TERMINATION AND RELEASE OF DRAINAGE COVENANT
Drainage File No. H16-D109

THIS TERMINATION AND RELEASE OF DRAINAGE COVENANT is made and entered into as of the 13th day of June, 2013, by the City of Albuquerque, (the "City").

WHEREAS, **Far West Inns, Inc.,** as Owner, entered into a Drainage Covenant ("Covenant") dated **August 15, 1995** recorded on **August 18, 1995**, in the office of the County Clerk of Bernalillo County, State of New Mexico, in Book **95-19**, pages **8318** to **8322** as Document # **95082690**.

Tract A of Plat entitled Plat of Tracts A & B, Sandia Foundation-AMAFCA Suvdivision recorded on **March 23, 1994**, in the office of the County Clerk of Bernalillo County, State of New Mexico, in Book **C-94**, folio **96**.

WHEREAS, the City has determined that the Covenant is no longer required.

THEREFORE, City terminates the Covenant and hereby releases and discharges the Property described in the grant of Covenant:

IN WITNESS WHEREOF, the undersigned has executed this Termination and Release of Drainage Covenant as of the day and year first above written.

RECOMMENDED:
CITY HYDROLOGIST

By: _____
Curtis Cherne, City Hydrologist
Date: 6-7-13

APPROVED:
CITY OF ALBUQUERQUE

By: _____
Bryan Wolfe, City Engineer
Date: 6/13/2013

6-11-13

6-7-13

STATE OF NEW MEXICO)
) **ss.**
COUNTY OF BERNALILLO)

This instrument was acknowledged before me on June 13, 2013 by **Bryan Wolfe, City Engineer**, on behalf of the City of Albuquerque, a New Mexico municipal corporation, on behalf of the corporation.

Notary Public
My Commission Expires:

OFFICIAL SEAL
LISA CORNEJO
NOTARY PUBLIC-STATE OF NEW MEXICO
My commission expires 7-5-2014

Doc# 2013069014
06/20/2013 09:47 AM Page: 1 of 2
TERM R:$25.00 M. Toulouse Oliver, Bernalillo County

EXHIBIT A

LEGAL DESCRIPTION OF THE PROPERTY

Plat of Tracts A-1 & A-2 (formerly Tract A) Sandia Foundation-A.M.A.F.C.A. Subdivision, filed in the office of the County Clerk of Bernalillo County, New Mexico on August 26, 1999 in Plat Book 99C, page 247; formerly known as Tract A of Plat entitled Plat of Tracts A & B, Sandia Foundation-AMAFCA subdivision filed March 23, 1994 filed in Plat Book C-94, folio 96, in Bernalillo County, New Mexico.

[end of page]

Bernalillo County, NM
One Civic Plaza NW
P.O. Box 542
Albuquerque, NM 87102

Receipt: 0473404

Product	Name	Extended
TERM	Termination	$25.00
	# Pages	2
	Document #2013069014	
	# Of Entries	0
	In Person/Interested Person	false
Total		$25.00

Tender (Check) $25.00
Check#1119
Paid ByInterstate Resteraunts LLC

Thank You!

6/20/13 9:47 AM dojgarcia

[redacted]

MARK GOODWIN

& ASSOCIATES

CONSULTING ENGINEERS

DRAINAGE CALCULATIONS

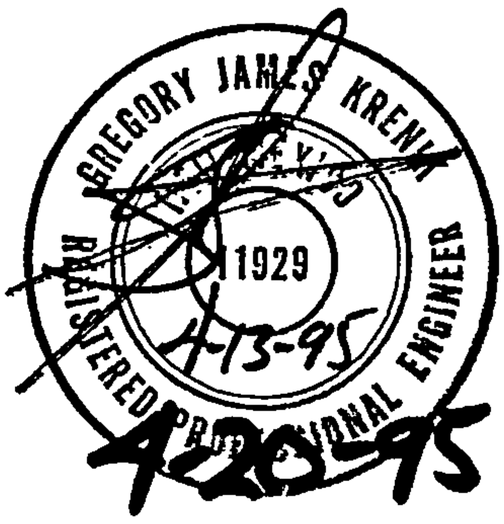
for

SHONEY'S SUITES

Prepared For

ShoLodge Inc.
600 W. Main Street
Gallatin, TN 37066

April 1995



5-19-95

D. Mark Goodwin & Associates, P.A.
Consulting Engineers and Surveyors

PROJECT HONEYS HOTEL
SUBJECT DRAINAGE CALCS
BY GSK DATE 4-20-95
CHECKED _____ DATE _____
SHEET 1 OF ____
REVISED 5-19-95

TOTAL SITE = 4.68 AC. * NO OFFSITE FLOWS ENTER THE SITE.

- THE SITE WILL HAVE THE WESTERN HALF DEVELOPED FIRST.
- THE EASTERN HAVE WILL HAVE FILL ADDED TO LEVEL IT AT 5100.00
- FREE DISCHARGE WILL BE ALLOWED FOR THE ENTIRE SITE WHEN DOWN STREAM IMPROVEMENTS PER THE "MENAUL DETENTION BASIN" ARE DONE. A TEMPORARY RETENTION POND WILL RETAIN ALL WATER OVER AND ABOVE THE HISTORICAL DISCHARGE.
- PER SMITH ENGINEERING'S PLANS FOR THE "MENAUL DETENTION BASIN AND STORMDRAIN PLAN SHEET 4 OF 29.
  - THE ALLOWABLE Q FOR BASIN 1 (WHICH THIS SITE IS PART OF) IS 141 CFS.
  - PER PAT CONLEY OF SMITH ENGINEERING THIS FLOW WAS CALCULATED WITH THIS SITE AS FULLY DEVELOPED.
- PER CARLOS MONTOYA OF CITY HYDROLOGY, THE IMPROVEMENTS WITHIN THE STORMDRAIN PLANS WILL MAKE IT POSSIBLE FOR THIS SITE TO FREE DISCHARGE. PHASE I OF THE PLANS IS TO TAKE CARE OF THE FLOODING AT UNIVERSITY AND MENAUL. CONSTRUCTION IS SET TO BEGIN IN ONE TO TWO YEARS.

Proposed

WESTERN HALF (HOTEL)

AREA = 2.325 AC

TYPE "B" = 0.6825 AC = 29.35%
TYPE "D" = 1.6425 AC = 70.65%

TP = 0.1333 HR DT = 0.03333 HR P_1 = 2.0 in P_6 = 2.3 in P_{24} = 2.65 in

FROM HYMO SHEETS 2-4

Q = 9.20 CFS
V = 0.3260 AC-FT

$V_{10\,day} = 0.3260 + \frac{1.6425\,(3.95-2.3)}{12}$

= 0.551844 AC-FT = 24,038 CF

EASTERN HALF (VACANT)

AREA = 2.355 AC

TYPE "B" = 2.2585 AC = 95.9%
TYPE "D" = 0.0965 AC = 4.1%

FROM HYMO SHEETS 5-7

Q = 5.49 CFS
V = 0.1594 AC-FT

$V_{10\,day} = 0.1594 + \frac{0.965\,(3.95-2.3)}{12}$

= 0.17267 AC-FT = 3522 CF

- CALCULATE ENTIRE SITE AS DEVELOPED

ASSUME TYPE "C" = 22% PER SMITH REPORT
TYPE "D" = 78%

TOTAL SITE = 4.68 AC

FROM HYMO SHEETS 8-10 Q = 20.25 CFS V = 0.7235 AC-FT

```
START                    TIME=0.0
**** HYDROGRAPH FOR SHONEY'S HOTEL
RAINFALL                 TYPE=1 RAIN QUARTER=0.0 IN
                         RAIN ONE=2.00 IN RAIN SIX=2.30 IN
                         RAIN DAY=2.65 IN DT=0.03333 HR
COMPUTE NM HYD           ID=1 HYD NO=101.1 AREA=0.003633 SQ MI
                         PER A=00.00 B=29.35  C=00  D=70.65
                         TP=0.1333 HR MASS RAINFALL=-1
PRINT HYD                 ID=1 CODE=1
FINISH
```

AHYMO PROGRAM (AHYMO194) - AMAFCA Hydrologic Model - January, 1994
RUN DATE (MON/DAY/YR) = 04/19/1995
START TIME (HR:MIN:SEC) = 11:38:42 USER NO.= M_GOODWN.I01
INPUT FILE = shonhot.dat

START TIME=0.0

***** HYDROGRAPH FOR SHONEY'S HOTEL

RAINFALL TYPE=1 RAIN QUARTER=0.0 IN

RAIN ONE=2.00 IN RAIN SIX=2.30 IN

RAIN DAY=2.65 IN DT=0.03333 HR

COMPUTED 6-HOUR RAINFALL DISTRIBUTION BASED ON NOAA ATLAS 2 - PEAK AT 1.40 HR

DT = .033330 HOURS END TIME = 5.999400 HOURS

.0000	.0013	.0026	.0039	.0052	.0066	.0080
.0095	.0109	.0125	.0140	.0156	.0173	.0190
.0207	.0225	.0243	.0263	.0282	.0303	.0324
.0346	.0369	.0392	.0417	.0443	.0470	.0498
.0528	.0560	.0593	.0649	.0708	.0771	.0906
.1210	.1677	.2348	.3263	.4467	.6001	.7911
1.0243	1.2408	1.3312	1.4075	1.4753	1.5371	1.5939
1.6467	1.6960	1.7423	1.7858	1.8268	1.8656	1.9023
1.9370	1.9700	2.0013	2.0310	2.0592	2.0654	2.0711
2.0765	2.0816	2.0865	2.0911	2.0956	2.0999	2.1041
2.1081	2.1119	2.1156	2.1192	2.1227	2.1261	2.1294
2.1327	2.1358	2.1388	2.1418	2.1447	2.1476	2.1503
2.1531	2.1557	2.1583	2.1609	2.1634	2.1658	2.1682
2.1706	2.1729	2.1752	2.1775	2.1797	2.1819	2.1840
2.1861	2.1882	2.1902	2.1922	2.1942	2.1962	2.1981
2.2000	2.2019	2.2037	2.2056	2.2074	2.2092	2.2109
2.2127	2.2144	2.2161	2.2178	2.2194	2.2211	2.2227
2.2243	2.2259	2.2274	2.2290	2.2305	2.2321	2.2336
2.2351	2.2365	2.2380	2.2395	2.2409	2.2423	2.2437
2.2451	2.2465	2.2479	2.2493	2.2506	2.2519	2.2533
2.2546	2.2559	2.2572	2.2585	2.2597	2.2610	2.2623
2.2635	2.2647	2.2660	2.2672	2.2684	2.2696	2.2708
2.2719	2.2731	2.2743	2.2754	2.2766	2.2777	2.2788
2.2800	2.2811	2.2822	2.2833	2.2844	2.2855	2.2865
2.2876	2.2887	2.2897	2.2908	2.2918	2.2929	2.2939
2.2949	2.2960	2.2970	2.2980	2.2990	2.3000	

