1.548		
COMPUTE NM HYD	712.00	-	3	.02160	64.68	2.391	2.07525	1.500	4.679	PER IMP=	97.00
ROUTE RESERVOIR	512.00	3	4	.02160	47.23	2.391	2.07524	1.600	3.417	AC-FT=	.432
ROUTE	712.10	4	6	.02160	47.19	2.391	2.07526	1.600	3.413		
ADD HYD	712.20	6& 7	2	.12590	136.09	12.237	1.82239	1.766	1.689		
ROUTE	712.30	2	7	.12590	135.84	12.237	1.82239	1.800	1.686		
COMPUTE NM HYD	716.00	-	3	.00940	24.40	.838	1.67101	1.500	4.056	PER IMP=	95.00
ADD HYD	716.10	3& 7	6	.13530	144.56	13.074	1.81187	1.766	1.669		
ROUTE	716.20	6	8	.13530	144.17	13.074	1.81187	1.800	1.665		
COMPUTE NM HYD	704.00	-	3	.01070	25.02	.954	1.67101	1.533	3.654	PER IMP=	95.00
ROUTE RESERVOIR	504.00	3	4	.01070	14.09	.954	1.67098	1.700	2.058	AC-FT=	.291
ROUTE	704.10	4	7	.01070	13.88	.954	1.67102	1.733	2.026		
COMPUTE NM HYD	703.00	-	3	.00790	20.51	.704	1.67101	1.500	4.056	PER IMP=	95.00
ROUTE RESERVOIR	503.00	3	4	.00790	6.65	.704	1.67097	1.800	1.314	AC-FT=	.333
ADD HYD	703.10	4& 7	2	.01860	20.48	1.658	1.67093	1.766	1.720		
ROUTE	703.20	2	7	.01860	20.49	1.658	1.67095	1.766	1.721		
COMPUTE NM HYD	709.00	-	3	.01400	35.64	1.211	1.62163	1.500	3.978	PER IMP=	50.00
ROUTE RESERVOIR	509.00	3	4	.01400	8.58	1.211	1.62162	1.933	.957	AC-FT=	.307
ADD HYD	709.10	4& 7	2	.03260	28.73	2.868	1.64974	1.800	1.377		
ROUTE	709.20	2	7	.03260	28.53	2.868	1.64975	1.833	1.367		
COMPUTE NM HYD	710.00	-	3	.03120	79.94	3.453	2.07525	1.533	4.003	PER IMP=	97.00
ADD HYD	710.10	3& 7	2	.06230	96.42	6.322	1.85782	1.567	2.901		
COMPUTE NM HYD	705.00	-	3	.01620	38.10	1.128	1.30565	1.500	3.482	PER IMP=	69.00
ROUTE RESERVOIR	505.00	3	4	.01620	10.17	1.128	1.30563	1.833	.983	AC-FT=	.386

COMPUTE NM HYD	716.00	-	3	.02250	51.24	1.013	1.05502	1.500	1.358	PER IMP=	20.00
ROUTE RESERVOIR	516.00	3	4	.02250	9.53	1.026	1.05501	1.700	.852	AC-FT=	1.009
ADD HYD	716.10	4& 7	5	.03870	19.61	2.754	1.03432	1.833	.772		
ROUTE	716.20	5	7	.03870	17.59	2.754	1.03432	1.900	.771		
ADD HYD	719.20	2& 7	8	.10250	110.19	9.076	1.06017	1.867	1.591		
ROUTE	719.30	8	7	.10250	109.24	9.076	1.06017	1.800	1.567		
COMPUTE NM HYD	718.00	-	3	.00940	22.84	.739	1.47352	1.500	3.746	PER IMP=	75.00
ROUTE RESERVOIR	518.00	3	4	.00940	5.23	.739	1.47349	1.866	1.035	AC-FT=	.232
ADD HYD	718.10	4& 7	2	.11190	114.48	9.814	1.04448	1.800	1.599		
ROUTE	718.20	2	7	.11190	112.06	9.814	1.04449	1.833	1.579		
COMPUTE NM HYD	718.00	-	3	.01520	38.69	1.318	1.06103	1.500	2.977	PER IMP=	50.00
ADD HYD	718.20	3& 7	2	.12710	139.15	11.132	1.04175	1.800	1.711		
ROUTE RESERVOIR	518.00	2	4	.12710	66.05	11.132	1.04175	2.200	.837	AC-FT=	1.253
ADD HYD	721.10	4& 8	9	.26240	208.03	24.203	1.02945	1.800	1.242		
COMPUTE NM HYD	719.00	-	3	.02090	27.58	1.100	.98697	1.500	2.612	PER IMP=	5.00
ROUTE RESERVOIR	519.00	3	4	.02090	12.11	1.100	.98696	1.766	.705	AC-FT=	.341
ROUTE	719.10	4	7	.02090	11.44	1.100	.98696	1.966	.655		
COMPUTE NM HYD	722.00	-	3	.02100	18.75	1.047	.93619	1.700	1.411	PER IMP=	10.00
ADD HYD	722.10	3& 7	2	.04190	29.81	2.149	.96142	1.733	1.063		
COMPUTE NM HYD	723.00	-	3	.01630	15.09	.814	.93619	1.667	1.447	PER IMP=	15.00
ADD HYD	723.10	2& 7	5	.05820	43.36	2.962	.95439	1.700	1.164		
ROUTE	723.20	5	7	.05820	43.37	2.962	.95439	1.700	1.164		
ADD HYD	723.30	7& 7	2	.32060	249.91	27.165	1.58876	1.766	1.218		
ROUTE	723.40	2	5	.32060	251.06	27.166	1.58876	1.800	1.224		
COMPUTE NM HYD	721.00	-	3	.03580	42.85	1.249	.65439	1.500	1.870	PER IMP=	5.00
ADD HYD	721.20	3& 6	8	.35640	264.06	28.415	1.49490	1.766	1.158		
COMPUTE NM HYD	717.00	-	3	.01590	34.27	1.158	1.36558	1.500	3.368	PER IMP=	45.00
ROUTE	717.10	3	4	.01590	30.11	1.158	1.36559	1.567	2.959		
COMPUTE NM HYD	720.00	-	3	.03540	42.37	1.235	.65439	1.500	1.870	PER IMP=	5.00
ADD HYD	720.10	3& 4	2	.05130	72.07	2.393	.87490	1.533	2.195		
ADD HYD	720.20	2& 6	5	.40770	294.80	30.809	1.41688	1.733	1.130		
COMPUTE NM HYD	724.00	-	3	.04650	59.72	1.736	.69990	1.500	2.007	PER IMP=	5.00
ADD HYD	721.30	3& 5	2	.45420	320.18	32.544	1.34347	1.700	1.101		
ROUTE RESERVOIR	521.00	2	4	.45420	264.28	32.544	1.34347	1.933	.909	AC-FT=	4.357
ROUTE	721.40	4	7	.45420	264.26	32.544	1.34347	1.966	.909		
COMPUTE NM HYD	726.10	-	3	.02420	32.57	.965	.74754	1.500	2.103	PER IMP=	3.00
ADD HYD	726.11	3& 7	2	.47840	268.81	33.509	1.31333	1.933	.878		
ROUTE RESERVOIR	526.10	2	5	.47840	263.32	33.509	1.31333	2.033	.860	AC-FT=	1.335
COMPUTE NM HYD	725.00	-	3	.02380	58.73	2.073	1.63305	1.500	3.656	PER IMP=	55.00
ROUTE RESERVOIR	525.00	3	4	.02380	25.30	2.073	1.63303	1.700	1.661	AC-FT=	.702
ROUTE	725.10	4	7	.02380	24.82	2.073	1.63305	1.800	1.629		
ADD HYD	725.20	6& 7	2	.50220	285.40	35.582	1.32848	2.033	.888		
ROUTE	725.30	2	7	.50220	285.30	35.582	1.32848	2.033	.888		
COMPUTE NM HYD	726.00	-	3	.15530	219.67	6.567	.79289	1.533	2.210	PER IMP=	.00
ADD HYD	726.10	3& 7	2	.65750	384.64	42.149	1.20197	1.567	.914		
ROUTE RESERVOIR	526.00	2	4	.65750	375.82	42.149	1.20197	1.633	.893	AC-FT=	.987
ROUTE	726.20	4	5	.65750	369.11	42.149	1.20197	1.667	.877		
COMPUTE NM HYD	728.00	-	3	.03440	47.65	1.533	.83561	1.500	2.255	PER IMP=	15.00
ADD HYD	728.10	3& 5	6	.69190	395.97	43.682	1.18376	1.667	.894		
COMPUTE NM HYD	727.00	-	3	.05270	80.94	2.349	.83561	1.567	1.807	PER IMP=	15.00
ROUTE RESERVOIR	527.00	3	4	.05270	59.57	2.349	.83560	1.600	1.766	AC-FT=	.127
ADD HYD	727.10	4& 6	2	.74460	449.84	46.031	1.15912	1.633	.944		

AHYMO SUMMARY TABLE AHYMO91 - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/17/1994
INPUT FILE = [redacted] USER NO.= MOLICONM.101

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF INCHES	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	1.87
COMPUTE NM HYD	701.00	-	1	.01120	14.73	.627	1.04742	1.567	2.060	PER IMP=	70.00
COMPUTE NM HYD	714.00	-	6	.03320	36.99	1.191	.95262	1.500	1.742	PER IMP=	55.00
ADD HYD	710.10	1& 6	4	.03440	47.13	1.818	.99186	1.533	1.224		
ROUTE RESERVOIR	810.10	4	2	.03440	14.73	1.813	.99186	1.700	.533	AC-FT=	.374
ROUTE	710.20	2	7	.03440	14.77	1.818	.99087	1.700	.533		
COMPUTE NM HYD	711.00	-	1	.02160	42.22	1.504	1.30561	1.500	3.054	PER IMP=	97.00
ROUTE RESERVOIR	811.00	1	4	.02160	13.10	1.504	1.30557	1.866	.948	AC-FT=	.713
ROUTE	711.10	4	6	.02160	13.10	1.504	1.30557	1.866	.948		
ADD HYD	710.20	6& 7	1	.05600	27.47	3.322	1.11224	1.700	.767		
ROUTE	711.30	1	7	.05600	27.47	3.322	1.11224	1.733	.766		
COMPUTE NM HYD	716.00	-	1	.01940	14.99	.483	.96262	1.500	2.472	PER IMP=	55.00
ROUTE RESERVOIR	812.10	1	4	.01940	7.57	.483	.96259	1.667	1.291	AC-FT=	.156
ADD HYD	716.10	4& 7	6	.05540	34.62	3.804	1.09072	1.733	.827		
ROUTE	712.20	6	8	.05540	34.47	3.804	1.09073	1.800	.824		
COMPUTE NM HYD	710.00	-	1	.03120	52.03	2.173	1.30561	1.533	2.606	PER IMP=	97.00
ROUTE RESERVOIR	819.00	1	2	.03120	29.22	2.173	1.30559	1.733	1.463	AC-FT=	.597
COMPUTE NM HYD	705.00	-	3	.01390	17.82	.497	.67108	1.500	2.003	PER IMP=	20.00
ROUTE RESERVOIR	505.00	3	4	.01390	5.56	.497	.67106	1.733	.625	AC-FT=	.239
ROUTE	705.10	4	7	.01390	5.56	.497	.67107	1.766	.625		
COMPUTE NM HYD	706.00	-	3	.02250	29.44	.805	.89606	1.500	2.045	PER IMP=	23.00
ROUTE RESERVOIR	806.00	3	4	.02250	6.31	.835	.89605	1.333	.439	AC-FT=	.474
ADD HYD	706.10	4& 7	5	.03640	11.84	1.333	.88649	1.766	.508		
ROUTE	715.21	5	7	.03640	11.82	1.333	.88650	1.800	.507		
ADD HYD	710.21	2& 7	5	.06760	40.89	3.505	.97223	1.733	.945		
ROUTE	710.30	5	7	.06760	40.88	3.505	.97223	1.766	.945		
COMPUTE NM HYD	715.00	-	3	.00940	13.32	.399	.79602	1.500	2.213	PER IMP=	75.00
ROUTE RESERVOIR	815.00	3	4	.00940	3.71	.399	.79599	1.800	.615	AC-FT=	.233
ADD HYD	715.10	4& 7	2	.07700	44.58	3.904	.95071	1.766	.905		
ROUTE	715.20	2	7	.07700	44.42	3.904	.95072	1.800	.901		
COMPUTE NM HYD	718.00	-	1	.01520	23.56	.747	.92097	1.500	2.422	PER IMP=	50.00
ADD HYD	718.20	1& 7	2	.09220	58.81	4.651	.94581	1.700	.895		
ROUTE RESERVOIR	518.00	2	4	.09220	40.19	4.651	.94581	2.100	.681	AC-FT=	1.160
ADD HYD	721.10	4& 8	9	.15760	72.70	8.455	1.00594	2.000	.721		
COMPUTE NM HYD	719.00	-	3	.02090	32.39	1.027	.92097	1.500	2.422	PER IMP=	50.00
ROUTE RESERVOIR	819.00	3	4	.02090	11.33	1.027	.92096	1.733	.347	AC-FT=	.433
ROUTE	719.10	4	7	.02090	10.77	1.027	.92096	2.000	.305		
COMPUTE NM HYD	722.00	-	3	.02100	19.05	1.021	.91127	1.567	1.417	PER IMP=	55.00
ADD HYD	722.10	3& 7	2	.04190	28.03	2.047	.91606	1.700	1.045		
COMPUTE NM HYD	723.00	-	3	.01630	14.58	.755	.86899	1.667	1.397	PER IMP=	50.00
ADD HYD	723.10	2& 3	5	.05820	42.41	2.803	.90287	1.700	1.139		
ROUTE	723.20	5	7	.05820	42.43	2.803	.90288	1.700	1.139		
ADD HYD	723.30	9& 7	2	.21580	106.54	11.258	.97814	1.766	.771		
ROUTE	723.40	2	6	.21580	102.86	11.258	.97814	1.933	.745		
COMPUTE NM HYD	721.00	-	3	.03580	49.62	1.659	.86886	1.500	2.166	PER IMP=	55.00
ADD HYD	721.20	3& 6	8	.25160	115.61	12.917	.96259	1.866	.718		
COMPUTE NM HYD	717.00	-	3	.01590	19.10	.631	.74469	1.500	1.877	PER IMP=	45.00
ROUTE	717.10	3	4	.01590	14.95	.632	.74470	1.567	1.469		
COMPUTE NM HYD	720.00	-	3	.03540	49.06	1.640	.86886	1.500	2.166	PER IMP=	55.00
ADD HYD	720.10	3& 4	2	.05130	60.84	2.272	.83036	1.533	1.353		
ADD HYD	721.20	2& 8	9	.30270	157.87	15.189	.94020	1.800	.711		
COMPUTE NM HYD	724.00	-	1	.04650	20.75	.892	.23872	1.533	.697	PER IMP=	5.00
ADD HYD	721.30	2& 9	2	.34940	156.45	15.781	.84684	1.533	.700		
ROUTE RESERVOIR	821.00	2	4	.34940	139.37	15.781	.84684	1.866	.624	AC-FT=	.748
ROUTE	721.40	4	7	.34940	139.57	15.781	.84684	1.866	.624		
COMPUTE NM HYD	726.10	-	1	.02420	12.07	.355	.27536	1.500	.779	PER IMP=	5.00
ADD HYD	726.11	1& 7	2	.37360	147.15	16.136	.30982	1.833	.595		

COMPUTE NM HYD	725.00	-	3	.02380	35.29	1.215	.95712	1.500	1.317	PER IMP=	85.00
ROUTE RESERVOIR	525.00	2	4	.02380	15.87	1.215	.95711	1.657	1.095	AC-FT=	.347
ROUTE	725.10	4	7	.02380	15.71	1.215	.95713	1.766	1.071		
ADD HYD	725.20	6& 7	3	.39740	157.83	17.351	.81864	1.833	.621		
ROUTE	725.30	2	7	.39740	157.31	17.351	.81864	1.866	.620		
COMPUTE NM HYD	726.00	-	3	.15530	55.30	1.355	.23437	1.533	.383	PER IMP=	.00
ADD HYD	726.10	3& 7	2	.55270	214.57	19.706	.66852	1.650	.506		
ROUTE RESERVOIR	526.00	2	4	.55270	209.16	19.706	.66852	1.633	.391	AC-FT=	.428
ROUTE	726.20	4	5	.55270	204.17	19.706	.66852	1.700	.377		
COMPUTE NM HYD	728.00	-	3	.03440	25.24	.824	.24554	1.500	.719	PER IMP=	15.00
ADD HYD	728.10	3& 5	6	.58710	213.31	20.340	.64959	1.700	.557		
COMPUTE NM HYD	727.00	-	3	.05270	24.39	.971	.24554	1.567	.718	PER IMP=	15.11
ROUTE RESERVOIR	527.00	2	4	.05270	24.22	.971	.24552	1.500	.718	AC-FT=	.351
ADD HYD	727.10	4& 6	2	.63980	232.19	21.311	.62455	1.667	.557		

AMXMD SUMMARY TABLE (PRXMD491) - AMAFCA VERSION OF HXMD - APRIL, 1991 RUN DATE (MON/DAY/YR) = 05/17/1994

INPUT FILE = [redacted] USER NO.= MOLZCONM.101

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN5=	2.250
COMPUTE NM HYD	730.00	-	3	.01120	23.92	1.052	1.76175	1.567	3.338	PER IMP=	70.00
COMPUTE NM HYD	714.00	-	5	.02320	50.20	2.168	1.67101	1.500	4.034	PER IMP=	55.10
ADD HYD	730.10	3& 5	4	.03440	80.05	3.120	1.73053	1.500	3.636		
ROUTE RESERVOIR	530.00	4	2	.03440	16.76	3.120	1.70053	2.166	.731	AC-FT=	1.733
ROUTE	731.30	2	7	.03440	16.75	3.120	1.70054	2.166	.731		
COMPUTE NM HYD	712.00	-	3	.05150	84.68	2.391	2.07525	1.500	4.579	PER IMP=	97.00
ROUTE RESERVOIR	512.30	3	4	.05150	14.25	2.391	2.07524	2.033	1.031	AC-FT=	1.309
ROUTE	712.10	4	8	.05160	14.25	2.391	2.07522	2.066	1.031		
ADD HYD	712.20	8& 7	2	.08600	31.00	5.511	1.84504	2.066	.865		
ROUTE	712.30	2	7	.08600	31.00	5.511	1.84505	2.100	.865		
COMPUTE NM HYD	715.00	-	3	.00940	24.40	.838	1.67101	1.500	4.056	PER IMP=	55.00
ROUTE RESERVOIR	512.00	3	4	.00940	11.36	.838	1.67098	1.700	1.888	AC-FT=	.188
ADD HYD	716.10	4& 7	6	.16540	41.20	6.348	1.82002	1.800	.984		
ROUTE	716.20	6	8	.06540	41.15	6.348	1.82002	1.833	.983		
COMPUTE NM HYD	710.00	-	3	.03120	79.94	3.453	2.07525	1.533	4.003	PER IMP=	97.00
ROUTE RESERVOIR	509.00	3	2	.03120	31.82	3.453	2.07524	1.866	1.594	AC-FT=	1.199
COMPUTE NM HYD	705.00	-	3	.01390	31.25	.983	1.32539	1.500	3.513	PER IMP=	20.00
ROUTE RESERVOIR	505.00	3	4	.01390	9.03	.983	1.32538	1.833	1.015	AC-FT=	.499
ROUTE	705.10	4	7	.01390	9.03	.983	1.32539	1.833	1.015		
COMPUTE NM HYD	706.00	-	3	.02250	51.24	1.626	1.35502	1.500	3.558	PER IMP=	23.00
ROUTE RESERVOIR	506.00	3	4	.02250	9.53	1.626	1.35501	1.966	.662	AC-FT=	1.009
ADD HYD	706.10	4& 7	5	.03640	18.45	2.609	1.34367	1.866	.792		
ROUTE	706.20	5	7	.03640	18.44	2.609	1.34367	1.900	.791		
ADD HYD	710.20	2& 7	8	.06760	50.25	6.062	1.68132	1.900	1.161		
ROUTE	710.30	8	7	.06760	50.24	6.062	1.68132	1.900	1.161		
COMPUTE NM HYD	715.00	-	3	.00940	22.64	.739	1.47352	1.500	3.764	PER IMP=	35.00
ROUTE RESERVOIR	515.00	3	4	.00940	6.23	.739	1.47349	1.866	1.035	AC-FT=	.282
ADD HYD	715.10	4& 7	2	.07700	56.45	6.800	1.65594	1.900	1.145		
ROUTE	715.20	2	7	.07700	56.37	6.800	1.65595	1.933	1.144		
COMPUTE NM HYD	713.00	-	3	.01520	38.69	1.315	1.62163	1.500	3.977	PER IMP=	50.00
ADD HYD	718.20	3& 7	2	.09220	75.54	8.115	1.65029	1.567	1.280		
ROUTE RESERVOIR	518.00	2	4	.09220	57.10	8.115	1.65028	2.200	.968	AC-FT=	1.697
ADD HYD	721.10	4& 8	9	.15760	96.14	14.463	1.72072	2.100	.953		
COMPUTE NM HYD	719.00	-	3	.02090	53.20	1.808	1.62163	1.500	3.977	PER IMP=	50.00
ROUTE RESERVOIR	519.00	3	4	.02090	14.66	1.808	1.62162	1.866	1.096	AC-FT=	.334
ROUTE	719.10	4	7	.02090	14.30	1.808	1.62162	2.033	1.069		
COMPUTE NM HYD	722.00	-	3	.02100	32.11	1.795	1.59370	1.667	2.389	PER IMP=	55.00
ADD HYD	722.10	3& 7	2	.04190	43.85	3.592	1.60759	1.700	1.635		
COMPUTE NM HYD	723.00	-	3	.01630	24.87	1.342	1.54357	1.667	2.384	PER IMP=	50.00
ADD HYD	723.10	2& 3	5	.05820	68.44	4.934	1.58965	1.700	1.837		
ROUTE	723.20	5	7	.05820	68.46	4.934	1.58966	1.700	1.838		
ADD HYD	723.30	9& 7	2	.21580	150.94	19.397	1.68537	1.766	1.093		
ROUTE	723.40	2	8	.21580	146.86	19.398	1.68537	1.866	1.063		
COMPUTE NM HYD	721.00	-	3	.03580	84.81	2.919	1.52862	1.500	3.702	PER IMP=	85.00
ADD HYD	721.20	3& 8	5	.25160	172.85	22.316	1.66307	1.800	1.073		
COMPUTE NM HYD	717.00	-	3	.01590	34.27	1.158	1.36558	1.500	3.368	PER IMP=	45.00
ROUTE	717.10	3	4	.01590	30.11	1.158	1.36559	1.567	2.959		
COMPUTE NM HYD	720.00	-	3	.03540	83.37	2.886	1.52862	1.500	3.702	PER IMP=	55.00
ADD HYD	720.10	3& 4	2	.05120	109.64	4.044	1.47808	1.533	3.340		
ADD HYD	720.20	2& 5	8	.30290	227.58	26.360	1.63174	1.833	1.236		
COMPUTE NM HYD	724.00	-	3	.04650	89.72	1.736	.69990	1.500	2.997	PER IMP=	5.00
ADD HYD	721.30	2& 8	2	.34940	299.04	28.096	1.50772	1.533	1.337		
ROUTE RESERVOIR	521.00	2	4	.34940	215.95	28.096	1.50772	1.866	.966	AC-FT=	1.566
ROUTE	721.40	4	7	.34940	215.95	28.096	1.50772	1.866	.966		
COMPUTE NM HYD	726.10	-	3	.02420	32.57	.955	.74754	1.500	2.103	PER IMP=	5.00
ADD HYD	726.11	3& 7	2	.37360	226.91	29.051	1.45948	1.533	.949		