COMPUTE NM HYD ID=1 HYD NO=101.1 AREA=0.003633 SQ MI

PER A=00.00 B=29.35 C=00 D=70.65

TP=0.1333 HR MASS RAINFALL=-1

K = .072649HR TP = .133300HR K/TP RATIO = .345000 SHAPE CONSTANT, N = 7.106420
UNIT PEAK = 10.134 CFS UNIT VOLUME = .9982 B = 526.28
P60 = 2.0000
[illegible]

NCHES PER HOUR
RUNOFF COMPUTED BY INITIAL ABSTRACTION/INFILTRATION NUMBER METHOD - DT
.033330

4

K = .131967HR TP = .133300HR K/TP RATIO = .990000 SHAPE
CONSTANT, N = 3.566429
UNIT PEAK = 2.6009 CFS UNIT VOLUME = .9951 B = 325.15
P60 = 2.0000
AREA = .001066 SQ MI IA = .50000 INCHES INF = 1.25000 I
NCHES PER HOUR
RUNOFF COMPUTED BY INITIAL ABSTRACTION/INFILTRATION NUMBER METHOD - DT
.033330

PRINT HYD ID=1 CODE=1

PARTIAL HYDROGRAPH 101.10

RUNOFF VOLUME = 1.68260 INCHES = .3260 ACRE-FEET
PEAK DISCHARGE RATE = 9.20 CFS AT 1.500 HOURS BASIN AREA =
.0036 SQ. MI.

```
START                    TIME=0.0
**** HYDROGRAPH FOR SHONEY'S HOTEL UNDEVELOPED SIDE
RAINFALL                 TYPE=1 RAIN QUARTER=0.0 IN
                         RAIN ONE=2.00 IN RAIN SIX=2.30 IN
                         RAIN DAY=2.65 IN DT=0.03333 HR
COMPUTE NM HYD           ID=1 HYD NO=101.1 AREA=0.00368 SQ MI
                         PER A=00.00 B=95.90  C=00  D=4.10
                         TP=0.1333 HR MASS RAINFALL=-1
PRINT HYD                 ID=1 CODE=1
FINISH
```

6

```
AHYMO PROGRAM (AHYMO194) - AMAFCA Hydrologic Model - January, 1994
     RUN DATE (MON/DAY/YR) = 04/19/1995
     START TIME (HR:MIN:SEC) = 06:44:52     USER NO.= M_GOODWN.I01
     INPUT FILE = SHONDIRT.DAT


START                 TIME=0.0

***** HYDROGRAPH FOR SHONEY'S HOTEL UNDEVELOPED SIDE

RAINFALL              TYPE=1 RAIN QUARTER=0.0 IN

                      RAIN ONE=2.00 IN RAIN SIX=2.30 IN

                      RAIN DAY=2.65 IN DT=0.03333 HR


                  COMPUTED 6-HOUR RAINFALL DISTRIBUTION BASED ON NOAA ATLAS 2
- PEAK AT 1.40 HR.
                  DT =      .033330 HOURS       END TIME =      5.999400 HOURS
                     .0000    .0013    .0026    .0039    .0052    .0066    .0080
                     .0095    .0109    .0125    .0140    .0156    .0173    .0190
                     .0207    .0225    .0243    .0263    .0282    .0303    .0324
                     .0346    .0369    .0392    .0417    .0443    .0470    .0498
                     .0523    .0560    .0593    .0649    .0708    .0771    .0906
                     .1210    .1677    .2348    .3263    .4467    .6001    .7911
                    1.0243   1.2406   1.3312   1.4075   1.4753   1.5371   1.5739
                    1.6467   1.6960   1.7423   1.7858   1.8266   1.8656   1.9023
                    1.9370   1.9700   2.0013   2.0310   2.0592   2.0654   2.0711
                    2.0765   2.0816   2.0865   2.0911   2.0956   2.0999   2.1041
                    2.1081   2.1119   2.1156   2.1192   2.1227   2.1261   2.1294
                    2.1327   2.1358   2.1388   2.1418   2.1447   2.1476   2.1503
                    2.1531   2.1557   2.1583   2.1609   2.1634   2.1658   2.1682
                    2.1706   2.1729   2.1752   2.1775   2.1797   2.1819   2.1840
                    2.1861   2.1882   2.1902   2.1922   2.1942   2.1962   2.1981
                    2.2000   2.2019   2.2037   2.2056   2.2074   2.2092   2.2109
                    2.2127   2.2144   2.2161   2.2178   2.2194   2.2211   2.2227
                    2.2243   2.2259   2.2274   2.2290   2.2305   2.2321   2.2336
                    2.2351   2.2365   2.2380   2.2395   2.2409   2.2423   2.2437
                    2.2451   2.2465   2.2479   2.2493   2.2506   2.2519   2.2533
                    2.2546   2.2559   2.2572   2.2585   2.2597   2.2610   2.2623
                    2.2635   2.2647   2.2660   2.2672   2.2684   2.2696   2.2708
                    2.2719   2.2731   2.2743   2.2754   2.2766   2.2777   2.2788
                    2.2800   2.2811   2.2822   2.2833   2.2844   2.2855   2.2865
                    2.2876   2.2887   2.2897   2.2908   2.2918   2.2929   2.2939
                    2.2949   2.2960   2.2970   2.2980   2.2990   2.3000

COMPUTE NM HYD        ID=1 HYD NO=101.1 AREA=0.00368 SQ MI

                      PER A=00.00 B=95.90  C=00  D=4.10

                      TP=0.1333 HR MASS RAINFALL=-1

      K =    .072649HR    TP =    .133300HR    K/TP RATIO =    .545000    SHAPE
CONSTANT, N =   7.106420
      UNIT PEAK =    .59560    CFS    UNIT VOLUME =    .9815    B =    326.28
      P60 = 2.0000
```

NCHES PER HOUR
RUNOFF COMPUTED BY INITIAL ABSTRACTION/INFILTRATION NUMBER METHOD - DT
.033330

7

K = .131967HR TP = .133300HR K/TP RATIO = .990000 SHAPE
CONSTANT, N = 3.566429
UNIT PEAK = 8.6082 CFS UNIT VOLUME = .9987 B = 325.1E
P60 = 2.0000
AREA = .003329 SQ MI IA = .50000 INCHES INF = 1.35000 I
NCHES PER HOUR
RUNOFF COMPUTED BY INITIAL ABSTRACTION/INFILTRATION NUMBER METHOD - DT
.033330

PRINT HYD ID=1 CODE=1

PARTIAL HYDROGRAPH 101.10

RUNOFF VOLUME = .6 028 INCHES = .1594 ACRE-FEET
PEAK DISCHARGE RATE = 5.49 CFS AT 1.533 HOURS BASIN AREA =
.0037 SQ. MI.

```
TART                    TIME=0.0
**** HYDROGRAPH FOR SHONEY'S HOTEL
RAINFALL                TYPE=1 RAIN QUARTER=0.0 IN
                        RAIN ONE=2.00 IN RAIN SIX=2.30 IN
                        RAIN DAY=2.65 IN DT=0.03333 HR
COMPUTE NM HYD          ID=1 HYD NO=101.1 AREA=0.007313 SQ MI
                        PER A=00.00 B=00.00  C=22.00  D=78.00
                        TP=0.1333 HR MASS RAINFALL=-1
PRINT HYD                ID=1 CODE=1
FINISH
```

AHYMO PROGRAM (AHYMO194) - AMAFCA Hydrologic Model - January 1994
RUN DATE (MON/DAY/YR) = 05/13/1993
START TIME (HR:MIN:SEC) = 07:43:08   USER NO.= M_[illegible]
INPUT FILE = SHON.DAT

START   TIME=0.0

***** HYDROGRAPH FOR SHONEY'S HOTEL

RAINFALL   TYPE=1 RAIN QUARTER=0.0 IN

RAIN ONE=2.00 IN RAIN SIX=[illegible] IN

RAIN DAY=2.65 IN DT=0.[illegible] HR

COMPUTED 6-HOUR RAINFALL DISTRIBUTION BASED ON NOAA ATLAS 2 - PEAK AT [illegible] HR.

DT = .0[illegible]30 HOURS   END TIME = [illegible] HOURS

.0000	.0013	.0026	.0039	.0052	.0066	[illegible]
.0095	.0109	.0125	.0140	[illegible]	.0173	.0190
.0207	.0225	.0243	.0263	.0283	[illegible]	.0324
.0346	.0369	.0392	.0417	.0443	[illegible]	.0499
.0529	.0560	.0593	.0649	[illegible]	[illegible]	[illegible]
.1210	.1677	.2748	.3763	[illegible]	[illegible]	[illegible]
1.0343	1.2408	1.3312	1.4[illegible]	[illegible]	[illegible]	[illegible]
1.6467	1.6760	1.7423	1.7958	[illegible]	[illegible]	1.9023
1.9370	1.9700	2.0013	[illegible]	[illegible]	[illegible]	[illegible]
2.0763	2.0816	2.0865	[illegible]	[illegible]	2.0997	[illegible]
2.1081	2.1119	2.1156	[illegible]	2.1227	[illegible]	2.1294
2.1327	2.1358	2.1388	2.1419	[illegible]	2.1476	[illegible]
2.1531	2.1557	2.1583	2.1609	[illegible]	[illegible]	[illegible]
2.1706	2.1729	2.1752	2.1775	[illegible]	[illegible]	[illegible]
2.1861	2.1882	2.1902	[illegible]	[illegible]	[illegible]	[illegible]
2.2000	[illegible]	2.2037	2.2056	[illegible]	[illegible]	[illegible]
2.2127	2.2144	2.2161	2.2178	[illegible]	[illegible]	[illegible]
2.2243	2.2258	2.2274	[illegible]	[illegible]	[illegible]	[illegible]
2.2351	2.2365	2.2380	2.2395	[illegible]	[illegible]	2.2437
2.2451	2.2465	2.2479	[illegible]	[illegible]	[illegible]	[illegible]
2.2546	2.2559	2.2572	[illegible]	[illegible]	[illegible]	[illegible]
2.2635	2.2647	2.2660	[illegible]	[illegible]	2.2696	[illegible]
2.2719	2.2731	2.2743	[illegible]	2.2766	2.2777	2.2788
2.2800	2.2811	2.2822	2.2833	2.2844	2.2855	2.2865
2.2876	2.2887	2.2897	2.2908	2.2918	[illegible]	[illegible]
2.2949	2.2960	2.2970	[illegible]	[illegible]	2.3000	

COMPUTE NM HYD   ID=1 HYD NO=101.1 AREA=0.007317 SQ MI

PER A=00.00 B=00.00 C=12.[illegible] [illegible]

TP=0.[illegible] HR MASS RAINFALL=-1

[illegible]

CONSTANT [illegible]

P60 = 2.0000
AREA = .005704 SQ MI IA = .10000 INCHES INF = .04000 I
NCHES PER HOUR
RUNOFF COMPUTED BY INITIAL ABSTRACTION/INFILTRATION NUMBER METHOD - DT
= .033330

10

K = .107331HR TP = .133300HR K/TP RATIO = .805200 SHAPE
CONSTANT, N = 4.445615
UNIT PEAK = 4.6330 CFS UNIT VOLUME = .9975 B = 383.86
P60 = 2.0000
AREA = .001609 SQ MI IA = .35000 INCHES INF = .83000 I
NCHES PER HOUR
RUNOFF COMPUTED BY INITIAL ABSTRACTION/INFILTRATION NUMBER METHOD - DT
= .033330

PRINT HYD ID=1 CODE=1

PARTIAL HYDROGRAPH 101.00

RUNOFF VOLUME = 1.85514 INCHES = .7273 ACRE-FEET
PEAK DISCHARGE RATE = [illegible] CFS AT 1.[illegible] HOURS BASIN AREA =
.0073 SQ. MI.

D. Mark Goodwin & Associates, P.A.
Consulting Engineers and Surveyors

PROJECT SHONEYS HOTEL
SUBJECT DRAINAGE CALCS
BY GJK DATE 4-20-95
CHECKED _____ DATE _____
SHEET 11 OF ____
REVISED 5-19-95

- SIZE TEMPORARY RETENTION POND

ELEVATION	AREA (SF)	STORAGE (CF)
91	1750	0
92	2606	2178
93	3596	5254
94	4579	9316.5
95	5695	14,453.5
96	6883	20,742.5

- DETERMINE DIRECT DISCHARGE

WEST HALF OF DEVELOPED SIDE

AREA = 1.08137 AC
TYPE B = 0.28361 AC = 26.23 %
TYPE D = 0.79776 AC = 73.77 %

FROM HYMO OUTPUT SHEETS 12-14
Q = 4.37 CFS
V = 0.1553 AC-FT

4.37 CFS < 7.06 CFS OK

TOTAL SITE UNDEVELOPED

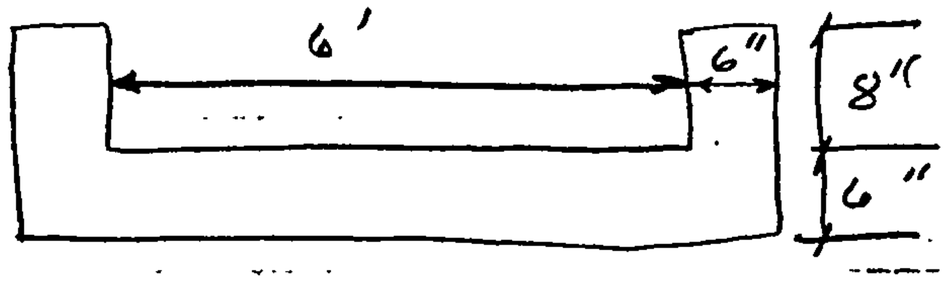
TYPE A = 100 %

FROM HYMO OUTPUT SHEETS 15-17
Q = 7.06 CFS
V = 0.2001 AC-FT

- SIZE CHANNEL TO MENAUL

THIS SHOULD BE SIZED FOR BUILDOUT

Q = 20.25 - 4.37 = 15.88 CFS



S = 1% n = 0.013

d = 0.5
wp = 7.0
A = 3.0
V = 6.50 F/S
Q = 19.49 CFS OK

```
START                    TIME=0.0
*** HYDROGRAPH FOR SHONEY'S HOTEL - EXISTING COND
RAINFALL                 TYPE=1 RAIN QUARTER=0.0 IN
                         RAIN ONE=2.00 IN RAIN SIX=2.30 IN
                         RAIN DAY=2.65 IN DT=0.03333 HR
COMPUTE NM HYD           ID=1 HYD NO=101.1 AREA=0.007313 SQ MI
                         PER A=100.00 B=00  C=00  D=00
                         TP=0.1333 HR MASS RAINFALL=-1
PRINT HYD                 ID=1 CODE=1
FINISH
```

AHYMO PROGRAM (AHYMO194) - AMAFCA Hydrologic Model - January, 1994
RUN DATE (MON/DAY/YR) = 05/18/1995
START TIME (HR:MIN:SEC) = 07:36:25 USER NO.= M_GOODWN.I01
INPUT FILE = SHONEX.DAT

START TIME=0.0

***** HYDROGRAPH FOR SHONEY'S HOTEL - EXISTING COND

RAINFALL TYPE=1 RAIN QUARTER=0.0 IN

RAIN ONE=2.00 IN RAIN SIX 2.30 IN

RAIN DAY=2.65 IN DT=0.03333 HR

COMPUTED 6-HOUR RAINFALL DISTRIBUTION BASED ON NOAA ATLAS 2
PEAK AT 1.40 HR.

DT = .033330 HOURS END TIME = 5.999400 HOURS

.0000	.0013	.0026	.0039	.0052	.0066	.0080
.0095	.0109	.0125	.0140	.0156	.0173	.0190
.0207	.0225	.0243	.0263	.0282	.0303	.0324
.0346	.0369	.0392	.0417	.0443	.0470	.0495
.0528	.0560	.0593	.0649	.0706	.0771	.0906
[illegible]	.1677	.2348	.3263	.4467	.6001	.7911
1.0243	1.2408	1.3312	1.4075	1.4753	1.5371	1.5939
1.6437	1.6960	1.7423	1.7856	1.8265	1.8656	1.9020
1.9370	1.9700	2.0013	2.0310	2.0592	2.0654	2.0711
2.0765	2.0816	2.0865	2.0911	2.0956	2.0999	2.1041
[illegible]	2.1119	2.1156	2.1192	2.1227	2.1261	2.1294
2.1327	2.1358	2.1388	2.1418	2.1447	2.1476	2.1503
2.1531	2.1557	2.1583	2.1609	2.1634	2.1658	2.1682
2.1706	2.1729	2.1752	2.1775	2.1797	2.1819	2.1840
2.1861	2.1882	2.1902	2.1922	2.1942	2.1962	2.1981
2.2000	2.2019	2.2037	2.2056	2.2074	2.2092	2.2107
2.2127	2.2144	2.2161	2.2177	2.2194	2.2211	2.2227
2.2243	2.2259	2.2274	2.2290	2.2305	2.2321	2.2336
2.2351	2.2365	2.2380	2.2395	2.2409	2.2423	2.2437
2.2451	2.2465	2.2479	2.2493	2.2506	2.2519	2.2533
2.2546	2.2559	2.2572	2.2585	2.2597	2.2610	2.2622
2.2635	2.2647	2.2660	2.2672	2.2684	2.2696	2.2708
2.2719	2.2731	2.2743	2.2754	2.2766	2.2777	2.2788
2.2800	2.2811	2.2822	2.2833	2.2844	2.2855	2.2865
2.2876	2.2887	2.2897	2.2908	2.2918	2.2929	2.2939
2.2949	2.2960	2.2970	2.2980	2.2990	2.3000	

COMPUTE NM HYD ID=1 HYD NO=101.1 AREA=0.00[illegible] SQ MI

PER A=100.00 B=00 C=00 D=00

TP=0.[illegible] HR MASS RAINFALL=-1

[illegible] TP = .133300 HR [illegible] TP RATIO = 1.203330 [illegible]
CONSTANT [illegible]
[illegible] CFS UNIT VOLUME = .9988 B = [illegible]

AREA = .007313 SQ MI IA = .65000 INCHES INF = 1.67100 I
NCHES PER HOUR
RUNOFF COMPUTED BY INITIAL ABSTRACTION/INFILTRATION NUMBER METHOD -- DT
= .733330

PRINT HYD ID=1 CODE=1

PARTIAL HYDROGRAPH 101.10

RUNOFF VOLUME = .01315 INCHES = .2001 ACRE-FEET
PEAK DISCHARGE RATE = .04 CFS AT 1.533 HOURS BASIN AREA =
.0073 SQ. MI.

FINISH

NORMAL PROGRAM FINISH END TIME (HR:MIN:SEC) = 07:36:14

```
START                    TIME=0.0
**** HYDROGRAPH FOR SHONEY'S HOTEL - DIRECT DISCH
RAINFALL                 TYPE=1 RAIN QUARTER=0.0 IN
                         RAIN ONE=2.30 IN RAIN SIX=2.30 IN
                         RAIN DAY=2.65 IN DT=0.03333 HR
COMPUTE NM HYD           ID=1 HYD NO=101.1 AREA=0.00149 SQ MI
                         PER A=00.00 B=26.23  C=00  D=73.77
                         TP=0.1333 HR MASS RAINFALL=-1
PRINT HYD                 ID=1 CODE=1
FINISH
```

AHYMO PROGRAM (AHYMO194) - AMAFCA Hydrologic Model - January, 1994
RUN DATE (MON/DAY/YR) = 05/18/1995
START TIME (HR:MIN:SEC) = 08:43:20 USER NO.= M_GOODWN.I01
INPUT FILE = SHONDIR.DAT

START TIME=0.0

***** HYDROGRAPH FOR SHONEY'S HOTEL - DIRECT DISCH .

RAINFALL TYPE=1 RAIN QUARTER=0.0 IN

RAIN ONE=2.00 IN RAIN SIX=2.30 IN

RAIN DAY=2.65 IN DT=0.03333 HR

COMPUTED 6-HOUR RAINFALL DISTRIBUTION BASED ON NOAA ATLAS 2
- PEAK AT 1.40 HR.