COMPUTE NM HYD	725.00	-	1	.02280	58.73	2.073	1.03305	1.500	1.356	PER IMP=	25.00
ROUTE RESERVOIR	525.30	2	4	.02280	25.30	2.073	1.03303	1.700	1.661	AC-FT=	.702
ROUTE	725.10	4	7	.02280	24.82	2.073	1.03305	1.900	1.629		
ADD HYD	725.20	5& 7	1	.03740	245.93	31.134	1.46893	1.833	.971		
ROUTE	725.30	1	7	.03740	24e.90	31.134	1.46893	1.8e6	.971		
COMPUTE NM HYD	726.00	-	3	.15530	219.67	5.567	.79289	1.533	1.210	PER IMP=	.0
ADD HYD	726.10	3& 7	2	.55270	465.14	37.701	1.27897	1.567	1.145		
ROUTE RESERVOIR	525.00	2	4	.55270	197.81	37.701	1.27879	1.833	1.117	AC-FT=	1.141
ROUTE	72e.20	4	5	.55270	263.72	37.711	1.27898	1.ee7	1.199		
COMPUTE NM HYD	729.00	-	3	.03440	47.e5	1.833	.9756l	1.300	1.255	PER IMP=	15.00
ADD HYD	726.10	3& 5	6	.58710	415.37	39.234	1.25300	1.633	1.116		
COMPUTE NM HYD	727.00	-	3	.15270	e0.54	2.549	.83561	1.567	1.307	PER IMP=	15.00
ROUTE RESERVOIR	527.00	3	4	.15270	59.37	2.549	.8756e	1.e31	1.766	AC-FT=	.127
ADD HYD	727.00	4& 6	2	.e3790	472.87	41.582	1.21862	1.e33	1.154		

APPENDIX C

SUMMARY PRINTOUTS
BASIN SW35

AHYMO SUMMARY TABLE (AHYMO491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/21/1994

INPUT FILE = SW35EX10.DAT USER NO.= MOLZCONM.101

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	1.670
COMPUTE NM HYD	904.00	-	3	.01290	4.24	.223	.33189	1.667	.513	PER IMP=	15.00
ROUTE	904.10	3	7	.01290	4.20	.228	.33193	1.667	.508		
COMPUTE NM HYD	901.00	-	3	.03020	33.70	1.322	.82101	1.567	1.744	PER IMP=	38.00
ROUTE RESERVOIR	501.00	3	4	.03020	22.78	1.322	.82100	1.733	1.178	AC-FT=	.310
ADD HYD	901.10	4& 7	2	.04310	26.90	1.551	.67460	1.700	.975		
ROUTE RESERVOIR	501.00	2	4	.04310	13.07	1.551	.67460	2.100	.474	AC-FT=	.568
ROUTE	901.20	4	7	.04310	13.06	1.551	.67461	2.233	.473		
COMPUTE NM HYD	902.00	-	3	.00600	9.74	.316	.98761	1.500	2.535	PER IMP=	58.00
ROUTE RESERVOIR	502.00	3	4	.00600	4.83	.316	.98756	1.667	1.257	AC-FT=	.107
ADD HYD	902.10	4& 7	2	.04910	17.10	1.867	.71283	1.933	.544		
ROUTE RESERVOIR	501.00	2	4	.04910	14.70	1.867	.71283	2.300	.468	AC-FT=	.270
ROUTE	902.20	4	7	.04910	14.70	1.867	.71283	2.433	.468		
COMPUTE NM HYD	903.00	-	3	.01120	13.06	.336	.56279	1.500	1.822	PER IMP=	7.00
ROUTE RESERVOIR	503.00	3	4	.01120	7.42	.336	.56276	1.633	1.035	AC-FT=	.134
ADD HYD	903.10	4& 7	2	.06030	17.41	2.203	.68495	2.033	.451		
COMPUTE NM HYD	905.00	-	3	.05270	20.33	.783	.27871	1.567	.603	PER IMP=	7.00
ADD HYD	905.10	3& 2	8	.11300	29.86	2.986	.49548	1.633	.413		
COMPUTE NM HYD	906.00	-	3	.00990	11.46	.293	.55446	1.500	1.808	PER IMP=	6.00

AHYMO SUMMARY TABLE (AHYMO491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/21/1994

INPUT FILE = **SW35E100.DAT** USER NO.= MOLICONM.101

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS*1 PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	2.350
COMPUTE NM HYD	804.00	-	3	.01290	10.38	.559	.81291	1.667	1.257	PER IMP=	15.00
ROUTE	804.10	3	7	.01290	10.28	.559	.81291	1.700	1.245		
COMPUTE NM HYD	301.00	-	3	.03020	57.44	2.421	1.50314	1.567	2.972	PER IMP=	38.00
ROUTE RESERVOIR	501.00	3	4	.03020	37.02	2.421	1.50312	1.733	1.916	AC-FT=	.614
ADD HYD	801.10	4& 7	2	.04310	47.05	2.980	1.29653	1.733	1.706		
ROUTE RESERVOIR	501.00	2	4	.04310	13.20	2.980	1.29653	2.366	.479	AC-FT=	1.596
ROUTE	801.20	4	7	.04310	13.19	2.980	1.29653	2.533	.478		
COMPUTE NM HYD	802.00	-	3	.00600	15.76	.544	1.70063	1.500	4.104	PER IMP=	58.00
ROUTE RESERVOIR	502.00	3	4	.00600	7.35	.544	1.70058	1.700	1.913	AC-FT=	.191
ADD HYD	802.10	4& 7	2	.04910	19.97	3.524	1.34589	1.766	.636		
ROUTE RESERVOIR	501.00	2	4	.04910	14.74	3.524	1.34589	2.566	.469	AC-FT=	.424
ROUTE	802.20	4	7	.04910	14.74	3.524	1.34589	2.566	.469		
COMPUTE NM HYD	303.00	-	3	.01120	23.74	.715	1.19703	1.500	3.312	PER IMP=	7.00
ROUTE RESERVOIR	503.00	3	4	.01120	13.81	.715	1.19700	1.633	1.926	AC-FT=	.234
ADD HYD	303.10	4& 7	2	.06030	26.16	4.239	1.31822	1.800	.678		
COMPUTE NM HYD	805.00	-	3	.05270	54.74	2.125	.75622	1.567	1.623	PER IMP=	7.00
ADD HYD	305.10	3& 2	8	.11300	72.89	6.365	1.05612	1.600	1.008		
COMPUTE NM HYD	806.00	-	3	.00990	20.89	.627	1.18715	1.500	3.297	PER IMP=	6.00

APPENDIX C

SUMMARY PRINTOUTS
BASIN M3

AHYMO SUMMARY TABLE (AHYM0491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/17/1994

INPUT FILE = [redacted] USER NO.= MDLZCONM.I01

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	1.570
COMPUTE NM HYD	906.00	-	3	.01920	34.05	1.156	1.12922	1.500	2.771	PER IMP=	75.00
ROUTE	906.10	3	7	.01920	32.82	1.156	1.12923	1.500	2.671		
COMPUTE NM HYD	903.00	-	3	.01270	17.42	.511	.75437	1.500	2.143	PER IMP=	30.00
ROUTE RESERVOIR	903.00	3	4	.01270	7.27	.511	.75435	1.700	.894	AC-FT=	.220
ADD HYD	903.10	4& 7	2	.03190	37.56	1.667	.97995	1.533	1.840		
ROUTE	903.20	2	7	.03190	37.02	1.667	.97997	1.533	1.813		
COMPUTE NM HYD	905.00	-	3	.00600	11.72	.415	1.29582	1.500	3.052	PER IMP=	95.00
ROUTE RESERVOIR	505.00	3	4	.00600	3.16	.415	1.29577	1.933	.823	AC-FT=	.221
ROUTE	905.10	4	8	.00600	3.16	.415	1.29563	1.933	.823		
COMPUTE NM HYD	904.00	-	3	.00380	7.59	.271	1.33747	1.500	3.121	PER IMP=	100.00
ROUTE RESERVOIR	504.00	3	4	.00380	5.55	.271	1.33739	1.600	2.281	AC-FT=	.066
ADD HYD	905.20	8& 4	2	.00980	7.91	.686	1.31173	1.600	1.262		
ROUTE	905.30	2	6	.00980	7.89	.686	1.31179	1.633	1.257		
ADD HYD	904.10	6& 7	2	.04170	43.13	2.353	1.05792	1.567	1.616		
COMPUTE NM HYD	902.00	-	3	.02850	23.85	1.248	.82101	1.700	1.308	PER IMP=	38.00
ROUTE RESERVOIR	502.00	3	4	.02850	19.42	1.248	.82099	1.833	1.065	AC-FT=	.238
ADD HYD	902.10	2& 4	5	.07020	50.12	3.601	.96173	1.567	1.116		
ROUTE	902.20	5	7	.07020	49.29	3.601	.96174	1.600	1.097		
COMPUTE NM HYD	901.00	-	3	.02080	21.31	.564	.50838	1.500	1.601	PER IMP=	10.00
ROUTE RESERVOIR	501.00	3	4	.02080	3.89	.564	.50837	1.833	.292	AC-FT=	.369
ROUTE	901.20	4	6	.02080	3.64	.564	.50831	2.033	.274		
COMPUTE NM HYD	901.10	-	3	.04650	11.17	1.322	.53310	2.233	.375	PER IMP=	10.00
ROUTE RESERVOIR	501.10	3	4	.04650	8.92	1.322	.53310	2.600	.300	AC-FT=	.286
ADD HYD	901.12	6& 4	2	.06730	11.82	1.886	.52541	2.533	.274		
ADD HYD	901.98	2& 7	4	.13750	51.19	5.487	.74817	1.633	.582		
ROUTE	901.99	4	7	.13750	50.96	5.487	.74818	1.700	.579		
COMPUTE NM HYD	907.00	-	3	.03440	3.92	.393	.21429	1.933	.178	PER IMP=	3.00
ADD HYD	907.10	3& 7	2	.17190	54.02	5.880	.64133	1.766	.491		

AHYMO SUMMARY TABLE (AHYMO491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/17/1994

INPUT FILE =[redacted] USER NO.= MGLZCONM.101

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	2.350
COMPUTE NM HYD	906.00	-	3	.01920	53.63	1.913	1.86850	1.500	4.364	PER IMP=	75.00
ROUTE	906.10	3	7	.01920	51.95	1.913	1.86851	1.500	4.228		
COMPUTE NM HYD	903.00	-	3	.01270	29.81	.965	1.42414	1.500	3.668	PER IMP=	30.00
ROUTE RESERVOIR	503.00	3	4	.01270	12.78	.965	1.42412	1.700	1.572	AC-FT=	.396
ADD HYD	903.10	4& 7	2	.03190	59.95	2.878	1.69156	1.533	2.936		
ROUTE	903.20	2	7	.03190	59.22	2.878	1.69157	1.533	2.901		
COMPUTE NM HYD	905.00	-	3	.00600	17.96	.661	2.06599	1.500	4.678	PER IMP=	95.00
ROUTE RESERVOIR	505.00	3	4	.00600	4.94	.661	2.06594	1.933	1.288	AC-FT=	.346
ROUTE	905.10	4	8	.00600	4.94	.661	2.06580	1.933	1.288		
COMPUTE NM HYD	904.00	-	3	.00380	11.56	.429	2.11536	1.500	4.754	PER IMP=	100.00
ROUTE RESERVOIR	504.00	3	4	.00330	9.26	.429	2.11529	1.567	3.807	AC-FT=	.089
ADD HYD	905.20	8& 4	2	.00980	12.75	1.090	2.08491	1.600	2.032		
ROUTE	905.30	2	6	.00980	12.74	1.090	2.08496	1.600	2.031		
ADD HYD	904.10	6& 7	2	.04170	70.16	3.968	1.78400	1.533	2.629		
COMPUTE NM HYD	902.00	-	3	.02350	41.06	2.285	1.50314	1.700	2.251	PER IMP=	38.00
ROUTE RESERVOIR	502.00	3	4	.02850	25.08	2.285	1.50312	1.966	1.375	AC-FT=	.551
ADD HYD	902.10	2& 4	5	.07020	82.33	6.252	1.66996	1.567	1.832		
ROUTE	902.20	5	7	.07020	80.96	6.252	1.66997	1.633	1.802		
COMPUTE NM HYD	901.00	-	3	.02080	40.91	1.229	1.10807	1.500	3.073	PER IMP=	10.00
ROUTE RESERVOIR	501.00	3	4	.02080	8.32	1.229	1.10806	1.900	.625	AC-FT=	.790
ROUTE	901.20	4	6	.02080	7.94	1.229	1.10800	2.033	.597		
COMPUTE NM HYD	901.10	-	3	.04650	23.47	2.842	1.14585	2.266	.789	PER IMP=	10.00
ROUTE RESERVOIR	501.10	3	4	.04650	18.86	2.842	1.14585	2.600	.634	AC-FT=	.605
ADD HYD	901.12	6& 4	2	.06730	25.14	4.071	1.13413	2.533	.584		
ADD HYD	901.98	2& 7	4	.13750	85.95	10.323	1.40769	1.667	.977		
ROUTE	901.99	4	7	.13750	85.53	10.323	1.40770	1.667	.972		
COMPUTE NM HYD	907.00	-	3	.03440	12.45	1.226	.66816	1.933	.566	PER IMP=	3.00
ADD HYD	907.10	3& 7	2	.17190	94.11	11.549	1.25970	1.700	.855		

AHYMD SUMMARY TABLE (AHYMD491) - AMAFCA VERSION OF HYMD - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/17/1994

INPUT FILE = [redacted] USER NO.= MOLZCONM.101

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	1.570
COMPUTE NM HYD	906.00	-	3	.01920	34.05	1.156	1.12922	1.500	2.771	PER IMP=	75.00
ROUTE	906.10	3	7	.01920	32.82	1.156	1.12923	1.500	2.671		
COMPUTE NM HYD	903.00	-	3	.01270	17.42	.511	.75437	1.500	2.143	PER IMP=	30.00
ROUTE RESERVOIR	503.00	3	4	.01270	7.27	.511	.75435	1.700	.894	AC-FT=	.220
ADD HYD	903.10	4& 7	2	.03190	37.56	1.667	.97995	1.533	1.840		
ROUTE	903.20	2	7	.03190	37.02	1.667	.97997	1.533	1.813		
COMPUTE NM HYD	905.00	-	3	.00600	11.72	.415	1.29582	1.500	3.052	PER IMP=	75.00
ROUTE RESERVOIR	505.00	3	4	.00600	3.16	.415	1.29577	1.933	.823	AC-FT=	.221
ROUTE	905.10	4	8	.00600	3.16	.415	1.29563	1.933	.823		
COMPUTE NM HYD	904.00	-	3	.00380	7.59	.271	1.33747	1.500	3.121	PER IMP=	100.00
ROUTE RESERVOIR	504.00	3	4	.00380	5.55	.271	1.33739	1.600	2.281	AC-FT=	.066
ADD HYD	905.20	8& 4	2	.00980	7.91	.686	1.31173	1.600	1.262		
ROUTE	905.30	2	6	.00980	7.89	.686	1.31179	1.633	1.257		
ADD HYD	904.10	6& 7	17	.04170	43.13	2.353	1.05792	1.567	1.616		
COMPUTE NM HYD	704.00	-	3	.01300	18.62	.667	.96262	1.533	2.238	PER IMP=	55.00
ROUTE RESERVOIR	504.00	3	4	.01300	5.82	.667	.96259	1.866	.699	AC-FT=	.337
ROUTE	704.10	4	7	.01300	5.80	.667	.96255	1.900	.697		
COMPUTE NM HYD	703.00	-	3	.00790	12.60	.406	.96262	1.500	2.493	PER IMP=	55.00
ROUTE RESERVOIR	503.00	3	4	.00790	5.99	.406	.96258	1.667	1.185	AC-FT=	.141
ADD HYD	703.10	4& 7	2	.02090	11.33	1.073	.96251	1.800	.847		
ROUTE	703.20	2	7	.02090	11.32	1.073	.96254	1.800	.847		
COMPUTE NM HYD	729.00	-	3	.01400	22.95	.750	1.00427	1.500	2.562	PER IMP=	60.00
ROUTE RESERVOIR	509.00	3	4	.01400	5.45	.750	1.00425	1.900	.608	AC-FT=	.399
ADD HYD	709.10	4& 7	2	.03490	16.73	1.823	.97924	1.833	.749		
ROUTE	709.20	2	7	.03490	16.66	1.823	.97925	1.866	.746		
COMPUTE NM HYD	701.00	-	3	.00420	6.78	.219	.97928	1.500	2.522	PER IMP=	57.00
ROUTE RESERVOIR	501.00	3	4	.00420	5.56	.219	.97922	1.567	2.070	AC-FT=	.029
ADD HYD	709.10	4& 7	2	.03910	19.36	2.042	.97923	1.700	.774		
ROUTE	701.10	2	7	.03910	19.31	2.042	.97924	1.733	.772		
COMPUTE NM HYD	702.00	-	3	.00450	7.59	.251	1.04592	1.500	2.634	PER IMP=	65.00
ROUTE RESERVOIR	502.00	3	4	.00450	5.45	.251	1.04586	1.600	1.892	AC-FT=	.032
ADD HYD	702.10	4& 7	2	.04360	24.35	2.293	.98611	1.733	.872		
ROUTE	702.20	2	7	.04360	24.28	2.293	.98612	1.733	.870		
COMPUTE NM HYD	708.00	-	3	.00540	8.38	.265	.92097	1.500	2.424	PER IMP=	50.00
ROUTE RESERVOIR	508.00	3	4	.00540	4.11	.265	.92091	1.667	1.189	AC-FT=	.095
ADD HYD	708.10	4& 7	2	.04900	28.30	2.558	.97891	1.733	.902		
ROUTE	708.20	2	7	.04900	28.26	2.558	.97892	1.733	.901		
COMPUTE NM HYD	707.00	-	3	.00850	11.66	.342	.75437	1.500	2.144	PER IMP=	30.00
ROUTE RESERVOIR	507.00	3	4	.00850	4.35	.342	.75434	1.733	.800	AC-FT=	.156
ADD HYD	707.10	4& 7	2	.05750	32.61	2.900	.94570	1.733	.886		
ROUTE	707.20	2	7	.05750	31.85	2.900	.94571	1.766	.865		
COMPUTE NM HYD	711.00	-	3	.02870	57.30	2.047	1.33747	1.500	3.119	PER IMP=	100.00
ROUTE RESERVOIR	511.00	3	4	.02870	37.09	2.047	1.33746	1.600	2.019	AC-FT=	.373
ADD HYD	711.10	4& 7	2	.08620	67.69	4.947	1.07613	1.766	1.227		
ROUTE	711.20	2	7	.08620	66.71	4.947	1.07614	1.766	1.209		
COMPUTE NM HYD	709.00	-	3	.01400	12.13	.688	.92097	1.733	1.354	PER IMP=	50.00
ROUTE RESERVOIR	509.00	3	4	.01400	9.55	.688	.92092	1.900	1.066	AC-FT=	.118
ROUTE	709.10	4	6	.01400	9.55	.688	.92095	1.900	1.065		
ADD HYD	709.20	6& 7	2	.10020	75.47	5.635	1.05445	1.766	1.177		
COMPUTE NM HYD	713.00	-	3	.00460	6.52	.195	.79602	1.500	2.215	PER IMP=	35.00
ROUTE RESERVOIR	513.00	3	4	.00460	5.28	.195	.79596	1.567	1.793	AC-FT=	.021
ADD HYD	713.10	2& 4	5	.10480	77.82	5.830	1.04310	1.766	1.160		
ROUTE	713.20	5	7	.10480	74.73	5.830	1.04310	1.833	1.114		
ADD HYD	902.10	19& 7	6	.14650	99.89	8.183	1.04732	1.800	1.065		
COMPUTE NM HYD	902.00	-	3	.02850	23.85	1.248	.82101	1.700	1.308	PER IMP=	38.00
ROUTE RESERVOIR	502.00	3	4	.02850	19.42	1.248	.82099	1.833	1.065	AC-FT=	.278

ROUTE	902.20	5	7	.17500	118.64	9.431	1.01046	1.833	1.059		
COMPUTE NM HYD	901.00	-	3	.02080	21.31	.564	.50838	1.500	1.601	PER IMP=	10.00
ROUTE RESERVOIR	501.00	3	4	.02080	3.89	.564	.50837	1.833	.292	AC-FT=	.369
ROUTE	901.20	4	6	.02080	3.64	.564	.50831	2.033	.274		
COMPUTE NM HYD	901.10	-	3	.04650	11.17	1.322	.53310	2.233	.375	PER IMP=	10.00
ROUTE RESERVOIR	501.10	3	4	.04550	8.92	1.322	.53310	2.600	.300	AC-FT=	.235
ADD HYD	901.12	6& 4	2	.06730	11.82	1.886	.52541	2.533	.274		
ADD HYD	901.98	2& 7	4	.24230	123.85	11.317	.87573	1.833	.799		
ROUTE	901.99	4	7	.24230	122.98	11.317	.87574	1.833	.793		
COMPUTE NM HYD	907.00	-	3	.03440	3.92	.393	.21429	1.933	.178	PER IMP=	3.00
ADD HYD	907.10	3& 7	2	.27670	126.67	11.710	.79350	1.833	.715		