DT = .033330 HOURS END TIME = 5.999400 HOURS

.0000	.0013	.0026	.0039	.0052	.0066	.0081
.0095	.0109	.0125	.0140	.0156	.0173	.0190
.0207	.0225	.0243	.0262	.0282	.0303	.0324
.0346	.0369	.0392	.0417	.0443	.0470	.0498
.0529	.0560	.0593	.0649	.0706	.0771	.0906
.1210	.1677	.2348	.3263	.4467	.6001	.7911
1.0243	1.3408	1.3712	1.4075	1.4753	1.5371	1.5939
[illegible]	1.6960	1.7423	1.7856	1.8265	1.8656	1.9033
1.9370	1.9700	2.0013	2.0310	2.0592	[illegible]	2.0711
2.0765	2.0816	2.0865	2.0911	2.0956	2.0999	2.1041
2.1081	2.1119	2.1156	2.1193	2.1227	2.1261	2.1294
2.1327	2.1358	2.1388	2.1418	2.1447	2.1475	2.1503
2.1531	2.1557	2.1583	2.1609	2.1634	2.1658	2.1682
2.1706	2.1729	2.1752	2.1775	2.1797	2.1819	2.1840
2.1861	2.1882	2.1902	2.1922	2.1942	2.1962	2.1981
2.2000	2.2019	2.2037	2.2056	2.2074	2.2092	2.2109
2.2127	2.2144	2.2161	2.2178	2.2194	2.2211	2.2227
2.2243	2.2259	2.2274	2.2290	2.2305	2.2321	2.2336
2.2351	2.2365	2.2380	2.2395	2.2409	2.2423	2.2437
2.2451	2.2465	2.2479	2.2493	2.2506	2.2519	2.2533
2.2546	2.2559	2.2572	2.2585	2.2597	2.2610	2.2623
2.2635	2.2647	2.2660	2.2672	2.2684	2.2696	2.2708
2.2719	2.2731	2.2743	2.2754	2.2766	2.2777	2.2788
2.2800	2.2811	2.2822	2.2833	2.2844	2.2855	2.2865
2.2876	2.2887	2.2897	2.2908	2.2918	2.2929	2.2939
2.2949	2.2960	2.2970	2.2980	2.2990	2.3000	

COMPUTE NM HYD ID=1 HYD NO=101.1 AREA=0.[illegible]169 SQ MI

PER A=00.00 B=26.23 C=00 D=73.77

TP=0.1333 HR MASS RAINFALL=-1

K = .07264 HR TP = .133300HR K/TP RATIO = .545000 SHAPE
CONSTANT, N = 3.106420
[illegible]

```
    P60 = 2.0000
       AREA =        .001247 SQ MI     1A =     .10000 INCHES     INF =     .04000 I
NCHES PER HOUR
       RUNOFF COMPUTED BY INITIAL ABSTRACTION/INFILTRATION NUMBER METHOD - DT
=    .033330
```

17

```
       K =    .131967HR     TP =    .133300HR    K/TP RATIO =    .990000      SHAPE
 CONSTANT, N =  3.556429
       UNIT PEAK =   1.0613     CFS   UNIT VOLUME =     .9981         B =   325.13
    P60 = 2.0000
       AREA =        .000443 SQ MI     IA =     .50000 INCHES     INF =    1.25000 I
NCHES PER HOUR
       RUNOFF COMPUTED BY INITIAL ABSTRACTION/INFILTRATION NUMBER METHOD - DT
=    .033330

     PRINT HYD             ID=1 CODE=1

                                     PARTIAL HYDROGRAPH    101.10

       RUNOFF VOLUME =      1.72340 INCHES      =        .1557 ACRE-FEET
       PEAK DISCHARGE RATE =        4.77 CFS  AT    1.500 HOURS   BASIN AREA =
 .0017 SQ. MI.
```

D. Mark Goodwin & Associates, P.A.
Consulting Engineers and Surveyors

PROJECT SHONEYS HOTEL
SUBJECT DRAINAGE CALCS
BY GJK DATE 4-20-95
CHECKED _____ DATE _____
SHEET 18 OF ____

• SIZE WIER OPENING FOR CHANNEL

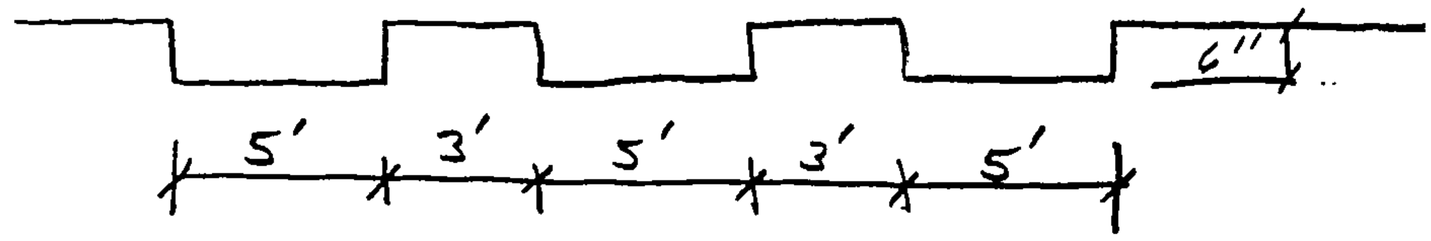
$Q = 2.95 L H^{1.5}$

$H = 0.5'$

$L = 14.49'$

$Q = 15.1125$

SAY 3 - 5' OPENINGS IN CURB.



• DETERMINE VOLUME TO GO INTO POND

FROM SHEET 1 & SHEET 11

$V = 0.326 + 0.1594 - 0.1553$

$= 0.3301$ AC-FT

$V_{10day} = 0.3301 + \frac{0.89426(3.45-2.3)}{12}$

$= 0.453$ AC-FT

$= 19,736$ CF $<$ 20,742.5 OK SEE SHEET 11

• DETERMINE IF DOUBLE O INLET IN SUMP IS OKAY

$\frac{Q}{P} = 3.0 H^{3/2}$

P = Perimeter = 12 - SIDES DO NOT COUNT
ASSUME 25% CLOGGED.

$P = 12 \times .75 = 9$

$Q = 9.20 + 5.49 - 4.37$

$= 10.32$ CFS

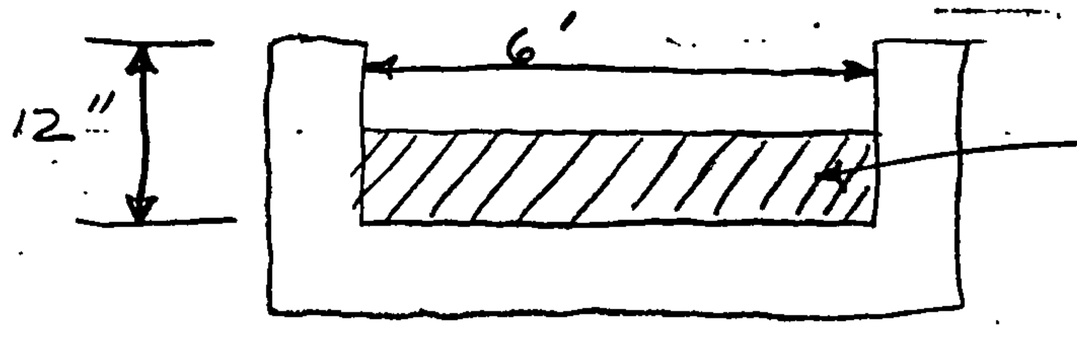
$H = 0.53 < .67$ OK

18" RCP AT 1% CARRY'S 11CFS

• SIZE SPILLWAY

CONCRETE CHANNEL WILL ACT AS SPILLWAY

WIER WILL BE OVER ASPHALT CURB



$d = 0.34$ $V = 5.1837$ F/S

$w_p = 6.68$

$A = 2.04$

$Q = 10.57$ CFS ≈ 10.32

OK

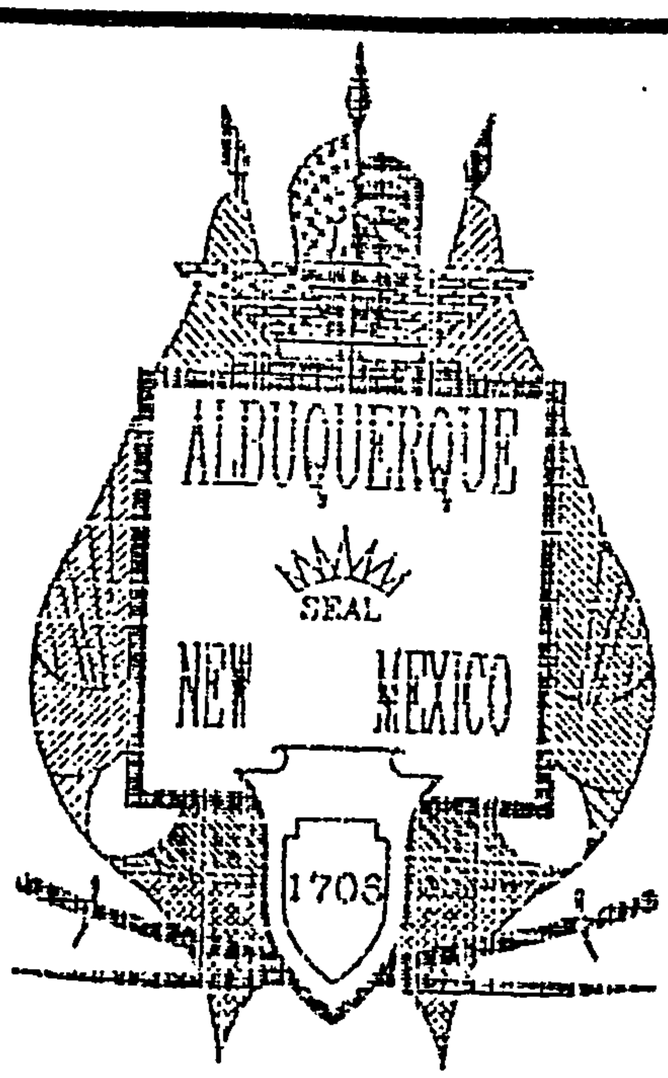
D. Mark Goodwin & Associates, P.A.
Consulting Engineers and Surveyors

PROJECT SHONEY'S HOTEL
SUBJECT DRAINAGE CALCS
BY GJK DATE 5-19-95
CHECKED _____ DATE _____
SHEET 19 OF ____

THE TEMPORARY RETENTION POND IS ONLY UNTIL THE DOWNSTREAM DRAINAGE IMPROVEMENTS ARE COMPLETED. (SEE SHEET 1)→

ONCE THE IMPROVEMENTS ARE COMPLETED (PHASE I) THEN THE 7" ASPHALT CURB IN THE 6' CHANNEL CAN BE REMOVED. THE DOUBLE "D" INLET WILL THEN BE FILLED WITH DIRT AND A CONCRETE SLAB POURED TO MATCH THE CHANNEL. THE 18" RCP WILL BE REMOVED WITHIN THE POND AREA AND CAPPED WITH CONCRETE.

APPENDIX A



DESIGN ANALYSIS REPORT

VOLUME ONE

MENAUL DETENTION BASIN AND STORM DRAIN

PROJECT NO. 4252.90

S-SEC Smith–Scheuch Engineering Company
PROJECT NO. 89-01-06

February 1992

6

MENAUL DETENTION BASIN AND

STORM DRAIN IMPROVEMENTS

Prepared for:

City of Albuquerque, Hydrology Division

Project Manager: Carlos Montoya

Prepared by:

Smith-Scheuch Engineering Company

6400 Uptown Boulevard, N.E. Ste. 500E
Albuquerque, New Mexico 87110
505/884-0700

February, 1991
Project No. 4252.90

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EXECUTIVE SUMMARY

This report was prepared under City of Albuquerque (COA) Contract No.4252-01 by Smith-Scheuch Engineering Company (S-SEC) for the Hydrology Division of the COA Public Works Department. The purpose of the Contract is to provide an improved storm drain system which will reduce flooding at the intersection of Menaul Boulevard and University Boulevard.

The basis for the improvements was taken from the Menaul/Mildred Drainage Improvements report, March 1991, and from discussions with the COA Project Manager, Mr. Carlos Montoya. This report also incorporates the comments which were received from the Public Works Department after their review of the preliminary report.