AHYMC SUMMARY TABLE (AHYMC491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/17/1994

INPUT FILE = m3fu106.dat USER NO.= MOLZCONM.101

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	2.350
COMPUTE NM HYD	906.00	-	3	.01920	53.63	1.913	1.86850	1.500	4.364	PER IMP=	75.00
ROUTE	906.10	3	7	.01920	51.65	1.913	1.86851	1.500	4.228		
COMPUTE NM HYD	903.00	-	3	.01270	29.21	.965	1.42414	1.500	3.668	PER IMP=	30.00
ROUTE RESERVOIR	503.00	3	4	.01270	12.73	.965	1.42412	1.700	1.572	AC-FT=	.395
ADD HYD	903.10	4& 7	2	.03190	59.95	2.878	1.69156	1.533	2.936		
ROUTE	903.20	2	7	.03190	59.22	2.878	1.69157	1.533	2.901		
COMPUTE NM HYD	905.00	-	3	.00600	17.96	.661	2.06599	1.500	4.678	PER IMP=	95.00
ROUTE RESERVOIR	505.00	3	4	.00600	4.94	.661	2.06594	1.933	1.288	AC-FT=	.346
ROUTE	905.10	4	8	.00600	4.94	.661	2.06580	1.933	1.289		
COMPUTE NM HYD	904.00	-	3	.00380	11.56	.429	2.11536	1.500	4.754	PER IMP=	100.00
ROUTE RESERVOIR	504.00	3	4	.00380	9.26	.429	2.11529	1.567	3.807	AC-FT=	.089
ADD HYD	905.20	8& 4	2	.00980	12.75	1.090	2.08491	1.600	2.032		
ROUTE	905.30	2	6	.00980	12.74	1.090	2.08496	1.600	2.031		
ADD HYD	904.10	6& 7	19	.04170	70.16	3.968	1.78400	1.533	2.629		
COMPUTE NM HYD	701.00	-	3	.00420	11.00	.379	1.69076	1.500	4.090	PER IMP=	57.00
ROUTE RESERVOIR	501.00	3	4	.00420	8.99	.379	1.69069	1.567	3.343	AC-FT=	.056
ROUTE	701.10	4	7	.00420	8.77	.379	1.69079	1.600	3.264		
COMPUTE NM HYD	702.00	-	3	.00450	12.14	.425	1.76975	1.500	4.214	PER IMP=	65.00
ROUTE RESERVOIR	502.00	3	4	.00450	6.86	.425	1.76969	1.633	2.381	AC-FT=	.087
ADD HYD	702.10	4& 7	2	.00870	15.58	.803	1.73154	1.600	2.797		
ROUTE	702.20	2	7	.00870	15.33	.803	1.73160	1.600	2.754		
COMPUTE NM HYD	708.00	-	3	.00540	13.76	.467	1.62163	1.500	3.981	PER IMP=	50.00
ROUTE RESERVOIR	508.00	3	4	.00540	6.33	.467	1.62158	1.700	1.832	AC-FT=	.170
ADD HYD	708.10	4& 7	2	.01410	21.53	1.270	1.68941	1.633	2.385		
ROUTE	708.20	2	7	.01410	21.51	1.270	1.68944	1.633	2.384		
COMPUTE NM HYD	707.00	-	3	.00850	19.96	.646	1.42414	1.500	3.669	PER IMP=	30.00
ROUTE RESERVOIR	507.00	3	4	.00850	6.93	.646	1.42411	1.766	1.274	AC-FT=	.299
ADD HYD	707.10	4& 7	2	.02260	28.03	1.916	1.58960	1.667	1.938		
ROUTE	707.20	2	7	.02260	27.48	1.916	1.58962	1.700	1.900		
COMPUTE NM HYD	711.00	-	3	.02870	67.26	3.238	2.11537	1.500	4.751	PER IMP=	100.00
ROUTE RESERVOIR	511.00	3	4	.02870	41.11	3.238	2.11536	1.667	2.238	AC-FT=	.848
ADD HYD	711.10	4& 7	2	.05130	68.58	5.154	1.86373	1.700	2.089		
ROUTE	711.20	2	7	.05130	67.84	5.154	1.88374	1.766	2.066		
COMPUTE NM HYD	709.00	-	3	.01400	20.33	1.211	1.62164	1.733	2.269	PER IMP=	50.00
ROUTE RESERVOIR	509.00	3	4	.01400	14.88	1.211	1.62159	1.933	1.661	AC-FT=	.248
ROUTE	709.10	4	6	.01400	14.85	1.211	1.62162	1.966	1.658		
ADD HYD	709.20	6& 7	2	.06530	80.99	6.365	1.82753	1.833	1.938		
COMPUTE NM HYD	713.00	-	3	.00460	11.04	.362	1.47351	1.500	3.749	PER IMP=	35.00
ROUTE RESERVOIR	513.00	3	4	.00460	8.37	.361	1.47345	1.600	2.843	AC-FT=	.052
ADD HYD	713.10	2& 4	5	.06990	86.32	6.726	1.80422	1.800	1.930		
ROUTE	713.20	5	7	.06990	84.63	6.726	1.80423	1.866	1.892		
ADD HYD	902.10	19& 7	6	.11160	126.41	10.694	1.79667	1.733	1.770		
COMPUTE NM HYD	902.00	-	3	.02850	41.06	2.285	1.50314	1.700	2.251	PER IMP=	38.00
ROUTE RESERVOIR	502.00	3	4	.02850	25.08	2.285	1.50312	1.966	1.375	AC-FT=	.351
ADD HYD	902.10	6& 4	5	.14010	150.84	12.979	1.73695	1.733	1.682		
ROUTE	902.20	5	7	.14010	150.72	12.979	1.73695	1.766	1.681		
COMPUTE NM HYD	901.00	-	3	.02080	40.91	1.229	1.10807	1.500	3.073	PER IMP=	10.00
ROUTE RESERVOIR	501.00	3	4	.02080	8.32	1.229	1.10806	1.900	.625	AC-FT=	.770
ROUTE	901.20	4	6	.02080	7.94	1.229	1.10800	2.033	.597		
COMPUTE NM HYD	901.10	-	3	.04650	23.47	2.842	1.14585	2.266	.789	PER IMP=	10.00
ROUTE RESERVOIR	501.10	3	4	.04650	18.86	2.842	1.14585	2.600	.634	AC-FT=	.606
ADD HYD	901.12	6& 4	2	.06730	25.14	4.071	1.13413	2.833	.834		
ADD HYD	901.98	2& 7	4	.20740	161.24	17.049	1.54134	1.833	1.215		
ROUTE	901.99	4	7	.20740	161.17	17.049	1.54134	1.866	1.214		
COMPUTE NM HYD	907.00	-	3	.03440	12.45	1.226	.66816	1.933	.565	PER IMP=	5.00
ADD HYD	907.10	3& 7	2	.24180	173.41	18.275	1.41711	1.900	1.121		

AHYMO SUMMARY TABLE (AHYM0491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/17/1994

INPUT FILE = futpondf.dat USER NO.= MOLZ@ONM.101

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	1.570
COMPUTE NM HYD	603.00	-	3	.00970	14.78	.464	.89598	1.500	2.381	PER IMP=	47.00
ROUTE RESERVOIR	503.00	3	4	.00970	4.83	.464	.89595	1.766	.778	AC-FT=	.200
ROUTE	603.10	4	7	.00970	4.82	.464	.89597	1.900	.777		
COMPUTE NM HYD	602.00	-	3	.01240	19.33	.615	.92930	1.500	2.436	PER IMP=	51.00
ROUTE RESERVOIR	502.00	3	4	.01240	15.48	.615	.92928	1.567	1.951	AC-FT=	.080
ADD HYD	602.10	4& 7	2	.02210	19.80	1.078	.91463	1.567	1.400		
ROUTE RESERVOIR	501.00	2	4	.02210	13.74	1.078	.91463	1.766	.971	AC-FT=	.104
ROUTE	602.20	4	7	.02210	13.74	1.078	.91466	1.833	.971		
COMPUTE NM HYD	601.00	-	3	.01020	16.45	.533	.97928	1.500	2.520	PER IMP=	57.00
ROUTE RESERVOIR	501.00	3	4	.01020	10.98	.533	.97925	1.600	1.682	AC-FT=	.124
ADD HYD	601.10	4& 7	2	.03230	24.55	1.611	.93503	1.600	1.188		
ROUTE	601.20	2	7	.03230	24.38	1.611	.93505	1.633	1.180		
ROUTE	601.21	7	5	.03230	24.06	1.611	.93505	1.665	1.164		
COMPUTE NM HYD	609.00	-	3	.02360	34.82	1.264	1.00427	1.533	2.305	PER IMP=	60.00
ROUTE RESERVOIR	513.00	3	4	.02360	19.77	1.231	.97834	1.700	1.309	AC-FT=	.442
ADD HYD	613.10	3& 4	6	.05590	43.74	2.841	.95294	1.698	1.222		
COMPUTE NM HYD	605.00	-	3	.02000	26.90	.938	.87932	1.533	2.102	PER IMP=	45.00
ROUTE RESERVOIR	505.00	3	4	.02000	12.99	.906	.84961	1.733	1.015	AC-FT=	.391
ROUTE	605.20	4	5	.02000	12.91	.906	.84931	1.765	1.008		
ADD HYD	605.10	5& 6	2	.07590	56.30	3.747	.92564	1.698	1.159		
ROUTE	605.20	2	7	.07590	55.38	3.747	.92552	1.733	1.150		
COMPUTE NM HYD	606.00	-	3	.01980	21.01	.973	.92097	1.633	1.658	PER IMP=	50.00
ROUTE RESERVOIR	506.00	3	4	.01980	10.66	.940	.88986	1.900	.841	AC-FT=	.405
ROUTE	606.10	4	5	.01980	10.59	.939	.88949	1.933	.836		
COMPUTE NM HYD	604.00	-	3	.03860	37.25	1.668	.81004	1.633	1.508	PER IMP=	40.00
ROUTE RESERVOIR	504.00	3	4	.03860	31.64	1.635	.79401	1.733	1.281	AC-FT=	.341
ROUTE	606.10	4	6	.03860	30.76	1.634	.79376	1.766	1.245		
ADD HYD	606.20	5& 6	2	.05840	39.56	2.573	.82622	1.800	1.058		
COMPUTE NM HYD	606.10	-	3	.01120	19.07	.635	1.06258	1.500	2.660	PER IMP=	67.00
ROUTE RESERVOIR	501.00	3	4	.01120	10.20	.605	1.01209	1.633	1.423	AC-FT=	.245
ADD HYD	606.20	2& 4	2	.06960	48.59	3.178	.85613	1.766	1.091		
ROUTE	606.30	2	3	.06960	48.03	3.177	.85588	1.800	1.078		
ADD HYD	606.20	7& 3	9	.14550	102.98	6.924	.89221	1.766	1.106		
COMPUTE NM HYD	609.00	-	3	.01300	22.48	.754	1.08757	1.500	2.702	PER IMP=	70.00
ROUTE RESERVOIR	508.00	3	4	.01300	16.57	.723	1.04263	1.600	1.992	AC-FT=	.215
ADD HYD	606.20	8& 4	1	.15850	113.66	7.646	.90454	1.733	1.120		
ROUTE	606.30	1	20	.15850	113.47	7.645	.90443	1.766	1.119		
COMPUTE NM HYD	610.00	-	3	.03900	36.77	1.685	.81004	1.633	1.473	PER IMP=	40.00
ROUTE RESERVOIR	508.00	3	4	.03900	22.51	1.652	.79403	1.833	.902	AC-FT=	.573
COMPUTE NM HYD	611.00	-	3	.03240	50.61	1.879	1.08757	1.533	2.441	PER IMP=	70.00
ROUTE RESERVOIR	508.00	3	5	.03240	31.66	1.846	1.06851	1.667	1.527	AC-FT=	.571
ROUTE	606.30	5	6	.03240	31.44	1.846	1.06834	1.700	1.516		
ADD HYD	608.10	4& 6	5	.07140	51.67	3.498	.91851	1.766	1.131		
COMPUTE NM HYD	607.00	-	3	.02100	29.16	1.031	.92097	1.533	2.170	PER IMP=	50.00
ROUTE RESERVOIR	508.00	3	4	.02100	17.57	.999	.89209	1.667	1.308	AC-FT=	.354
ADD HYD	608.10	20& 5	8	.22990	165.13	11.143	.90880	1.766	1.122		
ADD HYD	608.10	4& 8	10	.25090	181.45	12.142	.90740	1.766	1.130		
ROUTE	606.30	10	7	.25090	181.12	12.140	.90722	1.766	1.128		
COMPUTE NM HYD	612.00	-	3	.03550	70.87	2.532	1.33747	1.500	3.119	PER IMP=	100.00
ROUTE RESERVOIR	508.00	3	4	.03550	58.47	2.500	1.32019	1.567	2.573	AC-FT=	.464
ADD HYD	608.10	4& 7	2	.28640	216.02	14.639	.95841	1.733	1.179		
ROUTE	606.30	2	7	.28640	215.60	14.637	.95828	1.766	1.176		
COMPUTE NM HYD	614.00	-	3	.01500	25.26	.837	1.04592	1.500	2.631	PER IMP=	65.00
ROUTE RESERVOIR	508.00	3	4	.01500	16.89	.805	1.00640	1.600	1.760	AC-FT=	.261
ADD HYD	608.10	7& 4	2	.30140	228.82	15.442	.96067	1.733	1.186		

COMPUTE NM HYD	513.00	-	3	.04000	78.74	2.830	1.33747	1.500	3.005	PER IMP=	100.00
ROUTE RESERVOIR	508.00	3	4	.04000	65.79	2.320	1.32207	1.567	2.570	AC-FT=	.477
ADD HYD	608.10	2& 4	2	.34140	272.80	18.263	1.00302	1.700	1.249		
ROUTE	606.30	2	20	.34140	272.32	18.261	1.00290	1.733	1.246		
COMPUTE NM HYD	704.00	-	3	.01300	18.62	.667	.96262	1.533	2.238	PER IMP=	55.00
ROUTE RESERVOIR	504.00	3	4	.01300	13.03	.667	.96259	1.633	1.566	AC-FT=	.164
ROUTE	704.10	4	7	.01300	12.56	.667	.96248	1.700	1.509		
COMPUTE NM HYD	703.00	-	3	.00790	12.60	.406	.96262	1.500	2.493	PER IMP=	55.00
ROUTE RESERVOIR	503.00	3	4	.00790	9.92	.406	.96256	1.567	1.962	AC-FT=	.061
ADD HYD	703.10	4& 7	2	.02090	20.98	1.073	.96240	1.633	1.569		
COMPUTE NM HYD	729.00	-	3	.01400	22.95	.750	1.00427	1.500	2.562	PER IMP=	60.00
ROUTE RESERVOIR	509.00	3	4	.01400	21.33	.750	1.00425	1.533	2.381	AC-FT=	.075
ADD HYD	709.10	2& 4	2	.03490	39.33	1.822	.97914	1.567	1.761		
ROUTE	709.20	2	7	.03490	39.04	1.823	.97915	1.600	1.748		
COMPUTE NM HYD	701.00	-	3	.01000	18.54	.642	1.20419	1.500	2.897	PER IMP=	84.00
ROUTE RESERVOIR	501.00	3	4	.01000	16.05	.642	1.20416	1.567	2.507	AC-FT=	.068
ADD HYD	709.10	4& 7	2	.04490	54.39	2.465	1.02921	1.567	1.393		
ROUTE	701.10	2	7	.04490	54.33	2.465	1.02922	1.600	1.891		
COMPUTE NM HYD	702.00	-	3	.00500	8.67	.301	1.12922	1.500	2.773	PER IMP=	75.00
ROUTE RESERVOIR	502.00	3	4	.00500	8.80	.301	1.12916	1.500	2.750	AC-FT=	.014
ADD HYD	702.10	4& 7	2	.04990	61.14	2.766	1.03919	1.567	1.915		
ROUTE	702.20	2	7	.04990	58.39	2.766	1.03920	1.633	1.828		
COMPUTE NM HYD	709.00	-	3	.02000	22.66	.894	.83767	1.567	1.770	PER IMP=	40.00
ROUTE RESERVOIR	509.00	3	4	.02000	21.62	.893	.83765	1.633	1.689	AC-FT=	.077
ADD HYD	702.10	4& 7	2	.06990	80.01	3.659	.98150	1.633	1.788		
ROUTE	709.10	2	6	.06990	79.56	3.659	.98150	1.633	1.778		
COMPUTE NM HYD	707.00	-	3	.00500	7.76	.246	.92097	1.500	2.424	PER IMP=	50.00
ROUTE RESERVOIR	507.00	3	4	.00500	3.09	.246	.92091	1.700	.964	AC-FT=	.105
ROUTE	707.20	4	7	.00500	3.01	.246	.92088	1.766	.941		
COMPUTE NM HYD	711.00	-	3	.03680	57.08	2.625	1.33747	1.533	2.948	PER IMP=	100.00
ROUTE RESERVOIR	511.00	3	4	.03680	23.17	2.625	1.33746	1.900	.984	AC-FT=	1.145
ADD HYD	711.10	4& 7	2	.04180	26.09	2.870	1.28760	1.866	.975		
ROUTE	711.20	2	7	.04180	25.98	2.871	1.28761	1.900	.971		
COMPUTE NM HYD	713.00	-	3	.00320	6.39	.228	1.33747	1.500	3.122	PER IMP=	100.00
ROUTE RESERVOIR	513.00	3	4	.00320	5.22	.228	1.33737	1.567	2.549	AC-FT=	.020
ADD HYD	713.10	7& 4	5	.04500	27.99	3.099	1.29114	1.866	.972		
ROUTE	713.20	5	7	.04500	28.00	3.099	1.29115	1.866	.972		
ADD HYD	713.10	6& 7	5	.11490	104.97	6.758	1.10277	1.633	1.427		
ROUTE	713.20	5	7	.11490	105.08	6.758	1.10277	1.667	1.429		
COMPUTE NM HYD	901.00	-	3	.02100	31.80	.994	.88765	1.500	2.366	PER IMP=	46.00
ROUTE RESERVOIR	501.00	3	4	.02100	18.21	.994	.88764	1.633	1.355	AC-FT=	.310
ADD HYD	713.10	7& 4	5	.13590	123.08	7.752	1.06952	1.667	1.415		
ROUTE	713.20	5	7	.13590	122.91	7.752	1.06953	1.667	1.413		
ADD HYD	713.10	20& 7	1	.47730	393.45	26.013	1.02187	1.700	1.288		
COMPUTE NM HYD	615.00	-	3	.02850	56.90	2.033	1.33747	1.500	3.119	PER IMP=	100.00
ROUTE RESERVOIR	508.00	3	4	.02850	43.93	2.000	1.31599	1.567	2.408	AC-FT=	.440
ADD HYD	608.10	4& 1	2	.50580	427.58	28.013	1.03845	1.700	1.321		
COMPUTE NM HYD	616.00	-	3	.01500	25.26	.837	1.04592	1.500	2.631	PER IMP=	65.00
ROUTE RESERVOIR	508.00	3	4	.01500	16.89	.805	1.00640	1.600	1.760	AC-FT=	.261
ADD HYD	608.10	2& 4	1	.52080	442.29	28.818	1.03752	1.700	1.327		
ROUTE	606.30	1	19	.52080	442.40	28.815	1.03739	1.700	1.327		
COMPUTE NM HYD	906.00	-	3	.01920	34.05	1.156	1.12922	1.500	2.771	PER IMP=	75.00
ROUTE	906.10	3	7	.01920	32.82	1.156	1.12923	1.500	2.671		
COMPUTE NM HYD	903.00	-	3	.01270	17.42	.511	.75437	1.500	2.143	PER IMP=	30.00
ROUTE RESERVOIR	503.00	3	4	.01270	7.27	.511	.75435	1.700	.894	AC-FT=	.220
ADD HYD	903.10	4& 7	2	.03190	37.56	1.667	.97995	1.533	1.840		
ROUTE	903.20	2	7	.03190	37.02	1.667	.97997	1.533	1.813		
COMPUTE NM HYD	905.00	-	3	.00600	11.72	.415	1.29582	1.500	3.052	PER IMP=	95.00
ROUTE RESERVOIR	505.00	3	4	.00600	3.16	.415	1.29577	1.933	.823	AC-FT=	.221
ROUTE	905.10	4	9	.00600	3.16	.415	1.29563	1.933	.823		
COMPUTE NM HYD	904.00	-	3	.00380	7.59	.271	1.33747	1.500	3.121	PER IMP=	100.00
ROUTE RESERVOIR	504.00	3	4	.00380	5.55	.271	1.33739	1.600	2.281	AC-FT=	.066
ADD HYD	905.20	8& 4	2	.00980	7.91	.686	1.31173	1.600	1.262		
ROUTE	905.30	2	6	.00980	7.89	.686	1.31179	1.633	1.257		
ADD HYD	904.10	6& 7	18	.04170	43.13	2.353	1.05792	1.567	1.616		
COMPUTE NM HYD	902.00	-	3	.04310	38.50	2.040	.98745	1.700	1.384	PER IMP=	44.00

ADD HYD	902.10	18& 4	5	.08480	57.09	4.360	.96393	1.766	1.052		
ROUTE	902.20	5	7	.08480	56.93	4.359	.96386	1.766	1.049		
COMPUTE NM HYD	901.10	-	3	.04490	40.86	2.126	.88765	1.700	1.422	PER IMP=	46.00
ROUTE RESERVOIR	501.10	3	4	.04490	33.55	2.092	.87359	1.800	1.167	AC-FT=	.462
ADD HYD	901.98	7& 4	2	.12970	90.32	6.451	.93261	1.300	1.088		
ROUTE	901.99	2	7	.12970	89.83	6.450	.93250	1.833	1.082		
ADD HYD	901.98	19& 7	20	.65050	523.16	35.320	1.01807	1.733	1.257		
COMPUTE NM HYD	1101.00	-	3	.01400	27.95	.999	1.33747	1.500	3.120	PER IMP=	100.00
ROUTE RESERVOIR	501.00	3	4	.01400	9.28	.999	1.33744	1.833	1.036	AC-FT=	.464
COMPUTE NM HYD	1102.00	-	3	.01800	35.94	1.284	1.33747	1.500	3.120	PER IMP=	100.00
ROUTE RESERVOIR	501.00	3	5	.01800	11.93	1.284	1.33745	1.833	1.036	AC-FT=	.597
ADD HYD	901.98	4& 5	6	.03200	21.21	2.282	1.33739	1.833	1.036		
ROUTE	901.99	6	7	.03200	21.21	2.283	1.33740	1.833	1.036		
COMPUTE NM HYD	1104.00	-	3	.02200	35.67	1.159	.98761	1.500	2.533	PER IMP=	58.00
ROUTE RESERVOIR	501.10	3	4	.02200	22.09	1.126	.96005	1.633	1.569	AC-FT=	.373
ROUTE	901.99	4	6	.02200	19.37	1.125	.95916	1.733	1.375		
ADD HYD	901.98	7& 6	8	.05400	40.11	3.408	1.18330	1.733	1.160		
COMPUTE NM HYD	1103.00	-	3	.09420	113.03	5.129	1.02093	1.600	1.875	PER IMP=	62.00
ROUTE RESERVOIR	501.10	3	4	.09420	107.08	5.095	1.01415	1.667	1.776	AC-FT=	.517
ADD HYD	901.98	8& 4	9	.14820	145.47	8.503	1.07578	1.667	1.534		
ROUTE	901.99	9	7	.14820	144.02	8.502	1.07570	1.700	1.518		
ADD HYD	901.98	20& 7	2	.79870	666.40	43.773	1.02760	1.700	1.304		
COMPUTE NM HYD	1105.00	-	3	.14650	36.74	4.448	.56934	2.266	.392	PER IMP=	20.00
ADD HYD	901.98	2& 3	4	.94520	677.07	48.216	.95646	1.700	1.119		
ROUTE RESERVOIR	501.10	4	5	.94520	73.14	48.146	.95508	2.966	.121	AC-FT=	34.861

AHYMO SUMMARY TABLE (AHYMO491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/19/1994

INPUT FILE [redacted] USER NO.= MOLZCONM.I01

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	1.570
COMPUTE NM HYD	603.00	-	3	.00970	14.78	.464	.89598	1.500	2.381	PER IMP=	47.00
ROUTE RESERVOIR	503.00	3	4	.00970	4.83	.464	.89595	1.766	.778	AC-FT=	.200
ROUTE	603.10	4	7	.00970	4.82	.464	.89597	1.800	.777		
COMPUTE NM HYD	602.00	-	3	.01240	19.33	.615	.92930	1.500	2.436	PER IMP=	51.00
ROUTE RESERVOIR	502.00	3	4	.01240	15.48	.615	.92928	1.567	1.951	AC-FT=	.050
ADD HYD	602.10	4& 7	2	.02210	19.80	1.078	.91463	1.567	1.400		
ROUTE RESERVOIR	501.00	2	4	.02210	13.74	1.078	.91463	1.766	.971	AC-FT=	.104
ROUTE	602.20	4	7	.02210	13.74	1.078	.91466	1.833	.971		
COMPUTE NM HYD	601.00	-	3	.01020	16.45	.533	.97928	1.500	2.520	PER IMP=	57.00
ROUTE RESERVOIR	501.00	3	4	.01020	10.98	.533	.97925	1.600	1.682	AC-FT=	.124
ADD HYD	601.10	4& 7	2	.03230	24.55	1.611	.93503	1.600	1.188		
ROUTE	601.20	2	7	.03230	24.38	1.611	.93505	1.633	1.180		
ROUTE	601.21	7	5	.03230	24.06	1.611	.93505	1.665	1.164		
COMPUTE NM HYD	609.00	-	3	.02360	34.82	1.264	1.00427	1.533	2.305	PER IMP=	60.00
ROUTE RESERVOIR	513.00	3	4	.02360	19.77	1.231	.97834	1.700	1.309	AC-FT=	.442
ADD HYD	613.10	5& 4	6	.05590	43.74	2.841	.95294	1.698	1.222		
COMPUTE NM HYD	605.00	-	3	.02000	26.90	.938	.87932	1.533	2.102	PER IMP=	45.00
ROUTE RESERVOIR	505.00	3	4	.02000	12.99	.906	.84961	1.733	1.015	AC-FT=	.391
ROUTE	605.20	4	5	.02000	12.91	.906	.84931	1.765	1.008		
ADD HYD	605.10	5& 6	2	.07590	56.30	3.747	.92564	1.698	1.159		
ROUTE	605.20	2	7	.07590	55.88	3.747	.92552	1.733	1.150		
COMPUTE NM HYD	606.00	-	3	.01980	21.01	.973	.92097	1.633	1.658	PER IMP=	50.00
ROUTE RESERVOIR	506.00	3	4	.01980	10.66	.940	.88986	1.900	.841	AC-FT=	.405
ROUTE	606.10	4	5	.01980	10.59	.939	.88949	1.933	.836		
COMPUTE NM HYD	604.00	-	3	.03860	37.25	1.668	.91004	1.633	1.508	PER IMP=	40.00
ROUTE RESERVOIR	504.00	3	4	.03860	31.64	1.635	.79401	1.733	1.281	AC-FT=	.341
ROUTE	606.10	4	6	.03860	30.76	1.634	.79376	1.766	1.245		
ADD HYD	606.20	5& 6	2	.05840	39.56	2.573	.82622	1.800	1.058		
COMPUTE NM HYD	606.10	-	3	.01120	19.07	.635	1.06258	1.500	2.660	PER IMP=	67.00
ROUTE RESERVOIR	501.00	3	4	.01120	10.20	.605	1.01209	1.633	1.423	AC-FT=	.245
ADD HYD	606.20	2& 4	2	.06960	48.59	3.178	.85613	1.766	1.091		
ROUTE	606.30	2	3	.06960	48.03	3.177	.85588	1.800	1.078		
ADD HYD	606.20	7& 3	8	.14550	102.98	6.924	.89221	1.766	1.106		
COMPUTE NM HYD	608.00	-	3	.01300	22.48	.754	1.08757	1.500	2.702	PER IMP=	70.00
ROUTE RESERVOIR	508.00	3	4	.01300	16.57	.723	1.04263	1.600	1.992	AC-FT=	.215
ADD HYD	606.20	8& 4	1	.15850	113.66	7.646	.90454	1.733	1.120		
ROUTE	606.30	1	20	.15850	113.47	7.645	.90443	1.766	1.119		
COMPUTE NM HYD	610.00	-	3	.03900	36.77	1.685	.81004	1.633	1.473	PER IMP=	40.00
ROUTE RESERVOIR	508.00	3	4	.03900	22.51	1.652	.79403	1.833	.902	AC-FT=	.573
COMPUTE NM HYD	611.00	-	3	.03240	50.61	1.679	1.08757	1.533	2.441	PER IMP=	70.00
ROUTE RESERVOIR	508.00	3	5	.03240	31.66	1.846	1.06851	1.667	1.527	AC-FT=	.571
ROUTE	606.30	5	6	.03240	31.44	1.846	1.06834	1.700	1.516		
ADD HYD	608.10	4& 6	5	.07140	51.67	3.498	.91851	1.766	1.131		
COMPUTE NM HYD	607.00	-	3	.02100	29.16	1.031	.92097	1.533	2.170	PER IMP=	50.00
ROUTE RESERVOIR	508.00	3	4	.02100	17.57	.999	.89209	1.667	1.308	AC-FT=	.354
ADD HYD	608.10	20& 5	8	.22990	165.13	11.143	.90880	1.766	1.122		
ADD HYD	608.10	4& 8	10	.25090	181.45	12.142	.90740	1.766	1.130		
ROUTE	606.30	10	7	.25090	181.12	12.140	.90722	1.766	1.128		
COMPUTE NM HYD	612.00	-	3	.03550	70.87	2.532	1.33747	1.500	3.119	PER IMP=	100.00
ROUTE RESERVOIR	508.00	3	4	.03550	58.47	2.500	1.32019	1.567	2.573	AC-FT=	.464
ADD HYD	608.10	4& 7	2	.28640	216.02	14.639	.95841	1.733	1.179		
ROUTE	606.30	2	7	.28640	215.60	14.637	.95828	1.766	1.176		
COMPUTE NM HYD	614.00	-	3	.01500	25.26	.837	1.04592	1.500	2.631	PER IMP=	65.00
ROUTE RESERVOIR	508.00	3	4	.01500	16.89	.805	1.00640	1.600	1.760	AC-FT=	.261
ADD HYD	608.10	7& 4	2	.30140	209.92	15.442	.96067	1.733	1.186		

ROUTE RESERVOIR	508.00	3	4	.04000	65.79	2.820	1.32207	1.567	2.570	AC-FT=	.477
ADD HYD	608.10	2& 4	2	.34140	272.80	13.263	1.00302	1.700	1.249		
ROUTE	606.30	2	20	.34140	272.32	18.261	1.00290	1.733	1.246		
COMPUTE NM HYD	704.00	-	3	.01300	18.62	.667	.96262	1.533	2.238	PER IMP=	55.00
ROUTE RESERVOIR	504.00	3	4	.01300	13.03	.667	.96259	1.633	1.666	AC-FT=	.154
ROUTE	704.10	4	7	.01300	12.56	.667	.96248	1.700	1.609		
COMPUTE NM HYD	703.00	-	3	.00790	12.60	.406	.96262	1.500	2.493	PER IMP=	55.00
ROUTE RESERVOIR	503.00	3	4	.00790	9.92	.406	.96256	1.567	1.962	AC-FT=	.081
ADD HYD	703.10	4& 7	2	.02090	20.98	1.073	.96240	1.633	1.568		
COMPUTE NM HYD	729.00	-	3	.01400	22.95	.750	1.00427	1.500	2.562	PER IMP=	60.00
ROUTE RESERVOIR	509.00	3	4	.01400	21.33	.750	1.00425	1.533	2.381	AC-FT=	.075
ADD HYD	709.10	2& 4	2	.03490	39.33	1.822	.97914	1.567	1.761		
ROUTE	709.20	2	7	.03490	39.04	1.823	.97915	1.600	1.746		
COMPUTE NM HYD	701.00	-	3	.01000	18.54	.642	1.20419	1.500	2.897	PER IMP=	84.00
ROUTE RESERVOIR	501.00	3	4	.01000	16.05	.642	1.20416	1.567	2.507	AC-FT=	.089
ADD HYD	709.10	4& 7	2	.04490	54.39	2.465	1.02921	1.567	1.893		
ROUTE	701.10	2	7	.04490	54.33	2.465	1.02922	1.600	1.891		
COMPUTE NM HYD	702.00	-	3	.00500	8.87	.301	1.12922	1.500	2.773	PER IMP=	75.00
ROUTE RESERVOIR	502.00	3	4	.00500	8.80	.301	1.12916	1.500	2.750	AC-FT=	.014
ADD HYD	702.10	4& 7	2	.04990	61.14	2.766	1.03919	1.567	1.915		
ROUTE	702.20	2	7	.04990	58.39	2.766	1.03920	1.633	1.828		
COMPUTE NM HYD	709.00	-	3	.02000	22.66	.894	.83767	1.567	1.770	PER IMP=	40.00
ROUTE RESERVOIR	509.00	3	4	.02000	21.62	.893	.83765	1.633	1.689	AC-FT=	.077
ADD HYD	702.10	4& 7	2	.06990	80.01	3.659	.98150	1.633	1.788		
ROUTE	709.10	2	6	.06990	79.56	3.659	.98150	1.633	1.778		
COMPUTE NM HYD	707.00	-	3	.00500	7.76	.246	.92097	1.500	2.424	PER IMP=	50.00
ROUTE RESERVOIR	507.00	3	4	.00500	3.09	.246	.92091	1.700	.964	AC-FT=	.105
ROUTE	707.20	4	7	.00500	3.01	.246	.92088	1.766	.941		
COMPUTE NM HYD	711.00	-	3	.03680	67.08	2.625	1.33747	1.533	2.948	PER IMP=	100.00
ROUTE RESERVOIR	511.00	3	4	.03680	23.17	2.625	1.33746	1.900	.984	AC-FT=	1.145
ADD HYD	711.10	4& 7	2	.04180	26.09	2.870	1.28760	1.866	.975		
ROUTE	711.20	2	7	.04180	25.99	2.871	1.28761	1.900	.971		
COMPUTE NM HYD	713.00	-	3	.00320	6.39	.228	1.33747	1.500	3.122	PER IMP=	100.00
ROUTE RESERVOIR	513.00	3	4	.00320	5.22	.228	1.33737	1.567	2.549	AC-FT=	.020
ADD HYD	713.10	7& 4	5	.04500	27.99	3.099	1.29114	1.866	.972		
ROUTE	713.20	5	7	.04500	28.00	3.099	1.29115	1.866	.972		
ADD HYD	713.10	6& 7	5	.11490	104.97	6.758	1.10277	1.633	1.427		
ROUTE	713.20	5	7	.11490	105.08	6.758	1.10277	1.667	1.429		
COMPUTE NM HYD	901.00	-	3	.02100	31.80	.994	.88765	1.500	2.366	PER IMP=	46.00
ROUTE RESERVOIR	501.00	3	4	.02100	18.21	.994	.88764	1.633	1.355	AC-FT=	.310
ADD HYD	713.10	7& 4	5	.13590	123.08	7.752	1.06952	1.667	1.415		
ROUTE	713.20	5	7	.13590	122.91	7.752	1.06953	1.667	1.413		
ADD HYD	713.10	20& 7	1	.47730	393.45	26.013	1.02187	1.700	1.288		
COMPUTE NM HYD	615.00	-	3	.02850	56.90	2.033	1.33747	1.500	3.119	PER IMP=	100.00
ROUTE RESERVOIR	508.00	3	4	.02850	43.93	2.000	1.31599	1.567	2.408	AC-FT=	.440
ADD HYD	608.10	4& 1	2	.50580	427.58	28.013	1.03845	1.700	1.321		
COMPUTE NM HYD	616.00	-	3	.01500	25.26	.837	1.04592	1.500	2.631	PER IMP=	65.00
ROUTE RESERVOIR	508.00	3	4	.01500	16.89	.805	1.00640	1.600	1.760	AC-FT=	.261
ADD HYD	608.10	2& 4	1	.52080	442.29	28.818	1.03752	1.700	1.327		
ROUTE	606.30	1	19	.52080	442.40	28.815	1.03739	1.700	1.327		
COMPUTE NM HYD	906.00	-	3	.01920	34.05	1.156	1.12922	1.500	2.771	PER IMP=	75.00
ROUTE	906.10	3	7	.01920	32.82	1.156	1.12923	1.500	2.671		
COMPUTE NM HYD	903.00	-	3	.01270	17.42	.511	.75437	1.500	2.143	PER IMP=	30.00
ROUTE RESERVOIR	503.00	3	4	.01270	7.27	.511	.75435	1.700	.894	AC-FT=	.220
ADD HYD	903.10	4& 7	2	.03190	37.56	1.667	.97995	1.533	1.840		
ROUTE	903.20	2	7	.03190	37.02	1.667	.97997	1.533	1.813		
COMPUTE NM HYD	905.00	-	3	.00600	11.72	.415	1.29582	1.500	3.052	PER IMP=	95.00
ROUTE RESERVOIR	505.00	3	4	.00600	3.16	.415	1.29577	1.933	.823	AC-FT=	.221
ROUTE	905.10	4	8	.00600	3.16	.415	1.29563	1.933	.823		
COMPUTE NM HYD	904.00	-	3	.00380	7.59	.271	1.33747	1.500	3.121	PER IMP=	100.00
ROUTE RESERVOIR	504.00	3	4	.00380	5.55	.271	1.33739	1.600	2.281	AC-FT=	.056
ADD HYD	905.20	8& 4	2	.00980	7.91	.686	1.31173	1.600	1.262		
ROUTE	905.30	2	6	.00980	7.89	.686	1.31179	1.633	1.257		
ADD HYD	904.10	6& 7	18	.04170	43.13	2.353	1.05792	1.567	1.616		
COMPUTE NM HYD	902.00	-	3	.04310	38.50	2.040	.88765	1.700	1.396	PER IMP=	45.00

ADD HYD	902.10	18& 4	5	.08480	57.09	4.360	.96393	1.766	1.052		
ROUTE	902.20	5	7	.08480	56.93	4.359	.96386	1.766	1.049		
COMPUTE NM HYD	901.10	-	3	.04490	40.86	2.126	.88765	1.700	1.422	PER IMP=	46.00
ROUTE RESERVOIR	501.10	3	4	.04490	33.55	2.092	.87359	1.800	1.167	AC-FT=	.462
ADD HYD	901.98	7& 4	2	.12970	90.32	6.451	.93261	1.800	1.088		
ROUTE	901.99	2	7	.12970	89.93	6.450	.93250	1.833	1.082		
ADD HYD	901.98	19& 7	20	.65050	523.16	35.320	1.01807	1.733	1.257		
COMPUTE NM HYD	1101.00	-	3	.01400	27.95	.999	1.33747	1.500	3.120	PER IMP=	100.00
ROUTE RESERVOIR	501.00	3	4	.01400	9.28	.999	1.33744	1.833	1.036	AC-FT=	.464
COMPUTE NM HYD	1102.00	-	3	.01800	35.94	1.284	1.33747	1.500	3.120	PER IMP=	100.00
ROUTE RESERVOIR	501.00	3	5	.01800	11.93	1.284	1.33745	1.833	1.036	AC-FT=	.597
ADD HYD	901.98	4& 5	6	.03200	21.21	2.282	1.33739	1.833	1.036		
ROUTE	901.99	6	7	.03200	21.21	2.283	1.33740	1.833	1.036		
COMPUTE NM HYD	1104.00	-	3	.02200	35.67	1.159	.98761	1.500	2.533	PER IMP=	58.00
ROUTE RESERVOIR	501.10	3	4	.02200	22.09	1.126	.96005	1.633	1.569	AC-FT=	.373
ROUTE	901.99	4	6	.02200	19.37	1.125	.95916	1.733	1.375		
ADD HYD	901.98	7& 6	8	.05400	40.11	3.408	1.18330	1.733	1.160		
COMPUTE NM HYD	1103.00	-	3	.09420	113.03	5.129	1.02093	1.600	1.875	PER IMP=	52.00
ROUTE RESERVOIR	501.10	3	4	.09420	107.08	5.095	1.01415	1.667	1.776	AC-FT=	.517
ADD HYD	901.98	8& 4	9	.14820	145.47	8.503	1.07578	1.667	1.534		
ROUTE	901.99	9	7	.14820	144.02	8.502	1.07570	1.700	1.518		
ADD HYD	901.98	20& 7	2	.79870	666.40	43.773	1.02760	1.700	1.304		
COMPUTE NM HYD	1105.00	-	3	.14650	36.74	4.448	.56934	2.266	.392	PER IMP=	20.00
ADD HYD	901.98	2& 3	4	.94520	577.07	48.216	.95646	1.700	1.119		
ROUTE RESERVOIR	501.10	4	5	.94520	73.14	48.146	.95508	2.966	.121	AC-FT=	34.861

AHYMO SUMMARY TABLE (AHYMO491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/17/1994

INPUT FILE = [redacted] USER NO.= HOL7CONM.101

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS·1 PER·1 ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	1.350
COMPUTE NM HYD	603.00	-	3	.00970	24.41	.824	1.59201	1.500	3.932	PER IMP=	47.00
ROUTE RESERVOIR	503.00	3	4	.00970	5.24	.824	1.59198	1.766	.845	AC-FT=	.441
ROUTE	603.10	4	7	.00970	5.24	.824	1.59200	2.033	.845		
COMPUTE NM HYD	602.00	-	3	.01240	31.69	1.079	1.63151	1.500	3.993	PER IMP=	31.00
ROUTE RESERVOIR	502.00	3	4	.01240	22.39	1.079	1.63149	1.600	2.821	AC-FT=	.171
ADD HYD	602.10	4& 7	2	.02210	27.62	1.903	1.61413	1.600	1.946		
ROUTE RESERVOIR	501.00	2	4	.02210	13.82	1.903	1.61413	2.000	.977	AC-FT=	.400
ROUTE	602.20	4	7	.02210	13.82	1.903	1.61415	2.100	.977		
COMPUTE NM HYD	601.00	-	3	.01020	26.68	.920	1.69076	1.500	4.087	PER IMP=	57.00
ROUTE RESERVOIR	501.00	3	4	.01020	13.14	.920	1.69073	1.667	2.012	AC-FT=	.260
ADD HYD	601.10	4& 7	2	.03230	26.90	2.822	1.63832	1.700	1.301		
ROUTE	601.20	2	7	.03230	26.87	2.822	1.63833	1.700	1.300		
ROUTE	601.21	7	5	.03230	26.82	2.822	1.63833	1.765	1.297		
COMPUTE NM HYD	609.00	-	3	.02360	36.32	2.165	1.72038	1.533	3.729	PER IMP=	60.00
ROUTE RESERVOIR	513.00	3	4	.02360	33.65	2.132	1.69421	1.700	2.228	AC-FT=	.684
ADD HYD	613.10	5& 4	6	.05590	60.40	4.953	1.66127	1.698	1.688		
COMPUTE NM HYD	605.00	-	3	.02000	44.79	1.677	1.57226	1.533	3.499	PER IMP=	45.00
ROUTE RESERVOIR	505.00	3	4	.02000	23.39	1.644	1.54162	1.733	1.828	AC-FT=	.625
ROUTE	605.20	4	5	.02000	23.28	1.644	1.54131	1.732	1.819		
ADD HYD	605.10	5& 6	2	.07590	83.33	6.597	1.62966	1.698	1.716		
ROUTE	605.20	2	7	.07590	82.98	6.596	1.62954	1.733	1.706		
COMPUTE NM HYD	606.00	-	3	.01980	34.96	1.712	1.62163	1.633	2.759	PER IMP=	50.00
ROUTE RESERVOIR	506.00	3	4	.01980	19.04	1.679	1.59002	1.900	1.502	AC-FT=	.646
ROUTE	606.10	4	5	.01980	18.95	1.679	1.58965	1.933	1.495		
COMPUTE NM HYD	604.00	-	3	.03860	64.21	3.051	1.48196	1.633	2.599	PER IMP=	40.00
ROUTE RESERVOIR	504.00	3	4	.03860	37.16	3.018	1.46579	1.866	1.504	AC-FT=	.778
ROUTE	606.10	4	6	.03860	37.15	3.017	1.46554	2.000	1.504		
ADD HYD	606.20	5& 6	2	.05840	56.08	4.696	1.50761	1.933	1.501		
COMPUTE NM HYD	606.10	-	3	.01120	30.40	1.069	1.78950	1.500	4.242	PER IMP=	67.00
ROUTE RESERVOIR	501.00	3	4	.01120	17.99	1.037	1.73546	1.633	2.510	AC-FT=	.357
ADD HYD	606.20	2& 4	2	.06960	69.22	5.732	1.54428	1.800	1.554		
ROUTE	606.30	2	3	.06960	68.84	5.731	1.54402	1.833	1.545		
ADD HYD	606.20	7& 3	8	.14550	150.29	12.328	1.58863	1.766	1.614		
COMPUTE NM HYD	608.00	-	3	.01300	35.67	1.261	1.81913	1.500	4.288	PER IMP=	70.00
ROUTE RESERVOIR	508.00	3	4	.01300	22.31	1.229	1.77257	1.633	2.682	AC-FT=	.326
ADD HYD	606.20	8& 4	1	.15850	172.60	13.557	1.60372	1.766	1.701		
ROUTE	606.30	1	20	.15850	171.57	13.556	1.60360	1.766	1.692		
COMPUTE NM HYD	610.00	-	3	.03900	63.47	3.082	1.48196	1.633	2.543	PER IMP=	40.00
ROUTE RESERVOIR	508.00	3	4	.03900	36.62	3.049	1.46575	1.866	1.467	AC-FT=	.991
COMPUTE NM HYD	611.00	-	3	.03240	80.49	3.143	1.81913	1.533	3.982	PER IMP=	70.00
ROUTE RESERVOIR	508.00	3	5	.03240	50.50	3.110	1.79991	1.667	2.436	AC-FT=	.882
ROUTE	606.30	5	6	.03240	50.48	3.110	1.79974	1.700	2.434		
ADD HYD	608.10	4& 6	5	.07140	87.05	6.159	1.61731	1.733	1.905		
COMPUTE NM HYD	607.00	-	3	.02100	48.06	1.816	1.62163	1.533	3.576	PER IMP=	50.00
ROUTE RESERVOIR	508.00	3	4	.02100	29.00	1.784	1.59247	1.700	2.158	AC-FT=	.553
ADD HYD	608.10	20& 5	8	.22990	257.65	19.714	1.60786	1.766	1.751		
ADD HYD	608.10	4& 8	10	.25090	286.65	21.498	1.60657	1.766	1.785		
ROUTE	606.30	10	7	.25090	285.76	21.496	1.60638	1.766	1.780		
COMPUTE NM HYD	612.00	-	3	.03550	107.94	4.005	2.11536	1.500	4.751	PER IMP=	100.00
ROUTE RESERVOIR	508.00	3	4	.03550	70.74	3.972	2.09798	1.600	3.113	AC-FT=	.777
ADD HYD	608.10	4& 7	2	.28640	354.27	25.468	1.66732	1.766	1.933		
ROUTE	606.30	2	7	.28640	354.17	25.466	1.66719	1.766	1.932		
COMPUTE NM HYD	614.00	-	3	.01500	40.41	1.416	1.76975	1.500	4.210	PER IMP=	65.00
ROUTE RESERVOIR	508.00	3	4	.01500	25.11	1.383	1.72931	1.633	2.616	AC-FT=	.392
ADD HYD	608.10	7& 4	2	.30140	377.15	26.849	1.67028	1.766	1.955		
COMPUTE NM HYD	613.00	-	3	.04000	117.23	4.513	2.11537	1.500	4.379	PER IMP=	100.00