Project Area Description

The project area is divided into east and west areas (see Drainage Basin Map, Plate 1 in Appendix U). The boundaries of the east side are the North Diversion Channel between Menaul and Candelaria; Candelaria Boulevard from the North Diversion Channel west to I-25; I-25 from Candelaria south to approximately Indian School Road; and from Indian School Road east along the North Diversion Channel to Menaul Boulevard.

The boundaries of the west area are I-25 right-of-way from Candelaria north to the Motor Vehicle Department; Candelaria west of I-25 to the existing detention basin; south from the existing detention basin to Menaul Boulevard; and east on Menaul Boulevard to I-25. The latter portion is bounded on the east by I-25.

Hydrologic Analysis

The hydrologic analysis was performed utilizing the Albuquerque Metropolitan Arroyo Flood Control Authority's (AMAFCA) computer program AHYMO991. This program is a modified version of the Hydrologic Modeling (HYMO) program developed by the United States Department of Agriculture. Rainfall depths are taken from the Menaul/Mildred

Drainage Improvements report. The design storm for the storm water conveyance system was the 6-Hour, 100-Year storm. The design storm for determining the volume of the detention basin was the 24-Hour, 100-Year storm event. As directed by the COA, complete development conditions of the basins were assumed for the analysis (worst case scenario).

Storm Drain Improvements

The improvements detailed in the report were developed to provide for mitigation of the flooding at Menaul and University and to provide conveyance of storm runoff to the improved open channel. The hydraulic analysis of the system was accomplished using the computer program Hydrology, Version 6.0, by Engineering Data Systems.

The existing storm drain system is shown on Plate 2 in Appendix U, and the proposed improvements to the existing system are shown on Plate 3 in Appendix U. Minor modifications are proposed for the existing system on Phoenix Avenue, Princeton Drive, and a small portion of Menaul Boulevard. Major improvements are proposed for the system between Princeton Road and the 10-foot x 16-foot corrugated metal pipe arch (CMPA) culvert under I-25, just west of the Union 76 Truck Stop, and from Menaul Road south on University. These improvements will provide for interception and conveyance of approximately 85% of the 6-Hour, 100-Year storm runoff.

Open Channel Improvements

The proposed improvements to the existing unlined channel will provide for conveyance of the runoff from the storm drain outfall at the I-25 west frontage road to the improved detention basin. The proposed improvements to the existing channel are shown on Plate 3 in Appendix U. Improvements consist of regrading the channel and providing for erosion protection at the new drop structures and at the Broadbent Parkway crossing entrance and exit. The existing 84-inch culvert at Broadbent Parkway will be replaced with a crossing which consists of 8-foot x 16-foot precast arches to accommodate larger flows. Supplier information sheets for the arches are provided in Appendix L. A baffled apron spillway is provided at the channel outfall to the detention basin. The hydraulic analysis of the channel

was performed using the computer program, Hydrology, Version 6.0, by Engineering Data Systems.

Detention Basin Improvements

Proposed improvements to the detention basin will provide approximately 66 acre-feet of storm runoff storage for the 24-Hour, 100-Year storm event, an increase of approximately 17 acre-feet over the existing capacity. The increased basin capacity will be achieved by regrading the existing basin and constructing an earthen dam at the west end.

Flows from the basin will be control discharge to the Alameda Drain at a maximum rate of 33 cubic feet per second (cfs) during the 100-Year storm event. This requirement was established by the Middle Rio Grande Conservancy District (MRGCD). The discharge rate will be achieved by placing an 18-inch diameter orifice plate over the entrance of the 54-inch principle spillway pipe at the trash rack.

Emergency spillway relief will be provided by an 80-foot wide wier at the top of the dam. A gabion apron will be provided at the base of the emergency spillway to protect against erosion of the dam embankment. The detention basin layout is presented on Plate 4 in Appendix U.

Improvements Downstream of the Dam

The existing junction-box (J-box) at Broadway Boulevard will be modified to convey the dam discharge and the Alameda Lateral discharge through the middle and north drains of the three existing parallel 48-inch reinforced concrete pipe (RCP) storm drains. The south 48-inch RCP will be used to convey discharge from the Claremont Detention basin to the Alameda Drain.

Easement Acquisition

The proposed alignments will require acquisition of approximately 43,565 square feet (1.0001 acres) of drainage easement. The locations of the required easements are portions of land on the Union 76 Truck Stop, on the FINA Truck Stop, and on a parcel of property approximately 650 feet west of the intersection of Menaul Boulevard and University Boulevard. A more detailed description of the requirements is provided in this report.

Additional Studies:

Existing Outfall

The existing outfall system from the detention basin was investigated to determine the cause of pipe flows surcharging through manhole covers during storm events. An analysis was performed on the system using the hydraulic software cited previously, and the analysis indicates that flows should back up into the Alameda Lateral prior to surcharging through the manholes.

Potholing

S-SEC coordinated with Aquavac to provide utility location services for this project. Aquavac locates utilities using water powered augering equipment that results in minimal damage to pavement and subgrade and does not damage the utilities. They were subcontracted to verify the location of utilities in the areas where the drainage improvements will be constructed. The results of the location survey are shown on the plans and have been incorporated into the design to minimize utility conflict.

Water Quality

S-SEC coordinated an additional study which was conducted by the Weston Company to determine possible sources of pollution in the drainage basins east of I-25, identify the pollutants which are present in the runoff at this time, and make recommendations on cost

effective control/treatment of the existing runoff. The final water quality report was unavailable at the time this report was prepared, therefore, an addendum to this report will be prepared and issued at a later date.

Probable Construction Cost

A preliminary opinion of probable construction cost was prepared for the proposed improvements. The cost is estimated to be $ 2,549,910 and includes taxes and a ten percent contingency. This does not include costs for easement acquisition. The detailed breakdown of the costs are provided in Appendix S.

I. INTRODUCTION

Purpose

The purpose of this study is to develop and analyze the following:

- An alternative storm drain alignment to the system proposed in the Albuquerque Metropolitan Drainage Study (AMDS);
- Improvements to the unlined channel such that the channel will convey the design storm runoff;
- An outfall structure from the channel to the detention basin;
- Improvements to the existing detention basin;
- A principal spillway for the detention basin;
- An emergency spillway for the detention basin, and;
- Proposed improvements to the existing basin outfall system.

This study analyzes alternatives which were initially identified by the S-SEC design team with concurrence by the COA Project Manager. The overall system should mitigate flooding which occurs on Menaul Boulevard while conveying storm runoff to the detention basin.

Scope

The scope of this phase of the project is:

- Meet with the City Project Engineer and other City personnel as necessary to develop the alternative;
- Perform the necessary field surveys and geotechnical investigations;
- Identify permitting and approval agencies;
- Identify utility conflicts;
- Identify real estate and easement acquisition needs;
- Provide an outline of the specifications;

- Provide a list of the plan sheets;
- Provide a preliminary opinion of probable construction cost;
- Provide schematic drawings and sheets as necessary for clarity of the report, and;
- Provide a set of construction documents complete to the 65% level for review by the COA.

Supporting Study and Design Documents

The following documents were utilized in producing this report:

- Menaul/Mildred Drainage Study, March 1991;
- AMDS,1981;
- FEMA maps;
- COA facility maps;
- COA as-built drawings; and
- New Mexico State Highway and Transportation Department (NMSHTD) as-builts.

II. COORDINATION WITH OTHER AGENCIES

Coordination with three agencies in the design process and in the construction phase of this project will be required. These agencies are:

- The NMSHTD,
- The Middle Rio Grande Conservancy District (MRGCD), and
- The State Engineers' Office.

Coordination with the NMSHTD will result from the proposed modifications to the CMPA culvert crossing under I-25, the realignment of the 60-inch discharge pipe from the Menaul storm drain system, and the construction of the proposed de-silting basins located between the east and west bound lanes of I-40 at University Boulevard.

No MRGCD facilities will be involved in the construction. However, we suggest that they have an opportunity to review the design analysis report since the discharge from the basin will continue to affect the Alameda Drain.

The State Engineer has authority over the design and construction of dams in the State of New Mexico. The criteria for the dam design is the "Summary of New Mexico State Engineer Office Procedure on Design Criteria and Safety of Dams". The design will need the State Engineer's review and approval prior to the construction permit approval. The design analysis and a set of plans will be submitted to the State Engineer by S-SEC for their review and comment.

Each of these agencies has been contacted and informed by S-SEC regarding the impact this project will have on their facilities. Each agency will be provided a copy of the analysis report for their review and comment.

III. HYDROLOGIC ANALYSIS

S-SEC reviewed the Menaul/Mildred Drainage Improvements study dated March, 1991 for:

- Assumptions utilized in the study;
- Runoff routing through the basins;
- Basin layout and land usage; this includes a review of the latest AMDS maps and site visits by the project team, and
- Technical method used for calculating the runoff.

As a result of the review, S-SEC determined that a reanalysis of the hydrology of the area was in order to determine the runoff rates and volumes. This decision was predicated by the discovery that two drainage basins were not included in the Menaul/Mildred study, and that some of the basin runoff routings did not coincide with conditions identified in field visits. The result is a basin and sub-basin routing which differ from the Menaul/Mildred report and which significantly affects the final runoff values. The major drainage basin layout is shown on Plate 1 in Appendix U.

The two basins which were not identified in the Menaul/Mildred Study were Basin 1A and Basin 3. Basin 1A is the highway department right-of-way of I-40, between the north edge of I-40 and the Princeton Detention Basin. Basin 3 consists of highway department right-of-way bounded by I-25, I-40, Menaul Boulevard, and Indian School Road.

The land uses identified in the Menaul/Mildred Study are in general agreement with the actual conditions which exist in the field. The sub-basins and their respective land use treatments were developed from observations made during field visits. These observations, in conjunction with the land treatments listed in Table 4, page 5 of the COA DPM, formed the basis for developing the runoff rates for the basins. In cases where the land use could be clearly identified by function, Table 5, page 6 of the DPM was used to establish the percent impervious area for the sub-basin.

After the land use was established, the time-to-peak for the runoff was calculated using the Upland Method equation from the DPM:

$$Tp = (0.6667) \times (L/3600 \times V)$$

where, Tp = Time-to-Peak, hours
L = Length of travel of the runoff, feet
V = Average velocity of the runoff, feet/second

The average velocity was calculated using Equation 13, Section 22 of the DPM:

$$V = K \times S^{0.5}$$

where, V = Velocity, feet/second
K = Conveyance factors
S = Basin slope, feet/feet

If the time-to-peak was less than 0.1333 hours, 0.1333 hours was used in the computer analysis. This determination is in accordance with the DPM. The sub-basins, areas, land-use treatments, and time-to-peak values are presented in Appendix A.