ADD HYD	608.10	2& 4	2	.34140	453.96	31.329	1.72061	1.766	2.078			
ROUTE	606.30	2	20	.34140	453.75	31.327	1.72050	1.766	2.077			
COMPUTE NM HYD	704.00	-	3	.01300	30.40	1.159	1.67101	1.533	3.653	PER IMP=	55.00	
ROUTE RESERVOIR	504.00	3	4	.01300	18.61	1.159	1.67098	1.667	2.237	AC-FT=	.286	
ROUTE	704.10	4	7	.01300	18.60	1.158	1.67087	1.833	2.236			
COMPUTE NM HYD	703.00	-	3	.00790	20.51	.704	1.67101	1.500	4.056	PER IMP=	55.00	
ROUTE RESERVOIR	503.00	3	4	.00790	12.59	.704	1.67097	1.633	2.490	AC-FT=	.153	
ADD HYD	703.10	4& 7	2	.02090	31.19	1.862	1.67079	1.800	2.331			
COMPUTE NM HYD	729.00	-	3	.01400	37.03	1.285	1.72038	1.500	4.133	PER IMP=	60.00	
ROUTE RESERVOIR	509.00	3	4	.01400	22.94	1.285	1.72036	1.633	2.561	AC-FT=	.210	
ADD HYD	709.10	2& 4	2	.03490	54.12	3.147	1.69062	1.800	2.423			
ROUTE	709.20	2	7	.03490	54.11	3.147	1.69064	1.800	2.422			
COMPUTE NM HYD	701.00	-	3	.01000	28.83	1.044	1.95737	1.500	4.505	PER IMP=	84.00	
ROUTE RESERVOIR	501.00	3	4	.01000	18.52	1.044	1.95734	1.600	2.894	AC-FT=	.173	
ADD HYD	709.10	4& 7	2	.04490	72.56	4.191	1.74998	1.766	2.525			
ROUTE	701.10	2	7	.04490	72.56	4.191	1.74999	1.766	2.525			
COMPUTE NM HYD	702.00	-	3	.00500	13.98	.498	1.86850	1.500	4.365	PER IMP=	75.00	
ROUTE RESERVOIR	502.00	3	4	.00500	8.81	.498	1.86845	1.600	2.754	AC-FT=	.064	
ADD HYD	702.10	4& 7	2	.04990	81.37	4.689	1.76182	1.766	2.549			
ROUTE	702.20	2	7	.04990	91.12	4.689	1.76183	1.766	2.540			
COMPUTE NM HYD	709.00	-	3	.02000	38.43	1.624	1.52289	1.567	3.003	PER IMP=	40.00	
ROUTE RESERVOIR	509.00	3	4	.02000	22.64	1.624	1.52287	1.766	1.768	AC-FT=	.309	
ADD HYD	702.10	4& 7	2	.06990	103.75	6.313	1.69342	1.766	2.319			
ROUTE	709.10	2	6	.06990	103.66	6.313	1.69343	1.800	2.317			
COMPUTE NM HYD	707.00	-	3	.00500	12.74	.432	1.62163	1.500	3.981	PER IMP=	50.00	
ROUTE RESERVOIR	507.00	3	4	.00500	4.86	.432	1.62158	1.733	1.517	AC-FT=	.183	
ROUTE	707.20	4	7	.00500	4.81	.432	1.62155	1.800	1.503			
COMPUTE NM HYD	711.00	-	3	.03680	102.24	4.152	2.11536	1.533	4.341	PER IMP=	100.00	
ROUTE RESERVOIR	511.00	3	4	.03680	31.24	4.152	2.11536	1.966	1.327	AC-FT=	1.934	
ADD HYD	711.10	4& 7	2	.04180	35.93	4.584	2.05626	1.900	1.343			
ROUTE	711.20	2	7	.04180	35.88	4.584	2.05627	1.933	1.341			
COMPUTE NM HYD	713.00	-	3	.00320	9.74	.361	2.11537	1.500	4.754	PER IMP=	100.00	
ROUTE RESERVOIR	513.00	3	4	.00320	7.51	.361	2.11528	1.567	3.668	AC-FT=	.044	
ADD HYD	713.10	7& 4	5	.04500	40.09	4.945	2.06046	1.800	1.392			
ROUTE	713.20	5	7	.04500	40.09	4.945	2.06047	1.800	1.392			
ADD HYD	713.10	6& 7	5	.11490	143.75	11.258	1.83717	1.800	1.955			
ROUTE	713.20	5	7	.11490	143.69	11.258	1.83717	1.800	1.954			
COMPUTE NM HYD	901.00	-	3	.02100	52.62	1.772	1.58214	1.500	3.915	PER IMP=	46.00	
ROUTE RESERVOIR	501.00	3	4	.02100	30.94	1.772	1.58212	1.633	2.302	AC-FT=	.527	
ADD HYD	713.10	7& 4	5	.13590	170.82	13.030	1.79776	1.733	1.964			
ROUTE	713.20	5	7	.13590	170.67	13.030	1.79776	1.766	1.962			
ADD HYD	713.10	20& 7	1	.47730	624.43	44.357	1.74250	1.766	2.044			
COMPUTE NM HYD	615.00	-	3	.02850	86.66	3.215	2.11537	1.500	4.751	PER IMP=	100.00	
ROUTE RESERVOIR	508.00	3	4	.02850	56.82	3.183	2.09376	1.600	3.115	AC-FT=	.665	
ADD HYD	608.10	4& 1	2	.50580	676.66	47.539	1.76229	1.733	2.090			
COMPUTE NM HYD	616.00	-	3	.01500	40.41	1.416	1.76975	1.500	4.210	PER IMP=	65.00	
ROUTE RESERVOIR	508.00	3	4	.01500	25.11	1.383	1.72931	1.633	2.616	AC-FT=	.392	
ADD HYD	608.10	2& 4	1	.52080	701.76	48.923	1.76134	1.733	2.105			
ROUTE	606.30	1	19	.52080	699.90	48.919	1.76121	1.766	2.100			
COMPUTE NM HYD	906.00	-	3	.01920	53.63	1.913	1.86850	1.500	4.364	PER IMP=	75.00	
ROUTE	906.10	3	7	.01920	51.95	1.913	1.86851	1.500	4.228			
COMPUTE NM HYD	903.00	-	3	.01270	29.81	.965	1.42414	1.500	3.668	PER IMP=	30.00	
ROUTE RESERVOIR	503.00	3	4	.01270	12.78	.965	1.42412	1.700	1.572	AC-FT=	.396	
ADD HYD	903.10	4& 7	2	.03190	59.95	2.878	1.69156	1.533	2.936			
ROUTE	903.20	2	7	.03190	59.22	2.878	1.69157	1.533	2.901			
COMPUTE NM HYD	905.00	-	3	.00600	17.96	.661	2.06599	1.500	4.678	PER IMP=	95.00	
ROUTE RESERVOIR	505.00	3	4	.00600	4.94	.661	2.06594	1.933	1.288	AC-FT=	.346	
ROUTE	905.10	4	8	.00600	4.94	.661	2.06580	1.933	1.288			
COMPUTE NM HYD	904.00	-	3	.00380	11.56	.429	2.11536	1.500	4.754	PER IMP=	100.00	
ROUTE RESERVOIR	504.00	3	4	.00380	9.26	.429	2.11529	1.567	3.807	AC-FT=	.089	
ADD HYD	905.20	8& 4	2	.00980	12.75	1.090	2.08491	1.600	2.032			
ROUTE	905.30	2	6	.00980	12.74	1.090	2.08496	1.600	2.031			
ADD HYD	904.10	6& 7	18	.04170	70.16	3.968	1.78400	1.533	2.629			
COMPUTE NM HYD	902.00	-	3	.04310	64.96	3.637	1.58214	1.700	2.355	PER IMP=	46.00	
ROUTE RESERVOIR	502.00	3	4	.04310	38.46	3.603	1.56726	1.966	1.394	AC-FT=	.952	

ROUTE	902.20	5	7	.08480	92.35	7.370	1.67377	1.667	1.702	
COMPUTE NM HYD	901.10	-	3	.04490	68.89	3.789	1.58214	1.700	2.397 PER IMP=	46.00
ROUTE RESERVOIR	501.10	3	4	.04490	40.77	3.755	1.56788	1.966	1.419 AC-FT=	.982
ADD HYD	901.98	7& 4	2	.12970	133.06	11.324	1.63711	1.667	1.603	
ROUTE	901.99	2	7	.12970	131.27	11.324	1.63700	1.700	1.581	
ADD HYD	901.98	19& 7	20	.65050	827.64	60.298	1.73903	1.766	1.988	
COMPUTE NM HYD	1101.00	-	3	.01400	42.57	1.579	2.11537	1.500	4.751 PER IMP=	100.00
ROUTE RESERVOIR	501.00	3	4	.01400	14.37	1.579	2.11534	1.833	1.604 AC-FT=	.719
COMPUTE NM HYD	1102.00	-	3	.01800	54.73	2.031	2.11536	1.500	4.751 PER IMP=	100.00
ROUTE RESERVOIR	501.00	3	5	.01800	18.48	2.031	2.11535	1.833	1.604 AC-FT=	.924
ADD HYD	901.98	4& 5	6	.03200	32.85	3.610	2.11529	1.833	1.604	
ROUTE	901.99	6	7	.03200	32.65	3.610	2.11530	1.833	1.604	
COMPUTE NM HYD	1104.00	-	3	.02200	57.74	1.995	1.70063	1.500	4.101 PER IMP=	58.00
ROUTE RESERVOIR	501.10	3	4	.02200	35.61	1.963	1.67289	1.633	2.529 AC-FT=	.574
ROUTE	901.99	4	6	.02200	33.72	1.962	1.67199	1.733	2.395	
ADD HYD	901.98	7& 6	8	.05400	65.92	5.572	1.93469	1.733	1.907	
COMPUTE NM HYD	1103.00	-	3	.09420	185.35	8.742	1.74013	1.600	3.074 PER IMP=	52.00
ROUTE RESERVOIR	501.10	3	4	.09420	113.03	8.709	1.73327	1.800	1.875 AC-FT=	1.627
ADD HYD	901.98	8& 4	9	.14820	178.94	14.280	1.80666	1.733	1.887	
ROUTE	901.99	9	7	.14820	178.68	14.279	1.80658	1.766	1.894	
ADD HYD	901.98	20& 7	2	.79870	1006.32	74.528	1.74959	1.766	1.969	
COMPUTE NM HYD	1105.00	-	3	.14650	76.61	9.160	1.17239	2.266	.817 PER IMP=	20.00
ADD HYD	901.98	2& 3	4	.94520	1037.47	83.682	1.66001	1.766	1.715	
ROUTE RESERVOIR	501.10	4	5	.94520	95.18	83.358	1.65358	3.200	.157 AC-FT=	63.798

APPENDIX C

SUMMARY PRINTOUTS
BASIN W3

AHYMO SUMMARY TABLE (AHYM0491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/21/1994
INPUT FILE = [redacted] USER NO.= MOL2CONM.101

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	1.570
COMPUTE NM HYD	101.00	-	3	.01400	26.71	.936	1.25417	1.500	2.981	PER IMP=	90.00
ROUTE RESERVOIR	501.00	3	4	.01400	8.78	.936	1.25414	1.833	.980	AC-FT=	.439
COMPUTE NM HYD	106.00	-	3	.02360	9.16	.507	.40261	1.733	.606	PER IMP=	3.00
ADD HYD	106.10	3& 4	2	.03760	17.81	1.443	.71965	1.733	.740		
ROUTE	106.20	2	7	.03760	17.79	1.443	.71966	1.766	.739		
COMPUTE NM HYD	105.00	-	3	.01400	19.20	.563	.75437	1.500	2.143	PER IMP=	30.00
ROUTE RESERVOIR	505.00	3	4	.01400	14.35	.563	.75435	1.600	1.601	AC-FT=	.121
ADD HYD	105.10	4& 7	2	.05160	29.38	2.006	.72906	1.667	.890		
ROUTE	105.20	2	7	.05160	29.33	2.006	.72907	1.667	.888		
COMPUTE NM HYD	102.00	-	3	.00370	7.39	.264	1.33747	1.500	3.121	PER IMP=	100.00
ROUTE RESERVOIR	502.00	3	4	.00370	6.98	.264	1.33738	1.533	2.947	AC-FT=	.019
ROUTE	102.10	4	6	.00370	6.90	.264	1.33754	1.533	2.912		
COMPUTE NM HYD	103.00	-	3	.00380	3.39	.166	.82101	1.600	1.601	PER IMP=	38.00
ROUTE RESERVOIR	503.00	3	4	.00380	3.19	.166	.82088	1.700	1.312	AC-FT=	.030
ADD HYD	103.10	4& 6	2	.00750	8.88	.430	1.07568	1.567	1.850		
ADD HYD	103.20	2& 7	5	.05910	37.10	2.437	.77305	1.633	.981		
COMPUTE NM HYD	108.00	-	3	.02140	17.58	.561	.49134	1.567	1.283	PER IMP=	5.00
ROUTE RESERVOIR	508.00	3	4	.02140	9.10	.561	.49132	1.733	.664	AC-FT=	.150
ROUTE	108.10	4	6	.02140	8.89	.561	.49135	1.833	.649		
ADD HYD	108.20	5& 6	2	.08050	45.15	2.997	.69815	1.667	.876		
ROUTE	103.30	2	7	.08050	44.72	2.997	.69816	1.700	.868		
COMPUTE NM HYD	104.00	-	3	.05000	28.33	1.422	.53310	1.700	.885	PER IMP=	10.00
ADD HYD	104.10	3& 7	2	.13050	73.04	4.419	.63491	1.700	.875		

AHYMO SUMMARY TABLE (AHYM0491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/21/1994

INPUT FILE = [redacted] USER NO.= MOLZCONM.I01

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	2.350
COMPUTE NM HYD	101.00	-	3	.01400	41.19	1.506	2.01662	1.500	4.597	PER IMP=	90.00
ROUTE RESERVOIR	501.00	3	4	.01400	13.83	1.506	2.01660	1.833	1.544	AC-FT=	.692
COMPUTE NM HYD	106.00	-	3	.02360	20.74	1.217	.96676	1.733	1.373	PER IMP=	3.00
ADD HYD	106.10	3& 4	2	.03760	34.40	2.723	1.35763	1.766	1.430		
ROUTE	106.20	2	7	.03760	34.36	2.723	1.35764	1.766	1.428		
COMPUTE NM HYD	105.00	-	3	.01400	32.85	1.063	1.42414	1.500	3.668	PER IMP=	30.00
ROUTE RESERVOIR	505.00	3	4	.01400	24.11	1.063	1.42412	1.600	2.691	AC-FT=	.214
ADD HYD	105.10	4& 7	2	.05160	54.83	3.786	1.37567	1.667	1.660		
ROUTE	105.20	2	7	.05160	54.46	3.786	1.37568	1.667	1.649		
COMPUTE NM HYD	102.00	-	3	.00370	11.26	.417	2.11536	1.500	4.754	PER IMP=	100.00
ROUTE RESERVOIR	502.00	3	4	.00370	9.51	.417	2.11529	1.567	4.018	AC-FT=	.036
ROUTE	102.10	4	6	.00370	9.52	.417	2.11543	1.567	4.020		
COMPUTE NM HYD	103.00	-	3	.00380	6.66	.305	1.50314	1.600	2.737	PER IMP=	38.00
ROUTE RESERVOIR	503.00	3	4	.00380	5.52	.305	1.50303	1.700	2.268	AC-FT=	.052
ADD HYD	103.10	4& 6	2	.00750	13.83	.722	1.80506	1.633	2.881		
ADD HYD	103.20	2& 7	5	.05910	67.92	4.508	1.43016	1.667	1.796		
COMPUTE NM HYD	108.00	-	3	.02140	34.21	1.251	1.09630	1.567	2.498	PER IMP=	5.00
ROUTE RESERVOIR	508.00	3	4	.02140	9.40	1.251	1.09628	1.933	.686	AC-FT=	.586
ROUTE	108.10	4	6	.02140	9.35	1.251	1.09630	2.333	.682		
ADD HYD	108.20	5& 6	2	.08050	76.62	5.759	1.34140	1.667	1.487		
ROUTE	103.30	2	7	.08050	75.28	5.759	1.34140	1.700	1.461		
COMPUTE NM HYD	104.00	-	3	.05000	56.03	3.056	1.14585	1.700	1.751	PER IMP=	10.00
ADD HYD	104.10	3& 7	2	.13050	131.31	8.815	1.26647	1.700	1.572		

APPENDIX C

SUMMARY PRINTOUTS
BASIN UE

AHYMO SUMMARY TABLE (AHYM0491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/21/1994

INPUT FILE = [redacted] USER NO.= MOLZCONM.I01

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	1.570
COMPUTE NM HYD	201.00	-	3	.04590	20.48	.584	.23872	1.533	.697	PER IMP=	5.00
COMPUTE NM HYD	202.00	-	7	.02330	12.47	.362	.29094	1.500	.836	PER IMP=	8.00
ADD HYD	202.10	3& 7	2	.06920	32.90	.946	.25629	1.533	.743		
ROUTE	202.30	2	7	.06920	33.16	.946	.25630	1.533	.749		
ROUTE	202.40	7	4	.06920	26.44	.946	.25630	1.600	.597		
COMPUTE NM HYD	203.00	-	3	.02780	12.17	.382	.25785	1.500	.684	PER IMP=	10.00
ROUTE	203.10	3	7	.02780	12.16	.382	.25786	1.533	.684		
ROUTE	203.20	7	3	.02780	9.33	.382	.25786	1.600	.524		
ADD HYD	203.30	3& 4	2	.09700	35.77	1.328	.25674	1.600	.576		
COMPUTE NM HYD	205.00	-	3	.10450	36.33	1.671	.29979	1.600	.543	PER IMP=	10.00
ADD HYD	205.10	3& 2	5	.20150	72.10	2.999	.27907	1.600	.559		
ROUTE RESERVOIR	505.00	5	4	.20150	28.25	2.999	.27907	1.900	.219	AC-FT=	1.120
ROUTE	205.20	4	7	.20150	28.25	2.999	.27907	1.933	.219		
ROUTE	205.30	7	6	.20150	28.14	2.999	.27907	2.100	.218		
COMPUTE NM HYD	204.00	-	3	.05440	23.80	.748	.25785	1.500	.684	PER IMP=	10.00
ROUTE	204.10	3	7	.05440	23.71	.748	.25786	1.500	.681		
ROUTE	204.20	7	4	.05440	19.85	.748	.25786	1.567	.570		
COMPUTE NM HYD	207.00	-	3	.04840	24.21	.874	.33867	1.533	.781	PER IMP=	15.00
ADD HYD	207.10	3& 4	2	.10280	43.34	1.622	.29590	1.567	.659		
ROUTE	207.20	2	7	.10280	43.37	1.622	.29590	1.567	.659		
ROUTE	207.30	7	4	.10280	36.56	1.622	.29590	1.633	.556		
ADD HYD	207.40	4& 6	2	.30430	57.06	4.621	.28475	1.700	.293		
ROUTE	206.10	2	4	.30430	55.10	4.621	.28475	1.766	.283		
COMPUTE NM HYD	206.00	-	3	.06250	32.10	1.152	.34554	1.533	.802	PER IMP=	15.00
ADD HYD	206.20	3& 4	2	.36680	71.83	5.773	.29511	1.700	.306		