After the basin parameters were determined, the computer modeling program AHYMO991 was used to determine the 6-Hour,100-Year and 24-Hour, 100-Year storm runoff rates and volumes. The following rainfall data was input to the program:

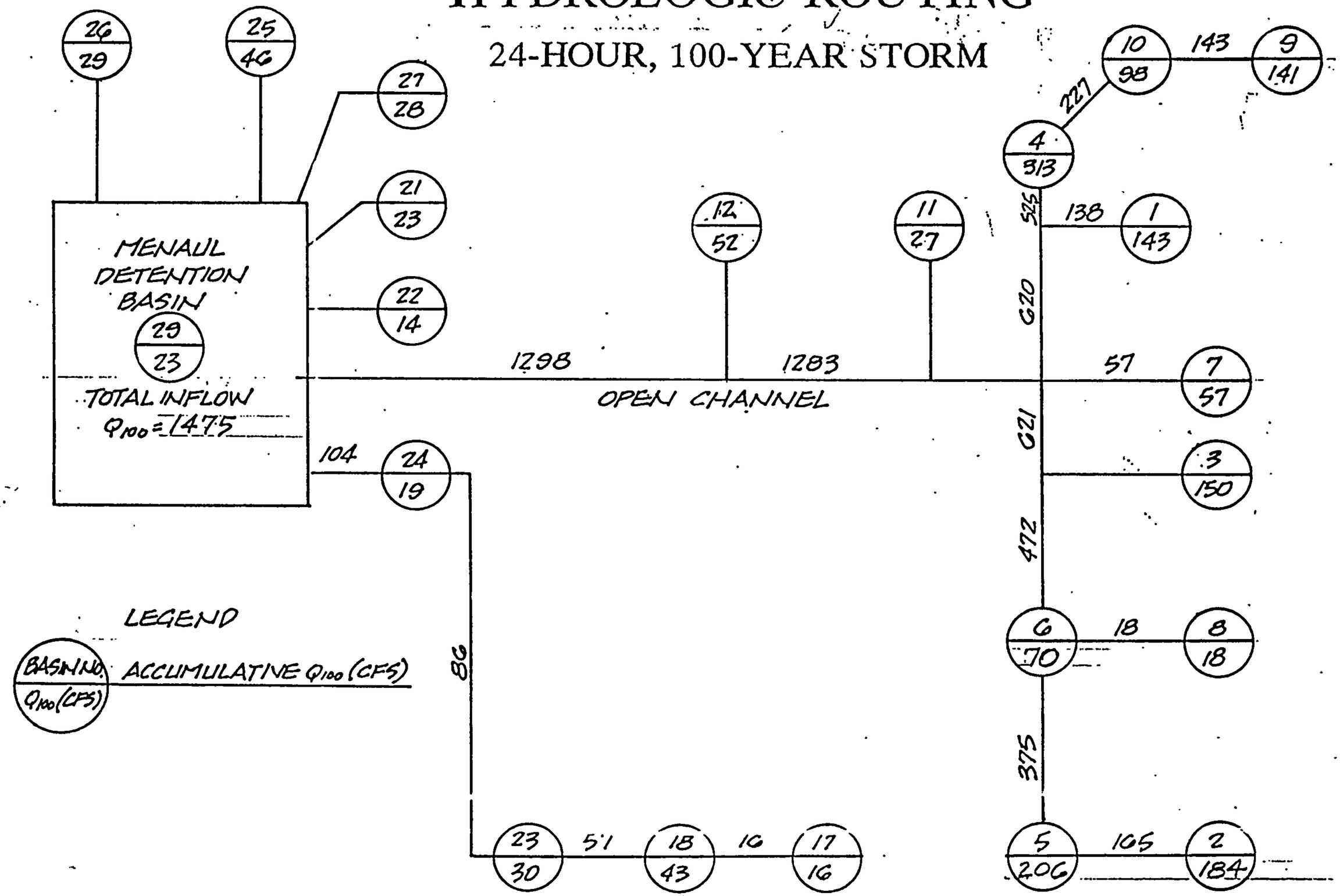
0.25-hour rainfall:	0.00 inches
1-hour rainfall:	2.01 inches
6-hour rainfall:	2.35 inches
24-hour rainfall:	2.75 inches

The time increment used to develop the unit hydrograph was 0.0333 hours for the 6-Hour, 100-Year storm and 0.0500 hours for the 24-Hour, 100-Year storm. These values are recommended in the DPM.

The 6-Hour, 100-Year runoff results were used to size the storm drain system. The 24-Hour, 100-Year results were used to determine the necessary storage capacity of the detention basin. Figure 1 on page 6 depicts the routing of the 24-hour, 100-year storm runoff through the basins. The peak flow rate is 1475 cfs at 1.600 hours and the volume of runoff is 86.04 acre-feet.

HYDROLOGIC ROUTING

24-HOUR, 100-YEAR STORM



LEGEND

BASIN NO. / Q100 (CFS) — ACCUMULATIVE Q100 (CFS)

FIGURE 1

A comparison of the runoff values for the basins from this study to the results of the March 1991 Menaul/Mildred Study indicate a reasonable correlation between specific basins. The difference in the total runoff rate and volume is a result of additional basins and the different runoff routings in the basins.

The one-half Probable Maximum Flood (PMF) was determined using the PMF flow rate data generated using the AHYMO991 program. The data for the rainfall volumes at each time interval was determined using the procedures outlined in the DPM. No elevation adjustment or depth-area reduction was needed for the area. The Probable Maximum Precipitation (PMP) rainfall amounts calculated were:

- PMP,360 (6-Hours) = 15.60 inches (from Plate I, DPM)
- PMP,60 (1-Hours) = 11.72 inches (from Plate H, DPM)
- PMP,15 (1/4 Hrs) = 7.97 inches (PMP,15 = 0.68 x PMP, 60)

After the hydrograph from the PMF analysis was generated using AHYMO991, the 1/2-PMF data was determined by using the "DIVIDE HYD" function in the program with a 50 percent split of the PMF data. The peak flow rate for the 1/2-PMF event is 6998 cfs at 2.300 hours and the volume of runoff is 313 acre-feet.

Because of the volume of the results, a copy of the AHYMO input and output for the 24-Hour, 100 Year storm and the 1/2-PMF storm has been provided to the COA on a floppy disk. A copy of the final hydrograph printout for each scenario is provided in Appendix P and Q. The inflow-outflow hydrographs for the basin are presented on Plate 5 in Appendix U.

IV. STORM DRAIN SYSTEM

Introduction

The purpose of this study was to develop an alternative storm drain system to the proposed system outlined in the AMDS. This alternative would convey storm runoff to the open channel west of I-25 and would minimize construction on Menaul Boulevard.

The proposed alternative described in this report would intercept flows at the intersection of Princeton Drive and Phoenix Avenue north of Menaul Boulevard. These flows would be conveyed to the open channel, west of I-25, through a new parallel 72-inch RCP. This interception would reduce the flows which currently reach the Menaul/University Boulevard intersection. The Menaul/University system would then be able to intercept more flows from University Boulevard which currently flood the area. This flooding is described in the AMDS and in the FEMA maps. The modifications to the existing Menaul/University system will be a new system from the intersection of University and Menaul Road south on University, a new line from the intersection of University and Menaul Road west on Menaul Road approximately 650 feet, and then an extension north to the existing storm drain system in Menaul Boulevard. This approach would result in substantial cost savings by limiting construction on Menaul Boulevard.

Objectives

The objective of the analysis was to:

- Determine the existing system's capacities;
- Minimize the modifications to the existing system;
- Minimize the need for additional easements;
- Minimize the construction on Menaul Boulevard;
- Maximize the interception of the storm runoff, and;
- Provide for future extensions of the storm sewer system at Princeton Drive north to Candelaria.

Method of Analysis

The existing system was set up in the computer model using location information from the COA facility maps, as-built information and field survey data. For each scenario, flow rates to the system were taken from the output of the AHYMO991 hydrologic program.

The computer program used for the hydraulic analysis utilizes the standard step method for calculating the hydraulic grade line (HGL) by balancing the energy equation for flow through the system. Headlosses (junction losses) are added to the upstream end of the HGL for each pipe. Junction loss coefficients were input to the program. The headlosses are calculated by multiplying the velocity head of the pipe flow times a junction loss coefficient. The analysis provides the following information:

- Hydraulic Grade Line (HGL) elevations and plots;
- Energy Grade Line (EGL) elevations;
- Existing flow rates, cfs;
- Full flow capacity, cfs;
- Flow velocity, feet/second (upstream and downstream); and
- Headloss, feet.

The analysis of the system was accomplished by assuming that the flows reached the inlets of a system at the same time. An HGL calculation was performed, and if the HGL was above ground level, the flows to the inlets were reduced. The reduction in flow rates was a random increment (e.g., 10% reduction). The HGL was then recalculated. These iterations continued until the HGL elevation was below the ground level at the inlets and manholes. The resulting flow rate through the system was the rate which was used for comparison of the existing system flow rates to the proposed system flow rates.

The junction loss coefficients at the manholes were determined using graphs from the City of Denver Drainage criteria Manual. Copies of the graphs which were used are provided in Appendix B. The loss coefficient used for the catch basins was 0.7 for pressure flow. This coefficient assumes the catch basin and connector pipe will act similar to a culvert with headwater. The coefficient for the catch basins without pressure flow was 0.08.

Pipes flowing with a free surface were considered to have an entrance loss coefficient of 0.08 at the manholes. All manholes were assumed to have three-quarter benching as shown in the manhole detail in the DPM.

The inlet capacities were determined using the graphs contained in Section 22.3 of the COA DPM.

Existing Storm Drain Analysis

The existing system is shown on Plate 2 in Appendix U. The system was divided into two systems, A and B, for the analysis. A is the system which terminates at the 90-inch outfall into the junction box (L1-A) just east of I-25 and extends east to the intersection of Princeton and Phoenix. This system branches three directions at this junction. One branch extends to the north on Princeton and terminates at the intersection of Princeton and Claremont. The second extends east on Phoenix and terminates at the intersection of Phoenix and Girard. The third branch extends to the south on Princeton to the intersection of Princeton and Menaul.

System B terminates at the 60-inch outfall to the open channel just north of the Country Club Inn Motel. This system extends to the south and branches in two directions at a manhole located near the intersection of Menaul and the west frontage road for I-25. One branch extends east on Menaul Boulevard and then branches south on University Boulevard to Menaul Road. The second branch extends south into the highway department right-of-way and drains the Big "I" interchange.

System A

The computer analysis of System A showed that the system has a capacity of 418 cfs pressure flow without surcharging the system. The major cause of the limited condition is the "choke" at Lines 11 and 12 just west of the intersection of Princeton and Phoenix. These lines have slopes of 0.0064 and 0.0061 foot-per-foot. The full flow capacity of these pipes is 293 and 288 cfs, respectively. The hydrologic analysis indicated that approximately 680 cfs will reach the intersection of Princeton and Phoenix, either overland or through pipe flow. This will result in localized flooding and runoff moving onto Menaul Boulevard. The printout of the System A analysis is in Appendix C.