AHYMO SUMMARY TABLE (AHYMO491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/21/1994

INPUT FILE = UEEX100.DAT USER NO.= MOLI6ONM.I01

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	2.350
COMPUTE NM HYD	201.00	-	3	.04590	58.95	1.713	.69990	1.500	2.007	PER IMP=	5.00
COMPUTE NM HYD	202.00	-	7	.02330	32.44	.960	.77217	1.500	2.175	PER IMP=	8.00
ADD HYD	202.10	3& 7	2	.06920	91.38	2.673	.72423	1.500	2.063		
ROUTE	202.30	2	7	.06920	91.23	2.673	.72423	1.500	2.060		
ROUTE	202.40	7	4	.06920	81.38	2.673	.72424	1.567	1.837		
COMPUTE NM HYD	203.00	-	3	.02780	34.82	1.055	.71137	1.500	1.957	PER IMP=	10.00
ROUTE	203.10	3	7	.02780	34.77	1.055	.71138	1.533	1.954		
ROUTE	203.20	7	3	.02780	30.26	1.055	.71138	1.567	1.701		
ADD HYD	203.30	3& 4	2	.09700	111.64	3.728	.72054	1.567	1.798		
COMPUTE NM HYD	205.00	-	3	.10450	96.39	4.344	.77940	1.600	1.441	PER IMP=	10.00
ADD HYD	205.10	3& 2	5	.20150	205.16	8.071	.75107	1.600	1.591		
ROUTE RESERVOIR	505.00	5	4	.20150	37.64	8.071	.75107	2.100	.292	AC-FT=	19.476
ROUTE	205.20	4	7	.20150	37.64	8.071	.75107	2.100	.292		
ROUTE	205.30	7	6	.20150	37.34	8.071	.75107	2.200	.290		
COMPUTE NM HYD	204.00	-	3	.05440	68.13	2.064	.71137	1.500	1.957	PER IMP=	10.00
ROUTE	204.10	3	7	.05440	68.09	2.064	.71138	1.500	1.956		
ROUTE	204.20	7	4	.05440	58.28	2.064	.71138	1.567	1.674		
COMPUTE NM HYD	207.00	-	3	.04840	59.84	2.128	.82422	1.533	1.932	PER IMP=	15.00
ADD HYD	207.10	3& 4	2	.10280	117.00	4.191	.76450	1.567	1.778		
ROUTE	207.20	2	7	.10280	117.36	4.192	.76450	1.567	1.784		
ROUTE	207.30	7	4	.10280	108.91	4.192	.76450	1.600	1.655		
ADD HYD	207.40	4& 6	2	.30430	132.46	12.263	.75560	1.633	.680		
ROUTE	206.10	2	4	.30430	129.14	12.263	.75561	1.667	.663		
COMPUTE NM HYD	206.00	-	3	.06250	78.46	2.785	.83561	1.533	1.961	PER IMP=	15.00
ADD HYD	206.20	3& 4	2	.36680	189.70	15.048	.76924	1.633	.808		

APPENDIX C

SUMMARY PRINTOUTS
BASIN SW3

AHYMO SUMMARY TABLE (AHYMO491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/21/1994
INPUT FILE = [redacted] USER NO.= MDLZCONM.I01

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	1.370
COMPUTE NM HYD	201.00	-	3	.01895	16.50	.125	.12345	1.433	1.361	PER IMP=	.00
COMPUTE NM HYD	202.00	-	7	.04280	20.16	.282	.12345	1.467	.736	PER IMP=	.00
ADD HYD	202.10	3& 7	2	.06175	33.15	.407	.12345	1.433	.839		
ROUTE	202.30	2	7	.06175	34.22	.407	.12346	1.467	.866		
COMPUTE NM HYD	203.00	-	3	.05090	33.95	.335	.12345	1.433	1.042	PER IMP=	.00
COMPUTE NM HYD	204.00	-	2	.04980	35.99	.328	.12345	1.433	1.129	PER IMP=	.00
ADD HYD	205.10	3& 2	5	.10070	69.94	.663	.12345	1.433	1.085		
ROUTE	205.20	5	7	.10070	66.27	.663	.12346	1.433	1.028		
COMPUTE NM HYD	205.00	-	3	.11750	39.95	.774	.12345	1.500	.531	PER IMP=	.00
ROUTE	205.10	3	5	.11750	40.29	.774	.12346	1.467	.536		

AHYMO SUMMARY TABLE (AHYMOA21) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/21/1994
INPUT FILE = SW3EX100.DAT USER NO.= MOLZCONM.101

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	2.350
COMPUTE NM HYD	201.00	-	3	.01695	48.05	.537	.53121	1.433	3.962	PER IMP=	.00
COMPUTE NM HYD	202.00	-	7	.04280	72.63	1.213	.53121	1.467	2.651	PER IMP=	.00
ADD HYD	202.10	3& 7	2	.06175	119.93	1.749	.53121	1.433	3.035		
ROUTE	202.30	2	7	.06175	117.57	1.750	.53123	1.433	2.975		
COMPUTE NM HYD	203.00	-	3	.05090	115.50	1.442	.53121	1.433	3.546	PER IMP=	.00
COMPUTE NM HYD	204.00	-	2	.04980	117.27	1.411	.53121	1.433	3.680	PER IMP=	.00
ADD HYD	205.10	3& 2	5	.10070	232.77	2.853	.53121	1.433	3.612		
ROUTE	205.20	5	7	.10070	224.95	2.853	.53122	1.433	3.490		
COMPUTE NM HYD	205.00	-	3	.11750	162.66	3.329	.53121	1.467	2.163	PER IMP=	.00
ROUTE	205.10	3	5	.11750	162.61	3.329	.53122	1.467	2.162		

AHYMO SUMMARY TABLE (AHYMO431) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/21/1994

INPUT FILE = SN3F10.DAT USER NO.= MDLZCONM.101

F4

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS[1] PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	1.570
COMPUTE NM HYD	201.00	-	3	.01895	16.50	.125	.12345	1.433	1.361	PER IMP=	.00
COMPUTE NM HYD	202.00	-	7	.04280	20.16	.282	.12345	1.467	.736	PER IMP=	.00
ADD HYD	202.10	3& 7	2	.06175	33.16	.407	.12345	1.433	.839		
ROUTE	202.30	2	7	.06175	34.22	.407	.12346	1.467	.866		
COMPUTE NM HYD	203.00	-	3	.03090	33.95	.335	.12345	1.433	1.042	PER IMP=	.00
COMPUTE NM HYD	204.00	-	2	.04980	35.99	.328	.12345	1.433	1.129	PER IMP=	.00
ADD HYD	205.10	3& 2	5	.10070	69.94	.663	.12345	1.433	1.085		
ROUTE	205.20	5	7	.10070	66.27	.663	.12346	1.433	1.028		
COMPUTE NM HYD	205.00	-	3	.11750	39.95	.774	.12345	1.500	.531	PER IMP=	.00
ROUTE	205.10	3	5	.11750	40.29	.774	.12346	1.467	.536		

AHYMO SUMMARY TABLE (AHYMO491) - ANAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/21/1994

INPUT FILE = [redacted] USER NO.= MOL2CONM.I01

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COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION
RAINFALL TYPE= 1										RAIN6= 2.350
COMPUTE NM HYD	201.00	-	3	.01895	48.05	.537	.53121	1.433	3.962	PER IMP= .00
COMPUTE NM HYD	202.00	-	7	.04290	72.63	1.213	.53121	1.467	2.651	PER IMP= .00
ADD HYD	202.10	3& 7	2	.06175	119.93	1.749	.53121	1.433	3.035	
ROUTE	202.30	2	7	.06175	117.57	1.750	.53123	1.433	2.975	
COMPUTE NM HYD	203.00	-	3	.05090	115.50	1.442	.53121	1.433	3.546	PER IMP= .00
COMPUTE NM HYD	204.00	-	2	.04980	117.27	1.411	.53121	1.433	3.680	PER IMP= .00
ADD HYD	205.10	3& 2	5	.10070	232.77	2.853	.53121	1.433	3.612	
ROUTE	205.20	5	7	.10070	224.95	2.953	.53122	1.433	3.490	
COMPUTE NM HYD	205.00	-	3	.11750	162.66	3.329	.53121	1.467	2.163	PER IMP= .00
ROUTE	205.10	3	5	.11750	162.61	3.329	.53122	1.467	2.162	

AHYMO SUMMARY TABLE (AHYMO491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/21/1994

INPUT FILE = SE3F010.DAT USER NO.= HOLZCONM.101

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	1.570
COMPUTE NM HYD	201.00	-	2	.01570	7.12	.105	.12345	1.467	.700	PER IMP=	.00
ROUTE	201.10	2	7	.01590	7.63	.105	.12349	1.467	.750		
COMPUTE NM HYD	202.00	-	3	.02520	12.30	.166	.12345	1.467	.763	PER IMP=	.00
ROUTE	202.10	3	6	.02520	13.32	.166	.12347	1.467	.826		
COMPUTE NM HYD	203.00	-	2	.06310	18.68	.415	.12345	1.500	.467	PER IMP=	.00
ROUTE	203.10	2	8	.06310	18.46	.415	.12346	1.500	.457		
COMPUTE NM HYD	204.00	-	3	.05210	17.30	.343	.12345	1.500	.519	PER IMP=	.00
ROUTE	204.10	3	9	.05210	17.51	.343	.12346	1.467	.525		
COMPUTE NM HYD	205.00	-	2	.02190	15.83	.144	.12345	1.433	1.129	PER IMP=	.00
ROUTE	205.10	2	10	.02190	14.75	.144	.12348	1.467	1.052		
COMPUTE NM HYD	206.00	-	3	.09330	37.30	.614	.12345	1.467	.625	PER IMP=	.00
ROUTE	206.10	3	11	.09330	39.52	.614	.12346	1.467	.662		
COMPUTE NM HYD	207.00	-	2	.04160	14.14	.274	.12345	1.500	.531	PER IMP=	.00
ROUTE	207.10	2	12	.04160	14.24	.274	.12347	1.500	.535		
COMPUTE NM HYD	208.00	-	3	.02850	10.98	.188	.12345	1.467	.602	PER IMP=	.00
ROUTE	208.10	3	13	.02850	11.24	.188	.12347	1.467	.616		
COMPUTE NM HYD	209.00	-	2	.05380	16.83	.354	.12345	1.500	.489	PER IMP=	.00
ROUTE	209.10	2	14	.05380	16.64	.354	.12346	1.500	.483		
COMPUTE NM HYD	210.00	-	3	.02030	9.56	.134	.12345	1.467	.736	PER IMP=	.00
ROUTE	210.10	3	15	.02030	9.98	.134	.12348	1.467	.768		
COMPUTE NM HYD	211.00	-	2	.01340	3.97	.088	.12345	1.500	.463	PER IMP=	.00
ROUTE	211.10	2	16	.01340	3.98	.088	.12349	1.500	.464		
COMPUTE NM HYD	208.00	-	3	.07740	23.89	.510	.12345	1.500	.482	PER IMP=	.00
ROUTE	212.10	3	17	.07740	23.57	.510	.12346	1.500	.476		
COMPUTE NM HYD	213.00	-	2	.06420	19.74	.423	.12345	1.500	.480	PER IMP=	.00
ROUTE	213.10	2	18	.06420	19.50	.423	.12346	1.500	.475		
COMPUTE NM HYD	214.00	-	3	.04440	14.74	.292	.12345	1.500	.519	PER IMP=	.00
ROUTE	214.10	3	19	.04440	14.71	.292	.12346	1.500	.518		
COMPUTE NM HYD	215.00	-	2	.11050	41.94	.728	.12345	1.467	.593	PER IMP=	.00
ROUTE	215.10	2	4	.11050	44.09	.728	.12346	1.467	.623		
COMPUTE NM HYD	216.00	-	3	.11210	44.42	.738	.12345	1.467	.619	PER IMP=	.00
ROUTE	216.10	3	5	.11210	45.85	.738	.12346	1.467	.639		

AHYMO SUMMARY TABLE (AHYMO491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/21/1994

INPUT FILE = SE3FU10A.DAT USER NO.= MOL\ZCONM.101

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	2.350
COMPUTE NM HYD	201.00	-	2	.01590	26.30	.450	.53121	1.467	2.584	PER IMP=	.00
ROUTE	201.10	2	7	.01590	26.53	.451	.53128	1.467	2.607		
COMPUTE NM HYD	202.00	-	3	.02520	44.31	.714	.53121	1.433	2.747	PER IMP=	.00
ROUTE	202.10	3	6	.02520	44.26	.714	.53126	1.467	2.744		
COMPUTE NM HYD	203.00	-	2	.06310	77.01	1.788	.53121	1.500	1.907	PER IMP=	.00
ROUTE	203.10	2	8	.06310	77.34	1.788	.53123	1.500	1.915		
COMPUTE NM HYD	204.00	-	3	.05210	69.41	1.476	.53121	1.467	2.081	PER IMP=	.00
ROUTE	204.10	3	9	.05210	69.34	1.476	.53123	1.467	2.080		
COMPUTE NM HYD	205.00	-	2	.02190	51.58	.620	.53121	1.433	3.680	PER IMP=	.00
ROUTE	205.10	2	10	.02190	51.24	.621	.53127	1.433	3.656		
COMPUTE NM HYD	206.00	-	3	.09330	145.81	2.643	.53121	1.467	2.442	PER IMP=	.00
ROUTE	206.10	3	11	.09330	146.75	2.643	.53122	1.467	2.458		
COMPUTE NM HYD	207.00	-	2	.04160	57.19	1.179	.53121	1.467	2.148	PER IMP=	.00
ROUTE	207.10	2	12	.04160	56.92	1.179	.53124	1.467	2.138		
COMPUTE NM HYD	208.00	-	3	.02950	43.15	.807	.53121	1.467	2.366	PER IMP=	.00
ROUTE	208.10	3	13	.02950	43.11	.808	.53125	1.467	2.363		
COMPUTE NM HYD	209.00	-	2	.05380	67.68	1.524	.53121	1.500	1.966	PER IMP=	.00
ROUTE	209.10	2	14	.05380	68.12	1.524	.53123	1.500	1.978		
COMPUTE NM HYD	210.00	-	3	.02030	34.45	.575	.53121	1.467	2.652	PER IMP=	.00
ROUTE	210.10	3	15	.02030	34.65	.575	.53127	1.467	2.667		
COMPUTE NM HYD	211.00	-	2	.01340	16.25	.380	.53121	1.500	1.895	PER IMP=	.00
ROUTE	211.10	2	16	.01340	16.19	.380	.53129	1.500	1.888		
COMPUTE NM HYD	208.00	-	3	.07740	96.91	2.193	.53121	1.500	1.956	PER IMP=	.00
ROUTE	212.10	3	17	.07740	97.69	2.193	.53123	1.500	1.972		
COMPUTE NM HYD	213.00	-	2	.06420	79.87	1.819	.53121	1.500	1.944	PER IMP=	.00
ROUTE	213.10	2	18	.06420	80.30	1.819	.53123	1.500	1.954		
COMPUTE NM HYD	214.00	-	3	.04440	59.15	1.258	.53121	1.467	2.082	PER IMP=	.00
ROUTE	214.10	3	19	.04440	58.83	1.258	.53124	1.467	2.070		
COMPUTE NM HYD	215.00	-	2	.11050	168.07	3.131	.53121	1.467	2.377	PER IMP=	.00
ROUTE	215.10	2	4	.11050	168.81	3.131	.53122	1.467	2.387		
COMPUTE NM HYD	216.00	-	3	.11210	175.23	3.176	.53121	1.467	2.442	PER IMP=	.00
ROUTE	216.10	3	5	.11210	176.35	3.176	.53122	1.467	2.458		

APPENDIX C

SUMMARY PRINTOUTS
BASIN MIDDLE

AHYMO SUMMARY TABLE (AHYM0491) - AMAFCA VERSION OF AHYMO - APRIL, 1991 RUN DATE (MON,DAY/YR) =06/17/1994

INPUT FILE = [redacted] USER NO.= MOLZCONM.101

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION
RAINFALL TYPE= 1										RAIN6= 1.570
COMPUTE NM HYD	1410.00	-	3	.16200	55.16	7.798	.90260	2.566	.532	PER IMP= 58.00
ROUTE RESERVOIR	501.01	3	5	.16200	55.00	7.793	.90260	2.600	.530	AC-FT= .011
ROUTE	1410.01	5	2	.16200	55.00	7.798	.90260	2.633	.531	
COMPUTE NM HYD	1412.00	-	3	.00330	6.38	.235	1.33747	1.500	3.122	PER IMP= 100.00
ROUTE RESERVOIR	501.01	3	5	.00330	6.38	.235	1.33738	1.533	3.022	AC-FT= .010
ADD HYD	1412.10	2& 5	3	.16530	55.16	8.034	.91123	2.566	.521	
ROUTE	1412.11	3	7	.16530	55.13	8.034	.91127	2.600	.521	
COMPUTE NM HYD	1420.00	-	3	.47000	134.50	21.315	.85033	2.633	.447	PER IMP= 51.00
ROUTE RESERVOIR	501.00	3	5	.47000	134.30	21.315	.85033	2.633	.446	AC-FT= .011
DIVIDE HYD	1420.01	5	16	.40777	87.00	18.493	.85033	2.166	.333	
	1420.02	AND	3	.06223	47.30	2.822	.85033	2.633	1.188	
DIVIDE HYD	1420.03	3	13	.03111	23.65	1.411	.85033	2.633	1.188	
	1420.04	AND	14	.03111	23.65	1.411	.85033	2.633	1.188	
COMPUTE NM HYD	1411.00	-	3	.01430	28.55	1.020	1.33747	1.500	3.120	PER IMP= 100.00
ROUTE RESERVOIR	501.01	3	5	.01430	28.46	1.020	1.33745	1.500	3.110	AC-FT= .010
ADD HYD	1411.10	14& 5	6	.04541	28.46	2.431	1.00371	1.500	.979	
ROUTE	1411.11	6	4	.04541	27.84	2.431	1.00373	1.500	.958	
ADD HYD	1411.21	7& 4	5	.21071	79.35	10.465	.93119	2.633	.588	
COMPUTE NM HYD	1413.00	-	3	.00500	8.25	.270	1.01260	1.500	2.578	PER IMP= 61.00
ROUTE RESERVOIR	501.01	3	4	.00500	8.04	.270	1.01254	1.533	2.514	AC-FT= .010
ADD HYD	1413.10	4& 5	6	.21571	79.50	10.735	.93308	2.600	.576	
ROUTE	1424.01	6	2	.21571	79.49	10.735	.93308	2.633	.576	
COMPUTE NM HYD	1424.10	-	3	.00550	9.56	.321	1.09590	1.500	2.717	PER IMP= 71.00
ROUTE RESERVOIR	501.01	3	4	.00550	9.30	.321	1.09585	1.533	2.643	AC-FT= .010
ADD HYD	1424.10	2& 4	8	.22121	79.66	11.056	.93712	2.633	.563	
ROUTE	1424.11	16	7	.40777	87.00	18.492	.85030	2.700	.333	
COMPUTE NM HYD	1421.00	-	4	.02230	33.37	1.036	.87099	1.500	2.338	PER IMP= 44.00
ROUTE RESERVOIR	501.01	4	5	.02230	33.23	1.036	.87098	1.500	2.328	AC-FT= .010
ADD HYD	1421.10	13& 5	3	.05341	33.23	2.447	.85895	1.500	.972	
ROUTE	1421.11	3	2	.05341	32.69	2.447	.85896	1.533	.956	
COMPUTE NM HYD	1422.00	-	3	.01580	31.55	1.127	1.33747	1.500	3.120	PER IMP= 100.00
ROUTE RESERVOIR	501.01	3	5	.01580	31.46	1.127	1.33745	1.500	3.111	AC-FT= .010
ADD HYD	1422.10	5& 2	5	.06921	63.24	3.574	.96818	1.500	1.428	
ROUTE	1422.11	5	2	.06921	63.22	3.574	.96820	1.533	1.427	
COMPUTE NM HYD	1423.00	-	3	.00520	8.81	.292	1.05425	1.500	2.648	PER IMP= 66.00
ROUTE RESERVOIR	501.01	3	5	.00520	8.58	.292	1.05420	1.533	2.579	AC-FT= .010
ADD HYD	1423.10	2& 5	5	.07441	71.80	3.866	.97420	1.533	1.508	
ROUTE	1423.11	5	2	.07441	72.04	3.866	.97420	1.533	1.513	
ADD HYD	1423.21	2& 7	5	.48219	111.97	22.359	.86942	2.633	.363	
ROUTE	1423.22	5	2	.48219	111.96	22.358	.86942	2.633	.363	
COMPUTE NM HYD	1424.20	-	3	.00740	12.60	.419	1.06258	1.500	2.661	PER IMP= 67.00
ROUTE RESERVOIR	501.01	3	4	.00740	12.38	.419	1.06254	1.500	2.614	AC-FT= .010
ADD HYD	1424.30	2& 4	5	.48959	112.18	22.778	.87233	2.633	.358	
ROUTE	1424.31	5	2	.48959	112.17	22.778	.87233	2.633	.358	
ADD HYD	1424.41	2& 8	7	.71080	191.83	33.834	.89249	2.633	.422	
COMPUTE NM HYD	1414.00	-	5	.01910	33.87	1.150	1.12922	1.500	2.771	PER IMP= 75.00
ROUTE RESERVOIR	501.01	5	4	.01910	33.77	1.150	1.12920	1.500	2.763	AC-FT= .010
ROUTE	1414.01	4	2	.01910	33.12	1.150	1.12923	1.500	2.709	
COMPUTE NM HYD	1415.00	-	5	.01910	32.16	1.065	1.04592	1.500	2.631	PER IMP= 65.00
ADD HYD	1415.10	5& 2	8	.03820	65.29	2.216	1.08755	1.500	2.670	
ROUTE	1415.11	8	2	.03820	63.59	2.216	1.08757	1.533	2.601	
ADD HYD	1415.21	7& 2	5	.74900	200.08	36.050	.90244	1.533	.417	
DIVIDE HYD	1415.31	5	4	.59774	108.30	28.770	.90244	1.433	.283	
	1415.41	AND	11	.15126	91.78	7.280	.90244	1.533	.948	
ROUTE	1415.42	4	13	.59774	108.08	28.767	.90238	3.366	.283	
ROUTE	1415.43	11	5	.15126	86.53	7.280	.90248	1.533	.894	