The branch extending north on Princeton has a capacity of 90 cfs. The flows which travel through Princeton are from Basin 9 (Candelaria Boulevard) and Basin 10 combined. These flows total 227 cfs. Catch basin capacity on the Princeton branch is 161 cfs.

The Phoenix branch has a capacity of 87 cfs, while total runoff from contributing basins is 313 cfs. Total catch basin capacity of the Phoenix system is 146 cfs. These values indicate that there is both insufficient catch basin capacity as well as inadequate storm drain capacity.

The branch which extends south to Menaul Boulevard does not have adequate capacity to carry the flows which reach the system. The limiting factor is the 48" × 76" arch pipe section (Line 15) just south of the junction box at the Princeton/Phoenix intersection. The slope of this pipe is 0.00249 foot per foot and the full flow capacity is 113 cfs. As a result, excess flows will travel west on Menaul Boulevard and contribute to the flooding in this area. The excess flows are estimated to be 63 cfs for the current system.

System B

System B is located in Menaul Boulevard, part of University Boulevard, and north of Menaul on the west frontage road for I-25. The capacity of the existing system was determined by distributing the runoff evenly to the catch basins. An HGL calculation was performed and, if the HGL was above the ground level, the flow rates were lowered, and the HGL was recalculated. The analysis indicated a system capacity of 236 cfs. This includes 75 cfs from the highway department right-of-way (Basin 3). Line 13 in the analysis is a 54-inch pipe with a full flow capacity of only 108 cfs. The slope of this pipe is 0.004 foot-per-foot. A second pipe, Line 7, is capable of 120 cfs, full flow. These two sections limit the overall capacity of the system. Catch basin capacity of the system is estimated to be 380 cfs. The analysis of System B is presented in Appendix D.

Proposed Improvements

System A

The proposed improvements to System A, Plate 3 in Appendix U, are, principally from the intersection of Princeton Drive and Phoenix Avenue to the J-Box (L1-A) just east of I-25. These improvements will allow the existing branch systems on Princeton, Phoenix, and Menaul/Princeton to operate more efficiently, and to intercept the runoff. Flows not intercepted at the existing catch basins on Princeton and Phoenix will be intercepted by "cattle guard" drains. One "cattle guard" drain will be placed transverse on Princeton just north of the intersection of Princeton and Phoenix, and will tie into the Princeton branch. The second drain will be placed transverse on Phoenix just east of the intersection, and will tie into the Phoenix branch. The pipe sizes from the "cattle guard" drains to the existing J-box will be increased to accommodate the higher flow rates into the systems. The printout of the Princeton branch analysis and the Phoenix branch analysis are presented in Appendix E and F, respectively. There will not be any improvements to the existing Menaul/Princeton branch.

The improvements from J-box L1-D at the Princeton/Phoenix intersection to J-box L1-A at the east side of I-25 will consist of a new 72-inch storm drain line which will parallel the existing 72-inch storm drain on the north side. The existing 72-inch line will tie into the new J-box. The new storm drain line parallels the existing 72-inch line on the north side because an automobile repair service building is constructed on the south drainage easement line. Any excavation near the building will require extraordinary shoring systems and would cut off the owner's access to the service bays.

The new J-box L1-C east of University Boulevard will transition the new 72-inch parallel line from the north side of the existing 72-inch line to the south side of the existing line. The new parallel line from J-box L1-C west to a new J-box L1-B just east of the existing CMPA will be a 72-inch diameter line. Manholes will be constructed at intervals along the length of this pipe as specified in the DPM. The existing 72-inch line and the new 72-inch line will merge into the new J-box L1-B and a new single 90-inch line will tie J-box L1-B

into the existing J-box L1-A at I-25. Appendix G contains the results of the hydraulic analysis of this system.

The new 72-inch parallel line proposed for the Union 76 Truck stop parking lot is on the south side of the existing 72-inch line because the restaurant is located near the north drainage easement. Again, any excavations next to this building would require extraordinary shoring. Additionally, foot traffic to the restaurant would be adversely impacted because of close proximity of the excavations to the main entry of the restaurant.

The last leg of the proposed system is a single 90-inch line. This line was proposed so that the flows into J-box L1-A would align directly into the 10' × 16' CMPA culvert. This layout should reduce any headloss in the existing J-box and will result in minimal modifications to the existing J-box.

The proposed parallel routing of the storm sewer line will allow the existing 72-inch line to remain functional throughout the construction of the new system. The increased capacity provided by the parallel system will allow extension of the Princeton system north to Candelaria Boulevard, at a later date, in order meet the one lane dry criteria on Candelaria. Also, a majority of the construction work will take place on vacant land, resulting in minimal pavement cuts, and thereby reducing construction costs and traffic control.

However, this proposed routing will require acquisition of some additional easements along the route. The extent of these additional easements is shown on Plate 3 in Appendix U. This routing will also require a detailed construction sequencing plan for the improvements which will occur in the Union 76 truck stop. This business is subject to extensive tractor-trailer traffic and disturbance to the parking area and the traffic movement on the site will need to be minimized.

System B

The main objective of the improvement to this system was to increase the capacity of the system while minimizing construction on Menaul Boulevard. Therefore, the improvements

to System B are mainly confined to University Boulevard and Menaul Road. There will be one perpendicular cut across Menaul Boulevard to tie the new system into the existing system in Menaul Boulevard at the new J-box (L4-A). The proposed improvements are shown on Plate 3 in Appendix U.

The new University system will be a 48-inch diameter pipe through the reach and will have catch basins placed on each side to intercept flows from the basin south of University Boulevard. The full flow capacity of the new 48-inch pipe is approximately 100 cfs. The new catch basins are proposed to be Type A catch basins with a total estimated interception capacity of 210 cfs (sump condition). A 24-inch diameter storm drain will extend west into the Hilton Hotel parking lot at the south entry. This will drain the flows which are collected on that site and convey them to the main trunk line on University.

The NMSHTD right-of-way between the east and westbound lanes of I-40, and adjacent to University Boulevard will be used as small detention basins. These basins will provide some relief to the flooding on University Boulevard, as well as perform as desilting basins. Rundowns will be constructed off of University Boulevard to these basins and stand pipes, which are connected to the new University system, will drain these areas.

The University Boulevard system will continue to the west from the intersection of Menaul Road and University Boulevard. This drain will be a 48-inch pipe and will be approximately 650 feet long. From the west end of this line, the 48-inch drain will transition to two parallel 30-inch drains which continue north and will intersect with the existing 54-inch line in Menaul Boulevard. The parallel 30-inch lines are necessary to clear the existing utilities in Menaul Boulevard. At this point, a new J-Box (L4-A) will be constructed. The existing 54-inch drain will be cut at this point and reconnected into J-box L4-A. A new 72-inch line will tie into the north side of J-box L4-A, will continue in a northwest direction, and will tie into the existing 10' × 16' CMPA culvert. J-box L4-A will allow the system to equalize and flows can either discharge north through the new 72-inch drain, or west through the existing 54-inch drain.

The hydraulic analysis shows the improved system is capable of removing 340 cfs flow from University and Menaul Boulevards, not including the flows from the NMSHTD right-of-way. Approximately 200 cfs will be intercepted by the catch basins on Menaul Boulevard. The peak flows to this area are approximately 400 cfs, therefore, approximately 60 cfs will pond or move west on Menaul Boulevard to the intersection of Menaul and Edith. The proposed improvements will result in about a 100 percent increase in capacity over the existing system. The hydraulic analysis of System B is contained in Appendix H and J.

The improvements which are proposed for this phase will require some additional easement acquisition which are shown on Plate 3 in Appendix U. As in the construction for System A, a detailed construction sequencing will be necessary to accomplish the installation of the new 72-inch line without considerably disrupting the business of the truck stop at University and Menaul. System B will require a cut on Menaul Boulevard to construct the new J-box. This construction will require minimal traffic control on Menaul Boulevard. The construction of the extension south on University Boulevard will also require traffic control and the major concern at this location will be continued access to the Hilton Hotel.

The improvements to the existing Menaul/University Boulevard system can be constructed without disrupting the existing system. This proposal will allow the area to drain during the construction of the improved system.

V. OPEN CHANNEL SYSTEM

Objective:

The objectives of the improvements to the channel are as follows:

- Provide for conveyance of the total flows which reach the channel;
- Maintain average velocities in the channel of three to four feet per second;
- Maintain 3:1 side slopes whenever possible;
- Maintain freeboard;
- Maintain accessibility to the channel for maintenance;
- Provide a new crossing at Broadbent Parkway with adequate capacity;

- Provide a structure at the outfall of the channel into the detention basin;
- Keep improvements within the existing 80-foot drainage and 20-foot water easement; and
- Minimize construction cost.

Method of Analysis

The analysis of the improvements to the channel was accomplished using the Water Surface Profile program from the Hydrology software. This program performs a backwater surface profile of the channel given the channel geometry and a starting water surface elevation at the outfall. The computational method is similar to the U.S. Bureau of Reclamation "Method B", in which an energy balance of the system is performed from one section to another. The summary tables of the computer analysis are provided in Appendix K.

Existing System Description

The head of the channel starts at the outfall of the 10-foot CMPA culvert on the west side of I-25. The channel extends west and crosses under Broadbent Parkway. This crossing is an 84-inch diameter RCP culvert. The channel continues west until it outfalls into the existing Menaul Detention Basin, located just northeast of the Menaul School. The channel is bounded on the north by the Broadbent Business Park and on the south by the Country Club Inn Motel. The land west of the motel and adjacent to the channel is vacant land.

The length of the channel is approximately 1,400 feet. The bottom width of the channel is an average of 10 feet. The side slopes vary from 1:1 to 3:1 along the length. Depth of the channel ranges from approximately 15 feet at the head to approximately 7 feet at the outfall. The channel is contained in an 80-foot drainage easement along the length. A 20-foot wide water line easement exists along the south side of the channel between the detention basin and Country Club Inn Motel.

The existing channel is unimproved along the total length. There is considerable vegetative growth which ranges from small weeds to trees. There is a considerable quantity of rubble which has been dumped near the outfall of the channel into the detention basin.

The grades on each side of the channel are approximately equal elevation (5025) until the Broadbent Parkway crossing. After this point, the north side of the channel remains at approximately elevation 5025. The south side drops from elevation 5023 at Broadbent Parkway to 5013 at the outfall of the channel. The flowline elevation at the outfall is about elevation 5007.

The 84-inch culvert at the Broadbent Parkway crossing was constructed when the Broadbent Business Park was built. The full flow capacity of the culvert is approximately 550 cfs. The total length of the culvert is 120 feet. There are no headwalls at the culvert, which has resulted in some piping. The City has temporarily repaired this problem by placing concrete around the head of the culvert.