ROUTE RESERVOIR	501.01	11	4	.01200	17.61	.648	1.01258	1.500	2.554	AC-FT=	.010
ADD HYD	1425.10	4& 5	5	.16326	105.65	7.928	.91057	1.533	1.011		
ROUTE	1425.11	5	8	.16326	92.28	7.928	.91057	1.567	.883		
COMPUTE NM HYD	1430.00	-	2	.14500	51.45	5.276	.68223	2.133	*1.554	PER IMP=	32.00
ROUTE RESERVOIR	501.01	2	4	.14500	51.45	5.276	.68222	2.133	*1.554	AC-FT=	.010
ROUTE	1430.01	4	7	.14500	49.90	5.276	.68223	2.233	*1.508		
ADD HYD	1430.11	13& 7	6	.74274	157.34	34.043	.85940	2.233	.330		
ADD HYD	1430.21	6& 8	19	.90600	227.36	41.972	.86862	1.566	.392		
COMPUTE NM HYD	1400.00	-	1	.03270	62.66	2.202	1.26250	1.500	2.994	PER IMP=	91.00
ROUTE RESERVOIR	501.01	1	4	.03270	62.61	2.202	1.26249	1.500	2.992	AC-FT=	.010
ROUTE	1400.01	4	7	.03270	55.76	2.202	1.26250	1.533	2.664		
COMPUTE NM HYD	1401.00	-	1	.03120	48.91	1.560	.93763	1.500	2.449	PER IMP=	52.00
ROUTE RESERVOIR	501.01	1	2	.03120	48.91	1.560	.93762	1.500	2.444	AC-FT=	.010
ADD HYD	1401.10	2& 7	4	.06390	102.69	3.762	1.10387	1.533	2.511		
ROUTE	1402.01	4	7	.06390	100.42	3.762	1.10388	1.533	2.456		
COMPUTE NM HYD	1402.10	-	1	.05400	36.98	2.332	.86961	1.800	1.079	PER IMP=	28.00
ADD HYD	1402.10	1& 7	4	.11790	115.52	6.094	.96909	1.567	1.531		
ROUTE RESERVOIR	107.32	4	2	.11790	115.58	6.094	.96909	1.567	1.532	AC-FT=	.010
ROUTE	1402.11	2	7	.11790	110.59	6.094	.96909	1.600	1.466		
COMPUTE NM HYD	1403.00	-	1	.01000	15.60	.504	.94589	1.500	2.437	PER IMP=	55.00
ADD HYD	1403.10	1& 7	4	.12790	122.40	6.598	.96727	1.567	1.495		
ROUTE RESERVOIR	501.01	4	2	.12790	122.36	6.598	.96727	1.567	1.495	AC-FT=	.010
ROUTE	1403.11	2	7	.12790	119.93	6.598	.96727	1.600	1.465		
COMPUTE NM HYD	1404.00	-	2	.00800	7.71	.281	.65887	1.500	1.696	PER IMP=	25.00
ADD HYD	1404.10	2& 7	4	.13590	126.82	6.879	.94911	1.600	1.458		
ROUTE RESERVOIR	501.01	4	5	.13590	126.89	6.879	.94911	1.600	1.459	AC-FT=	.010
ROUTE	1404.11	5	7	.13590	125.50	6.879	.94912	1.600	1.444		
ADD HYD	1404.21	19& 7	1	1.04190	315.52	48.851	.87912	1.600	.473		
ROUTE RESERVOIR	501.01	1	2	1.04190	315.23	48.851	.87912	1.600	.474	AC-FT=	.024
ROUTE	1404.22	2	3	1.04190	308.08	48.850	.87911	1.633	.462		
ROUTE	1404.23	3	2	1.04190	308.80	48.850	.87910	1.633	.463		
COMPUTE NM HYD	1450.00	-	15	.12100	54.80	3.077	.47684	1.733	.708	PER IMP=	12.00
ADD HYD	1450.10	2&15	18	1.16290	358.29	51.927	.83725	1.633	.481		
COMPUTE NM HYD	1440.00	-	4	.15700	167.20	8.182	.97720	1.633	1.664	PER IMP=	60.00
ROUTE RESERVOIR	501.01	4	5	.15700	165.48	8.182	.97719	1.633	1.647	AC-FT=	.012
ROUTE	1440.01	5	4	.15700	153.15	8.182	.97720	1.667	1.524		
COMPUTE NM HYD	1441.00	-	3	.01550	30.95	1.106	1.33746	1.500	3.120	PER IMP=	100.00
ROUTE RESERVOIR	501.01	3	5	.01550	30.86	1.106	1.33745	1.500	3.111	AC-FT=	.010
ADD HYD	1441.10	5& 4	5	.17250	168.54	9.288	1.00956	1.667	1.527		
ROUTE	1441.11	5	2	.17250	167.72	9.288	1.00957	1.667	1.519		
COMPUTE NM HYD	1444.00	-	3	.00550	6.81	.185	.62943	1.500	1.935	PER IMP=	15.00
ROUTE RESERVOIR	501.01	3	5	.00550	6.70	.185	.62937	1.533	1.902	AC-FT=	.010
ADD HYD	1444.10	2& 5	5	.17800	171.36	9.473	.99782	1.667	1.504		
COMPUTE NM HYD	1442.10	-	3	.00700	11.98	.400	1.07091	1.500	2.675	PER IMP=	68.00
ROUTE RESERVOIR	501.01	3	4	.00700	11.75	.400	1.07087	1.500	2.623	AC-FT=	.010
ADD HYD	1442.20	5& 4	9	.18500	177.46	9.872	1.00058	1.667	1.499		
COMPUTE NM HYD	1443.00	-	3	.01680	27.99	.922	1.02926	1.500	2.604	PER IMP=	63.00
ROUTE RESERVOIR	501.01	3	4	.01680	27.86	.922	1.02924	1.500	2.592	AC-FT=	.010
ROUTE	1443.01	4	2	.01680	27.34	.922	1.02927	1.500	2.543		
COMPUTE NM HYD	1442.20	-	3	.00980	15.81	.512	.97928	1.500	2.520	PER IMP=	57.00
ROUTE RESERVOIR	501.01	3	4	.00980	15.61	.512	.97925	1.500	2.488	AC-FT=	.010
ADD HYD	1442.30	2& 4	5	.02660	42.94	1.434	1.01083	1.500	2.523		
ROUTE	1442.31	5	2	.02660	40.74	1.434	1.01084	1.533	2.393		
ADD HYD	1442.41	2& 9	5	.21160	206.35	11.306	1.00187	1.633	1.524		
ROUTE	1442.42	5	2	.21160	206.07	11.306	1.00187	1.633	1.522		
COMPUTE NM HYD	1447.00	-	3	.01400	24.96	.849	1.13755	1.500	2.785	PER IMP=	76.00
ROUTE RESERVOIR	501.01	3	4	.01400	24.83	.849	1.13753	1.500	2.771	AC-FT=	.010
ADD HYD	1447.10	2& 4	7	.22560	220.53	12.156	1.01029	1.633	1.527		
COMPUTE NM HYD	1445.00	-	3	.01720	27.12	.868	.94596	1.500	2.464	PER IMP=	53.00
ROUTE RESERVOIR	501.01	3	4	.01720	26.98	.868	.94594	1.500	2.451	AC-FT=	.010
ROUTE	1445.01	4	2	.01720	24.28	.868	.94597	1.533	2.205		
COMPUTE NM HYD	1446.00	-	3	.03230	52.37	1.701	.98761	1.500	2.533	PER IMP=	58.00
ROUTE RESERVOIR	501.01	3	4	.03230	52.28	1.701	.98760	1.500	2.529	AC-FT=	.010
ADD HYD	1446.10	2& 4	5	.04950	74.40	2.569	.97313	1.533	2.348		
ROUTE	1446.11	5	2	.04950	73.75	2.569	.97314	1.533	2.328		

ROUTE	1448.22	5	2	.27310	280.02	14.725	1.00360	1.600	1.590
COMPUTE NM HYD	1448.00	-	3	.03080	54.06	1.328	1.11256	1.500	2.743 PER IMP= 73.00
ROUTE RESERVOIR	501.01	3	4	.03080	53.99	1.328	1.11255	1.500	2.739 AC-FT= .010
ADD HYD	1448.10	2& 4	5	.30590	318.45	16.552	1.01457	1.567	1.527
ROUTE	1448.11	5	4	.30590	312.77	16.552	1.01457	1.600	1.599
ROUTE	1448.12	4	20	.30590	306.03	16.552	1.01457	1.633	1.563
ADD HYD	1448.22	20&18	1	1.46880	664.32	68.480	.87418	1.633	1.707
ROUTE RESERVOIR	501.01	1	3	1.46880	97.57	68.485	.87425	3.666	.104 AC-FT= 42.866
COMPUTE NM HYD	1451.00	-	4	.07720	32.20	1.637	.39767	1.700	.652 PER IMP= 3.00
ADD HYD	1451.01	3& 4	1	1.54600	98.32	68.031	.82508	3.300	.099

AHYMO SUMMARY TABLE (AHYM0491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06.17/1994

INPUT FILE = [redacted] USER NO.= MOLZCONM.101

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	2.350
COMPUTE NM HYD	101.00	-	3	.16200	96.50	13.579	1.57169	2.366	.931	PER IMP=	56.00
ROUTE RESERVOIR	501.01	3	5	.16200	55.02	13.579	1.57169	3.233	.531	AC-FT=	2.474
ROUTE	101.01	5	2	.16200	55.23	13.579	1.57169	2.100	.533		
COMPUTE NM HYD	101.30	-	3	.00330	10.04	.372	2.11536	1.500	4.754	PER IMP=	100.00
ROUTE RESERVOIR	501.01	3	5	.00330	6.38	.372	2.11528	1.600	3.116	AC-FT=	.041
ADD HYD	101.31	2& 5	3	.16530	57.16	13.952	1.58254	2.100	.540		
ROUTE	101.32	3	7	.16530	56.77	13.952	1.58253	2.100	.537		
COMPUTE NM HYD	102.00	-	3	.47000	255.49	37.915	1.51257	2.600	.849	PER IMP=	51.00
ROUTE RESERVOIR	105.01	3	5	.47000	134.48	37.915	1.51257	3.400	.447	AC-FT=	8.168
DIVIDE HYD	134.01	5	15	.33479	87.00	27.008	1.51257	1.966	.406		
	134.02	AND	3	.13521	47.48	10.907	1.51257	3.400	.549		
DIVIDE HYD	134.11	3	13	.06760	23.74	5.454	1.51257	3.400	.549		
	134.12	AND	14	.06760	23.74	5.454	1.51257	3.400	.549		
COMPUTE NM HYD	101.10	-	3	.01430	43.48	1.613	2.11537	1.500	4.751	PER IMP=	100.00
ROUTE RESERVOIR	501.01	3	5	.01430	28.53	1.613	2.11535	1.600	3.116	AC-FT=	.148
ADD HYD	101.11	14& 5	6	.08190	33.05	7.067	1.61781	2.066	.630		
ROUTE	101.12	6	4	.08190	32.26	7.067	1.61782	2.100	.615		
ADD HYD	101.40	7& 4	5	.24720	89.03	21.019	1.59422	2.100	.563		
COMPUTE NM HYD	104.10	-	3	.00300	13.29	.461	1.73026	1.500	4.152	PER IMP=	61.00
ROUTE RESERVOIR	501.01	3	4	.00300	8.23	.461	1.73020	1.633	2.572	AC-FT=	.059
ADD HYD	104.11	4& 5	6	.25220	91.15	21.480	1.59692	2.100	.565		
ROUTE RESERVOIR	501.01	6	4	.25220	79.33	21.480	1.59692	3.166	.491	AC-FT=	.221
ROUTE	104.12	4	2	.25220	79.83	21.480	1.59691	2.066	.495		
COMPUTE NM HYD	104.20	-	3	.00550	15.16	.537	1.82900	1.500	4.306	PER IMP=	71.00
ROUTE RESERVOIR	501.01	3	4	.00550	9.54	.536	1.82895	1.600	2.710	AC-FT=	.054
ADD HYD	104.21	2& 4	8	.25770	82.76	22.016	1.60186	2.066	.502		
ROUTE	102.01	16	7	.33479	87.00	27.007	1.51254	2.366	.406		
COMPUTE NM HYD	102.10	-	4	.02230	55.43	1.658	1.56239	1.500	3.884	PER IMP=	44.00
ROUTE RESERVOIR	501.01	4	5	.02230	33.35	1.658	1.56237	1.633	2.337	AC-FT=	.241
ADD HYD	102.11	17& 5	3	.08990	33.41	7.312	1.52493	2.066	.581		
ROUTE	102.12	3	2	.08990	33.41	7.312	1.52493	1.467	.581		
COMPUTE NM HYD	103.70	-	3	.01580	48.04	1.783	2.11536	1.500	4.751	PER IMP=	100.00
ROUTE RESERVOIR	501.01	3	5	.01580	31.53	1.783	2.11535	1.600	3.118	AC-FT=	.162
ADD HYD	103.71	5& 2	5	.10570	64.94	9.094	1.61318	1.467	.960		
ROUTE	103.72	5	2	.10570	65.03	9.094	1.61319	1.467	.961		
COMPUTE NM HYD	103.60	-	3	.00520	14.07	.494	1.77963	1.500	4.229	PER IMP=	66.00
ROUTE RESERVOIR	501.01	3	5	.00520	8.79	.494	1.77957	1.633	2.641	AC-FT=	.061
ADD HYD	103.61	2& 5	5	.11090	73.82	9.588	1.62098	1.467	1.040		
ROUTE	103.62	5	2	.11090	73.83	9.588	1.62099	1.467	1.040		
ADD HYD	103.63	2& 7	5	.44570	129.86	36.595	1.53952	2.100	.455		
ROUTE RESERVOIR	501.01	5	7	.44570	111.96	36.595	1.53952	2.900	.392	AC-FT=	.403
ROUTE	103.64	7	2	.44570	112.94	36.595	1.53952	2.000	.396		
COMPUTE NM HYD	103.50	-	3	.00740	20.09	.706	1.78950	1.500	4.243	PER IMP=	67.00
ROUTE RESERVOIR	501.01	3	4	.00740	12.58	.706	1.78947	1.633	2.656	AC-FT=	.083
ADD HYD	103.51	2& 4	5	.45310	119.78	37.301	1.54360	1.800	.413		
ROUTE RESERVOIR	501.01	5	4	.45310	112.15	37.301	1.54360	2.900	.387	AC-FT=	.134
ROUTE	103.52	4	2	.45310	112.56	37.301	1.54360	1.800	.388		
ADD HYD	104.02	2& 8	7	.71080	194.91	59.318	1.56472	2.066	.428		
COMPUTE NM HYD	107.30	-	5	.01910	53.35	1.903	1.86850	1.500	4.364	PER IMP=	75.00
ROUTE RESERVOIR	501.01	5	4	.01910	33.85	1.903	1.86849	1.600	2.769	AC-FT=	.198
ROUTE	107.31	4	2	.01910	34.13	1.903	1.86851	1.467	2.792		
COMPUTE NM HYD	107.20	-	5	.01910	51.46	1.803	1.76976	1.500	4.210	PER IMP=	65.00
ADD HYD	107.21	5& 2	8	.03820	85.22	3.706	1.81911	1.500	3.486		
ROUTE RESERVOIR	501.01	8	5	.03820	65.27	3.706	1.81911	1.633	2.670	AC-FT=	.211
ROUTE	107.22	5	2	.03820	65.27	3.706	1.81913	1.733	2.670		
ADD HYD	104.03	7& 2	5	.74900	246.08	63.024	1.57770	1.800	.513		

DIVIDE HYD	134.01	/	4	.46512	108.30	39.137	1.57770	1.400	.364			
	134.02	AND	11	.28388	91.77	23.387	1.57770	2.266	.505			
ROUTE	134.10	4	13	.46512	108.30	39.136	1.57764	4.866	.364			
ROUTE	134.20	11	5	.28388	91.77	23.887	1.57771	2.266	.505			
COMPUTE NM HYD	134.00	-	11	.01200	31.86	1.107	1.73025	1.500	4.148	PER IMP=	61.00	
ROUTE RESERVOIR	501.01	11	4	.01200	19.76	1.107	1.73023	1.633	2.573	AC-FT=	.131	
ADD HYD	134.30	4& 5	5	.29588	111.53	24.994	1.58390	1.833	.589			
ROUTE	134.40	5	8	.29588	111.50	24.994	1.58390	1.833	.589			
COMPUTE NM HYD	105.00	-	2	.14500	69.47	10.133	1.31034	2.133	1.072	PER IMP=	32.00	
ROUTE RESERVOIR	501.01	2	4	.14500	54.00	10.133	1.31034	2.666	.582	AC-FT=	2.082	
ROUTE	105.01	4	7	.14500	54.00	10.133	1.31034	2.666	.582			
ADD HYD	105.02	13& 7	6	.61012	162.18	49.269	1.51411	3.366	.415			
ADD HYD	105.03	6& 3	19	.90600	264.22	74.253	1.53690	1.833	.456			
COMPUTE NM HYD	107.40	-	1	.03270	96.52	3.534	2.02649	1.500	4.612	PER IMP=	91.00	
ROUTE RESERVOIR	501.01	1	4	.03270	62.64	3.534	2.02649	1.600	2.993	AC-FT=	.327	
ROUTE	107.41	4	7	.03270	62.58	3.534	2.02650	1.800	2.990			
COMPUTE NM HYD	107.10	-	1	.03120	80.02	2.731	1.64138	1.500	4.007	PER IMP=	52.00	
ROUTE RESERVOIR	501.01	1	2	.03120	48.89	2.731	1.64137	1.633	2.445	AC-FT=	.326	
ADD HYD	107.11	2& 7	4	.06390	111.47	6.265	1.83845	1.800	2.726			
ROUTE	107.12	4	7	.06390	111.42	6.265	1.83846	1.800	2.724			
COMPUTE NM HYD	107.80	-	1	.05400	64.55	4.251	1.48637	1.800	1.868	PER IMP=	38.00	
ADD HYD	107.81	1& 7	4	.11790	175.96	10.546	1.67719	1.800	2.332			
ROUTE RESERVOIR	501.01	4	2	.11790	121.48	10.546	1.67719	1.900	1.610	AC-FT=	1.182	
ROUTE	107.82	2	7	.11790	121.47	10.546	1.67719	2.233	1.610			
COMPUTE NM HYD	107.70	-	1	.01000	25.58	.878	1.64628	1.500	3.997	PER IMP=	55.00	
ADD HYD	107.71	1& 7	4	.12790	141.41	11.424	1.67477	1.533	1.728			
ROUTE RESERVOIR	501.01	4	2	.12790	125.13	11.424	1.67477	2.066	1.529	AC-FT=	.314	
ROUTE	107.72	2	7	.12790	125.13	11.424	1.67477	2.233	1.529			
COMPUTE NM HYD	107.60	-	2	.00800	17.41	.552	1.29456	1.500	3.400	PER IMP=	25.00	
ADD HYD	107.61	2& 7	4	.13590	139.68	11.976	1.65239	1.533	1.606			
ROUTE RESERVOIR	510.01	4	5	.13590	126.90	11.976	1.65238	2.100	1.459	AC-FT=	.258	
ROUTE	107.62	5	7	.13590	126.90	11.977	1.65239	2.033	1.459			
ADD HYD	100.00	19& 7	1	1.04190	391.05	86.240	1.55197	1.833	.586			
ROUTE RESERVOIR	501.01	1	2	1.04190	390.54	86.240	1.55197	1.833	.586	AC-FT=	.031	
ROUTE	100.01	2	3	1.04190	389.08	86.239	1.55195	1.866	.583			
ROUTE	100.02	3	2	1.04190	388.92	86.239	1.55195	1.866	.583			
COMPUTE NM HYD	106.00	-	15	.12100	118.87	6.805	1.05448	1.733	1.535	PER IMP=	12.00	
ADD HYD	106.01	2&15	18	1.16290	493.50	93.044	1.50019	1.800	.663			
COMPUTE NM HYD	103.00	-	4	.15700	283.75	14.069	1.68017	1.633	2.824	PER IMP=	60.00	
ROUTE RESERVOIR	501.01	4	5	.15700	165.49	14.069	1.68017	1.866	1.647	AC-FT=	2.169	
ROUTE	103.01	5	4	.15700	165.00	14.069	1.68017	2.266	1.642			
COMPUTE NM HYD	103.10	-	3	.01550	47.13	1.749	2.11536	1.500	4.751	PER IMP=	100.00	
ROUTE RESERVOIR	501.01	3	5	.01550	30.93	1.749	2.11535	1.600	3.118	AC-FT=	.159	
ADD HYD	103.11	5& 4	5	.17250	193.81	15.817	1.71927	1.800	1.756			
ROUTE RESERVOIR	501.01	5	3	.17250	168.39	15.817	1.71927	2.233	1.525	AC-FT=	.663	
ROUTE	103.12	3	2	.17250	168.40	15.817	1.71927	2.233	1.525			
COMPUTE NM HYD	103.20	-	3	.00550	12.10	.374	1.27602	1.500	3.438	PER IMP=	15.00	
ROUTE RESERVOIR	501.01	3	5	.00550	6.79	.374	1.27597	1.633	1.929	AC-FT=	.069	
ADD HYD	103.21	2& 5	5	.17800	175.18	16.192	1.70558	1.700	1.538			
COMPUTE NM HYD	103.30	-	3	.00700	19.08	.672	1.79938	1.500	4.258	PER IMP=	68.00	
ROUTE RESERVOIR	501.01	3	4	.00700	11.96	.672	1.79934	1.633	2.670	AC-FT=	.079	
ADD HYD	103.31	5& 4	9	.18500	187.15	16.863	1.70912	1.700	1.581			
COMPUTE NM HYD	103.90	-	3	.01680	44.93	1.568	1.75000	1.500	4.179	PER IMP=	63.00	
ROUTE RESERVOIR	501.01	3	4	.01680	27.97	1.568	1.74999	1.633	2.602	AC-FT=	.178	
ROUTE	103.91	4	2	.01680	28.03	1.568	1.75002	1.467	2.606			
COMPUTE NM HYD	103.80	-	3	.00980	25.63	.884	1.69076	1.500	4.087	PER IMP=	57.00	
ROUTE RESERVOIR	501.01	3	4	.00980	15.79	.884	1.69073	1.633	2.518	AC-FT=	.109	
ADD HYD	103.81	2& 4	5	.02660	43.82	2.452	1.72816	1.467	2.574			
ROUTE	103.82	5	2	.02660	43.75	2.452	1.72817	1.833	2.570			
ADD HYD	103.83	2& 9	5	.21160	230.89	19.315	1.71152	1.833	1.705			
ROUTE RESERVOIR	501.01	5	9	.21160	206.34	19.315	1.71152	1.900	1.524	AC-FT=	.700	
ROUTE	103.84	9	2	.21160	207.83	19.315	1.71152	1.533	1.535			
COMPUTE NM HYD	103.50	-	3	.01400	39.25	1.403	1.87837	1.500	4.381	PER IMP=	76.00	
ROUTE RESERVOIR	501.01	3	4	.01400	24.94	1.403	1.87835	1.600	2.784	AC-FT=	.147	
ADD HYD	103.51	2& 4	7	.22560	232.78	20.718	1.72187	1.533	1.612			

ROUTE RESERVOIR	501.01	3	4	.01720	27.10	1.515	1.65124	1.633	2.462	AC-FT=	.185
ROUTE	103.71	4	2	.01720	27.10	1.515	1.65127	1.333	2.462		
COMPUTE NM HYD	103.60	-	3	.03230	84.76	2.930	1.70063	1.500	4.100	PER IMP=	58.00
ROUTE RESERVOIR	501.01	3	4	.03230	52.35	2.930	1.70062	1.633	2.533	AC-FT=	.336
ADD HYD	103.61	2& 4	5	.04950	79.45	4.444	1.68347	1.833	2.508		
ROUTE	103.62	5	2	.04950	79.45	4.444	1.68348	1.833	2.508		
ADD HYD	103.63	2& 7	5	.27510	310.96	25.162	1.71496	1.600	1.766		
ROUTE RESERVOIR	501.01	5	7	.27510	281.39	25.162	1.71496	1.866	1.598	AC-FT=	.840
ROUTE	103.64	7	2	.27510	281.39	25.162	1.71496	1.866	1.598		
COMPUTE NM HYD	103.40	-	3	.03080	85.41	3.037	1.84875	1.500	4.333	PER IMP=	73.00
ROUTE RESERVOIR	501.01	3	4	.03080	54.04	3.037	1.84874	1.600	2.742	AC-FT=	.313
ADD HYD	103.41	2& 4	5	.30590	335.43	28.199	1.72843	1.800	1.713		
ROUTE RESERVOIR	501.01	5	2	.30590	318.44	28.199	1.72843	1.833	1.627	AC-FT=	.458
ROUTE	100.50	2	4	.30590	318.44	28.199	1.72843	1.866	1.627		
ROUTE	100.60	4	20	.30590	318.43	28.197	1.72843	1.966	1.627		
ADD HYD	100.61	20&16	1	1.46880	811.89	121.243	1.54773	1.800	.864		
ROUTE RESERVOIR	100.62	1	3	1.46880	119.09	121.238	1.54766	4.799	.127	AC-FT=	82.124

APPENDIX C

SUMMARY PRINTOUTS
BASIN E END

AHYMO SUMMARY TABLE (AHYMO491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/17/1994

INPUT FILE = eeex10.dat USER NO.= MDL2CONM.I01

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	1.570
COMPUTE NM HYD	1503.00	-	3	.11000	95.38	5.892	1.00427	1.766	1.355	PER IMP=	60.00
ROUTE RESERVOIR	501.00	3	6	.11000	28.57	5.892	1.00426	2.333	.406	AC-FT=	2.708
ROUTE	1503.01	6	2	.11000	28.50	5.892	1.00427	3.466	.405		
COMPUTE NM HYD	1505.00	-	3	.01150	19.99	.672	1.09590	1.500	2.716	PER IMP=	71.00
ROUTE RESERVOIR	501.00	3	6	.01150	19.83	.672	1.09587	1.500	2.694	AC-FT=	.010
ADD HYD	1505.10	2& 6	8	.12150	35.53	6.564	1.01293	1.667	.457		
COMPUTE NM HYD	1506.00	-	3	.00600	10.27	.343	1.07091	1.500	2.675	PER IMP=	68.00
ROUTE RESERVOIR	501.00	3	6	.00600	10.02	.343	1.07086	1.500	2.608	AC-FT=	.010
ADD HYD	1506.10	6& 8	5	.12750	41.64	6.906	1.01566	1.633	.510		
ROUTE RESERVOIR	501.00	5	8	.12750	41.52	6.906	1.01566	1.633	.509	AC-FT=	.010
ROUTE	1506.11	8	6	.12750	39.75	6.906	1.01566	1.667	.487		
COMPUTE NM HYD	1507.00	-	4	.00900	13.34	.417	.86793	1.500	2.316	PER IMP=	45.00
ADD HYD	1507.10	4& 6	5	.13650	46.50	7.323	1.00591	1.633	.532		
ROUTE RESERVOIR	501.00	5	6	.13650	46.42	7.323	1.00591	1.633	.531	AC-FT=	.008
ROUTE	1507.11	6	11	.13650	46.07	7.323	1.00592	1.667	.527		
COMPUTE NM HYD	1508.00	-	4	.00800	11.86	.370	.86793	1.500	2.316	PER IMP=	45.00
ROUTE RESERVOIR	501.00	4	6	.00800	11.60	.370	.86789	1.533	2.265	AC-FT=	.010
ADD HYD	1508.10	6&11	7	.14450	53.51	7.693	.99827	1.567	.579		
COMPUTE NM HYD	1509.00	-	3	.02600	28.88	1.092	.78734	1.567	1.735	PER IMP=	36.00
ROUTE RESERVOIR	501.00	3	6	.02600	28.82	1.092	.78732	1.567	1.732	AC-FT=	.010
ROUTE	1509.01	6	8	.02600	28.53	1.092	.78734	1.600	1.714		
ADD HYD	1509.11	7& 8	4	.17050	81.68	8.785	.96610	1.600	.749		
ROUTE	1509.12	4	17	.17050	81.67	8.785	.96611	1.600	.748		
COMPUTE NM HYD	1500.00	-	3	.12000	79.24	4.618	.72153	1.700	1.019	PER IMP=	42.00
ROUTE	1500.01	3	4	.12000	48.76	4.618	.72153	1.933	.635		
COMPUTE NM HYD	1501.00	-	3	.13000	18.44	3.200	.40003	2.600	.192	PER IMP=	10.00
ADD HYD	1501.10	3& 4	2	.27000	58.60	7.818	.54291	2.033	.339		
COMPUTE NM HYD	1502.00	-	3	.08000	98.93	4.938	1.15733	1.633	1.932	PER IMP=	80.00
ROUTE RESERVOIR	501.00	3	4	.08000	22.41	4.938	1.15732	2.266	.438	AC-FT=	3.035
ADD HYD	1502.10	4& 2	6	.35000	80.09	12.756	.68334	2.066	.358		
ROUTE	1502.11	6	3	.35000	79.84	12.755	.68333	2.100	.356		
COMPUTE NM HYD	1510.00	-	3	.01940	24.98	.855	.82674	1.533	2.012	PER IMP=	45.00
ADD HYD	1510.10	3& 8	5	.36940	84.21	13.611	.69086	2.066	.356		
ROUTE RESERVOIR	501.00	5	6	.36940	84.21	13.611	.69086	2.066	.356	AC-FT=	.010
ROUTE	1510.11	6	7	.36940	84.20	13.611	.69086	2.066	.356		
COMPUTE NM HYD	1511.00	-	3	.00860	14.10	.461	1.00427	1.500	2.562	PER IMP=	60.00
ROUTE RESERVOIR	501.00	3	6	.00860	13.89	.461	1.00424	1.500	2.523	AC-FT=	.010
ADD HYD	1511.10	6& 7	5	.37800	86.69	14.071	.69799	2.033	.358		
ROUTE	1511.11	5	7	.37800	86.60	14.071	.69798	2.066	.358		
COMPUTE NM HYD	1512.00	-	3	.01340	19.46	.593	.82934	1.500	2.269	PER IMP=	39.00
ROUTE RESERVOIR	501.00	3	6	.01340	19.25	.593	.82932	1.500	2.245	AC-FT=	.010
ADD HYD	1512.10	6& 7	5	.39140	89.43	14.664	.70248	2.033	.357		
ROUTE	1512.11	5	7	.39140	89.08	14.664	.70246	2.066	.356		
COMPUTE NM HYD	1504.00	-	3	.13500	81.33	5.650	.78474	1.833	.941	PER IMP=	40.00
ROUTE RESERVOIR	501.00	3	8	.13500	81.30	5.650	.78474	1.833	.941	AC-FT=	.010
COMPUTE NM HYD	1513.00	-	3	.00150	2.79	.096	1.20419	1.500	2.906	PER IMP=	84.00
ROUTE RESERVOIR	501.00	3	6	.00150	2.54	.096	1.20401	1.533	2.646	AC-FT=	.009
ROUTE	1513.01	6	5	.00150	2.46	.096	1.20436	1.567	2.561		
ADD HYD	1513.11	8& 5	2	.13650	82.36	5.746	.78935	1.833	.943		
COMPUTE NM HYD	1514.00	-	3	.01100	17.35	.555	.94596	1.500	2.464	PER IMP=	53.00
ROUTE RESERVOIR	501.00	3	6	.01100	17.15	.555	.94593	1.500	2.437	AC-FT=	.010
ADD HYD	1514.10	2& 6	5	.14750	87.20	6.301	.80103	1.800	.924		
ROUTE	1514.11	5	4	.14750	87.17	6.301	.80103	1.833	.923		
ADD HYD	1514.21	4& 7	5	.53890	165.60	20.965	.72944	1.900	.480		
COMPUTE NM HYD	1515.00	-	3	.04460	65.53	2.012	.84600	1.500	2.296	PER IMP=	41.00

ADD HYD	1515.10	5& 3	4	.58350	180.45	22.977	.73835	1.866	.483	
ROUTE	1515.11	4	7	.58350	179.32	22.977	.73835	1.900	.480	
COMPUTE NM HYD	1516.00	-	3	.03000	32.57	1.340	.83740	1.600	1.676	PER IMP= 42.00
ROUTE RESERVOIR	501.00	3	6	.03000	32.54	1.340	.83739	1.600	1.675	AC-FT= .010
ROUTE	1516.01	6	4	.03000	32.35	1.340	.83740	1.600	1.665	
ADD HYD	1516.11	4& 7	5	.61350	193.96	24.317	.74319	1.866	.494	
ROUTE	1516.12	5	14	.61350	193.58	24.317	.74319	1.866	.493	

AHYMO SUMMARY TABLE (AHYMO491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/17/1994

INPUT FILE = eeex100.dat USER NO.= MOL2CONM.101

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	2.350
COMPUTE NM HYD	1503.00	-	3	.11000	159.28	10.093	1.72038	1.766	2.263	PER IMP=	60.00
ROUTE RESERVOIR	501.00	3	6	.11000	36.50	10.093	1.72037	2.433	.518	AC-FT=	1430.436
ROUTE	1503.01	6	2	.11000	36.37	10.093	1.72038	2.500	.517		
COMPUTE NM HYD	1505.00	-	3	.01150	31.67	1.122	1.82900	1.500	4.303	PER IMP=	71.00
ROUTE RESERVOIR	301.00	3	6	.01150	19.91	1.122	1.82898	1.600	2.705	AC-FT=	.124
ADD HYD	1505.10	2& 6	8	.12150	47.56	11.215	1.73065	1.833	.612		
COMPUTE NM HYD	1506.00	-	3	.00600	16.36	.576	1.79938	1.500	4.259	PER IMP=	68.00
ROUTE RESERVOIR	501.00	3	6	.00600	10.24	.576	1.79933	1.633	2.667	AC-FT=	.069
ADD HYD	1506.10	6& 8	5	.12750	57.80	11.790	1.73388	1.833	.708		
ROUTE RESERVOIR	501.00	5	8	.12750	42.88	11.790	1.73388	1.866	.526	AC-FT=	.356
ROUTE	1506.11	8	6	.12750	42.69	11.790	1.73389	2.000	.523		
COMPUTE NM HYD	1507.00	-	4	.00900	22.24	.747	1.55555	1.500	3.862	PER IMP=	45.00
ADD HYD	1507.10	4& 6	5	.13650	59.12	12.537	1.72212	1.533	.677		
ROUTE RESERVOIR	501.00	5	6	.13650	58.98	12.537	1.72212	1.533	.675	AC-FT=	.010
ROUTE	1507.11	6	11	.13650	57.32	12.537	1.72212	1.567	.656		
COMPUTE NM HYD	1508.00	-	4	.00800	19.78	.664	1.55555	1.500	3.862	PER IMP=	45.00
ROUTE RESERVOIR	501.00	4	6	.00800	11.77	.664	1.55552	1.633	2.299	AC-FT=	.094
ADD HYD	1508.10	6&11	7	.14450	69.09	13.201	1.71290	1.567	.747		
COMPUTE NM HYD	1509.00	-	3	.02600	49.73	2.022	1.45848	1.567	2.988	PER IMP=	36.00
ROUTE RESERVOIR	501.00	3	6	.02600	28.82	2.022	1.45847	1.766	1.732	AC-FT=	.318
ROUTE	1509.01	6	9	.02600	28.90	2.022	1.45849	1.533	1.737		
ADD HYD	1509.11	7& 8	4	.17050	97.93	15.223	1.67410	1.567	.897		
ROUTE	1509.12	4	17	.17050	97.46	15.223	1.67410	1.600	.893		
COMPUTE NM HYD	1500.00	-	3	.12000	145.00	8.575	1.33981	1.700	1.388	PER IMP=	42.00
ROUTE	1500.01	3	4	.12000	104.87	8.575	1.33981	1.900	1.366		
COMPUTE NM HYD	1501.00	-	3	.15000	44.30	7.532	.94149	2.566	.461	PER IMP=	10.00
ADD HYD	1501.10	3& 4	2	.27000	125.44	15.107	1.11852	1.966	.726		
COMPUTE NM HYD	1502.00	-	3	.08000	157.41	8.097	1.89777	1.633	3.075	PER IMP=	80.00
ROUTE RESERVOIR	501.00	3	4	.08000	37.68	8.097	1.89776	2.233	.736	AC-FT=	4.937
ADD HYD	1502.10	4& 2	6	.35000	159.60	24.204	1.29662	2.000	.712		
ROUTE	1502.11	6	8	.35000	158.81	24.203	1.29660	2.033	.709		
COMPUTE NM HYD	1510.00	-	3	.01940	42.56	1.545	1.49365	1.533	3.428	PER IMP=	45.00
ADD HYD	1510.10	3& 8	5	.36940	167.41	25.749	1.30695	2.033	.708		
ROUTE RESERVOIR	501.00	5	6	.36940	113.90	25.749	1.30695	2.666	.482	AC-FT=	4517.904
ROUTE	1510.11	6	7	.36940	113.90	25.749	1.30694	2.666	.482		
COMPUTE NM HYD	1511.00	-	3	.00860	22.75	.789	1.72038	1.500	4.134	PER IMP=	60.00
ROUTE RESERVOIR	501.00	3	6	.00860	14.00	.789	1.72035	1.633	2.544	AC-FT=	.098
ADD HYD	1511.10	6& 7	5	.37800	114.31	26.538	1.31635	2.666	.473		
ROUTE	1511.11	5	7	.37800	114.27	26.537	1.31634	2.700	.472		
COMPUTE NM HYD	1512.00	-	3	.01340	32.65	1.081	1.51301	1.500	3.807	PER IMP=	39.00
ROUTE RESERVOIR	501.00	3	6	.01340	19.38	1.081	1.51299	1.633	2.260	AC-FT=	.151
ADD HYD	1512.10	6& 7	5	.39140	118.95	27.619	1.32307	1.833	.475		
ROUTE	1512.11	5	7	.39140	118.25	27.618	1.32305	1.833	.472		
COMPUTE NM HYD	1504.00	-	3	.13500	148.42	10.396	1.44384	1.833	1.718	PER IMP=	40.00
ROUTE RESERVOIR	501.00	3	8	.13500	81.32	10.396	1.44383	2.233	.941	AC-FT=	2.073
COMPUTE NM HYD	1513.00	-	3	.00150	4.34	.157	1.95737	1.500	4.520	PER IMP=	84.00
ROUTE RESERVOIR	501.00	3	6	.00150	2.73	.157	1.95719	1.600	2.844	AC-FT=	.024
ROUTE	1513.01	6	5	.00150	2.73	.157	1.95756	1.766	2.844		
ADD HYD	1513.11	8& 5	2	.13650	84.04	10.552	1.44947	1.800	.962		
COMPUTE NM HYD	1514.00	-	3	.01100	28.33	.969	1.65126	1.500	4.025	PER IMP=	53.00
ROUTE RESERVOIR	501.00	3	6	.01100	17.32	.969	1.65124	1.633	2.460	AC-FT=	.122
ADD HYD	1514.10	2& 6	5	.14750	101.36	11.521	1.46452	1.800	1.074		
ROUTE	1514.11	5	4	.14750	101.36	11.521	1.46452	1.800	1.074		
ADD HYD	1514.21	4& 7	5	.53890	219.55	39.139	1.36177	1.833	.637		
COMPUTE NM HYD	1515.00	-	3	.04460	109.52	3.646	1.53276	1.500	3.837	PER IMP=	41.00

ADD HYD	1515.10	5& 5	4	.58350	285.03	42.785	1.37484	1.833	.763		
ROUTE	1515.11	4	7	.58350	284.77	42.785	1.37483	1.833	.763		
COMPUTE NM HYD	1516.00	-	3	.03000	33.36	2.426	1.31778	1.800	2.583	PER IMP=	42.00
ROUTE RESERVOIR	901.00	3	5	.03000	32.54	2.428	1.31777	1.800	1.695	AC-FT=	.373
ROUTE	1516.01	6	4	.03000	32.57	2.428	1.31779	1.800	1.697		
ADD HYD	1515.11	4& 7	5	.61350	317.31	45.213	1.38182	1.833	.808		
ROUTE	1516.12	5	14	.61350	317.04	45.213	1.38181	1.833	.807		

APPENDIX C

SUMMARY PRINTOUTS
BASIN AIR GUARD

AHYMO SUMMARY TABLE (AHYM0491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/17/1994
INPUT FILE = **agex10.dat** USER NO.=,MOLZCONM,101

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	.1 CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	1.570
COMPUTE NM HYD	1600.00	-	3	.18900	136.70	11.121	1.10323	1.933	1.130	PER IMP=	75.00
ROUTE RESERVOIR	501.00	3	6	.18900	121.13	11.120	1.10322	2.100	1.001	AC-FT=	1.448
ROUTE	1600.01	6	13	.18900	121.08	11.120	1.10322	2.133	1.001		
COMPUTE NM HYD	1601.00	-	7	.01600	15.67	.843	.98742	1.500	2.307	PER IMP=	50.00
ROUTE RESERVOIR	501.00	3	6	.01600	14.14	.843	.98750	1.633	1.381	AC-FT=	.257
ADD HYD	1601.10	13& 6	8	.20500	128.17	11.963	1.09419	2.066	.977		
ROUTE	1601.11	8	13	.20500	128.06	11.963	1.09420	2.100	.976		

AHYMO SUMMARY TABLE (AHYM0491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/17/1994

INPUT FILE = agex100.dat USER NO.= MOLZCONM,131

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	2.350
COMPUTE NM HYD	1600.00	-	3	.18900	230.18	18.441	1.82947	1.933	1.903	PER IMP=	75.00
ROUTE RESERVOIR	501.00	3	6	.18900	177.30	18.441	1.82946	2.200	1.470	AC-FT=	3.687
ROUTE	1600.01	6	13	.18900	177.46	18.441	1.82946	2.200	1.467		
COMPUTE NM HYD	1601.00	-	3	.01600	41.72	1.447	1.69571	1.500	4.074	PER IMP=	60.00
ROUTE RESERVOIR	501.00	3	6	.01600	20.73	1.447	1.69569	1.667	2.044	AC-FT=	.467
ADD HYD	1601.10	13&	6	8	.20500	187.41	19.888	1.81902	2.200	1.428	
ROUTE	1601.11	8	13	.20500	187.14	19.888	1.81902	2.200	1.426		

APPENDIX C

SUMMARY PRINTOUTS
BASIN FAR EAST END

AHYMO SUMMARY TABLE (AHYM0491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/17/1994

INPUT FILE = **teex10.dat**. USER NO.= MGLZCONM.101

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	1.570
COMPUTE NM HYD	1700.00	-	3	.22600	77.45	9.261	.76830	2.266	.549	PER IMP=	95.00
ROUTE RESERVOIR	501.00	3	4	.22600	44.35	9.261	.76830	2.633	.307	AC-FT=	3.294
COMPUTE NM HYD	1701.00	-	2	.01600	24.24	.772	.90416	1.500	2.367	PER IMP=	50.00
ROUTE RESERVOIR	501.00	2	3	.01600	8.98	.772	.90414	1.733	.877	AC-FT=	.245
ADD HYD	1701.10	3& 4	6	.24200	46.41	10.032	.77727	2.766	.300		
ROUTE	1701.11	6	13	.24200	46.40	10.032	.77727	2.766	.300		

AHYMO SUMMARY TABLE (AHYMO491) - AMAFCA VERSION OF HYMO - APRIL, 1991 RUN DATE (MON/DAY/YR) =06/17/1994

INPUT FILE = feex100.dat USER NO.= MOLZ1ONM.101

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION	
RAINFALL TYPE= 1										RAIN6=	2.350
COMPUTE NM HYD	1700.00	-	3	.22600	153.96	17.265	1.43242	2.266	1.064	PER IMP=	35.00
ROUTE RESERVOIR	301.00	3	4	.22600	92.64	17.265	1.43242	2.766	.640	AC-FT=	6.364
COMPUTE NM HYD	1701.00	-	2	.01600	40.13	1.363	1.59686	1.500	3.919	PER IMP=	50.00
ROUTE RESERVOIR	501.00	2	3	.01600	14.27	1.363	1.59684	1.766	1.394	AC-FT=	.506
ADD HYD	1701.10	3& 4	5	.24200	96.91	18.628	1.44329	2.700	.626		
ROUTE	1701.11	5	13	.24200	96.82	18.628	1.44329	2.733	.625		

APPENDIX D

COMPUTER PRINTOUTS

COMPUTER PRINTOUTS ARE UNDER SEPARATE COVER

APPENDIX E

PLATES