Flows are received at the head of the channel from the 10-foot × 16-foot CMPA culvert which runs under I-25, and from the 60-inch outfall pipe of the Menaul Boulevard system. There are four rundowns into the channel on the north side between Broadbent Parkway and the frontage road, two catch basins at the Broadbent Parkway crossing, connected into the 84-inch culvert, and a corrugated metal pipe (CMP) near the outfall. This CMP drains a catch basin near the Galles Racing building. The channel does not appear to receive any overland flows from the vacant land on the south side.

Proposed Improvements

The channel will be regraded to the dimensions necessary to covey the maximum flow rate of 1300 cfs. The side slopes of the channel are maintained at 3:1 throughout the reach on the south side. The north side slopes vary from 2:1 to 3:1. This grading was necessary to maintain the required area for flow while not requiring a retaining wall along the north side. The average slope of the channel will be approximately 0.001 foot-per-foot to keep velocities

low. There will be localized velocity increases around the drop structures but these appurtenances will be protected upstream and downstream with rip-rap aprons.

The upstream face of the channel at the CMPA culvert will be slope-paved with concrete. The channel just beyond the CMPA culvert outlet will be a rip-rap stilling basin approximately 70 feet in length. This will reduce the flow velocity and mitigate formation of a scour hole at the culvert outlet. An additional rip-rap apron will be constructed at the outfall of the 60-inch drain from the Menaul storm drain system. A drop structure will be constructed at about the middle of the east reach of the channel. The structure will be concrete with rip-rap upstream and downstream to reduce erosion.

At the Broadbent Parkway crossing, the existing culvert will be removed. A 90-degree headwall will be constructed at the entry point of the new crossing. This design assists in reducing the velocity of the flow in the channel. The channel will be paved with concrete 10 feet prior to the crossing. The new crossing will consist of six 8-foot high by 16-foot wide precast arches. Information sheets for these arches are presented in Appendix L. The cross-sectional flow area provided by this structure was necessary to prevent excessive exit velocities from the crossing into the west reach of the channel. The outfall of the crossing will have a headwall and the channel is concrete lined to a distance 10 feet downstream from the culvert. This section will be followed by a 10 foot section of rip-rap for erosion protection in the channel.

The channel will be sloped at approximately 0.001 foot-per-foot from the crossing to the channel outfall. The entry apron to the channel outfall will be 10 feet in length and will terminate at a 90-degree headwall entry into the lined channel. This 90-degree headwall is needed to produce the backwater effect in the upstream channel which maintains the low channel velocity.

The outfall will be a rectangular concrete channel 17 feet wide at the entry and which will flare to a width of 26 feet for entry into a baffled apron outfall structure. The concrete channel will be ten feet deep.

A 15-foot maintenance road will be constructed in the water line easement, located on the south side of the channel between Broadbent Parkway and the detention basin.

The following is a synopsis of the properties of the improved channel:

- Bottom width = 22 feet
- Side slopes = 3:1 (south side), north side varies
- Flow rate = 1300 cfs
- Average velocity = 4.1 feet per second west of the crossing
  = 4.8 feet per second east of the crossing
- Maximum velocity = 12.7 feet per second (at the outfall of the 10-foot CMPA).
- Slope of earthen sections = 0.001 foot-per-foot (average)
- Flow is subcritical through reach

The proposed channel geometry can be constructed in the existing easement, except for the reach from the outfall of the CMPA to the west end of the Broadbent Business Park parking area. This section will require a construction easement to allow for some grading on what will be the upper banks of the channel. This area is currently not being utilized for any purpose.

The maximum elevation drop through the reach of the channel was controlled by the sidebank elevations and by the existing 48-inch sanitary sewer main at the entrance of the channel into the basin. The crown of this pipe is at approximately elevation 4999. The elevation of the flow line of the outfall structure at this point will be approximately elevation 5002.

VI. CHANNEL OUTFALL STRUCTURE

Objective

The objective of this design was to provide an outfall structure which would minimize the scouring of the basin bottom by reducing the approach velocity of the maximum flows.

Existing Outfall Description

The existing outfall of the channel is a concrete rundown which was placed without forms or finishing. The outfall terminates at the unprotected bottom of the basin and flows are forming a scour hole at the terminus.

Proposed Improvement

The proposed outfall structure is a baffled apron spillway energy dissipating structure. This structure is referred to as a Basin IX by the Bureau of Reclamation. This design was selected because it will provide the reduction in velocity necessary to prevent excessive scour in the basin. Also, this structure has performed satisfactorily in previous applications both in the City of Albuquerque.

The structure was designed using the Bureau of Reclamation Engineering Monograph No. 25, Hydraulic Design of Stilling Basins and Energy Dissipators. The design information used to size the structure was:

- Design flow rate = 1300 cfs
- Flow line elevation at crest of structure = 5008.50
- Flow line elevation at bottom of structure = 4980.00
- Slope of apron = 2:1
- Unit discharge of the structure was set at 50 cfs/foot width

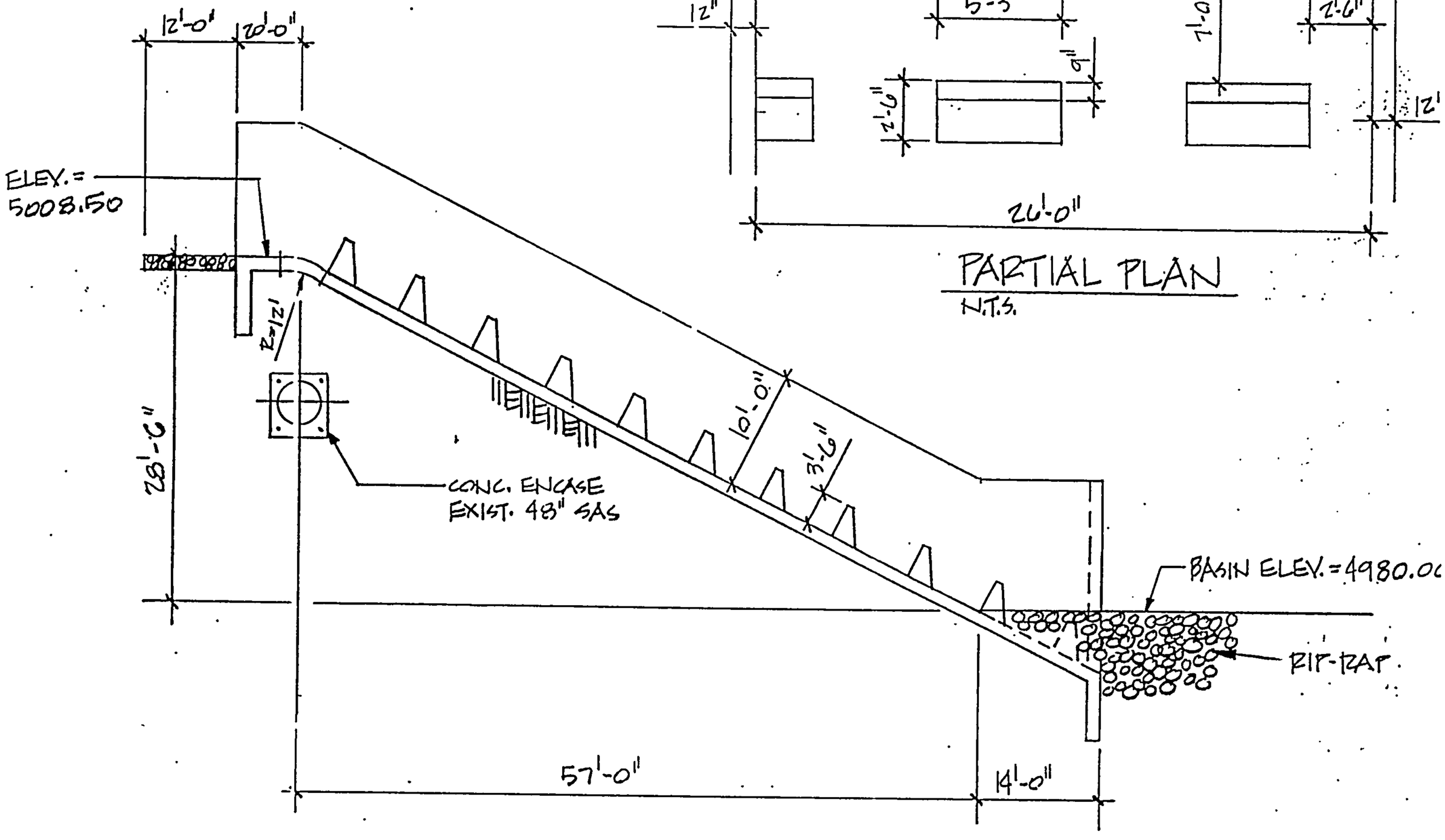
The resulting width of the structure is 26 feet. From the graphs in the design guide, the recommended approach velocity is 6.7 fps. Using this information, the dimensions of the structure and the baffle blocks were determined. The calculations are contained in Appendix M. A sketch of the structure is provided on page 22. The dimensions are as follows:

- Width = 26 feet
- Slope = 2:1
- Overall length = 73 feet
- Height of training walls = 10.5 feet
- Baffle width and spacing = 5.25 feet
- Baffle depth = 3.6 feet
- Distance between faces = 7 feet
- Terminal velocity = Approximately 4 fps

Rip-rap will be placed at the base of the structure to reduce scouring. Also, ninety degree wingwalls will be constructed at the base of the spillway. The low flow swale for the detention basin will be connected to the end of the apron and will terminate at the basin principal spillway, a 54-inch pipe.

BAFFLE APRON DROP STRUCTURE SECTION

N.T.S.



PARTIAL PLAN

N.T.S.

FIGURE 2

VII. BASIN AND DAM

Objectives

The objectives of the basin and dam design are to provide:

- Safe storage of the 24-Hour, 100-Year storm runoff;
- Controlled discharge of 36 cfs, maximum to the Alameda Drain; and
- Provide controlled discharge of the 1/2-PMF event through an emergency spillway.

Existing Basin Description

The existing basin was reported by Sergent, Hauskins and Beckwith (SHB) to have been constructed in the early sixties after the Menaul Dam was abandoned and leveled. The basin has not been maintained on a regular basis and, consequently, there is considerable shrub and tree growth in the area. The basin is not fenced, resulting in illegal dumping of trash and construction rubble throughout the basin. This dumping has caused clogging of the outfall pipes and has resulted in flooding of the residential areas downstream of the basin.

The side slopes vary, but are covered with native grasses which help to stabilize the soil. There are areas on the slopes which are subject to severe erosion from storm drain outfalls and rundowns. The top-of-slope elevations vary but are approximately 5020 on the east side and 4992 on the west end near the outfall. The outfall pipe (54-inch RCP) has an entry invert of 4077.

Geotechnical Field Investigation

Preliminary and final geotechnical investigations were performed by SHB to provide information on subsurface conditions and to make recommendations for design and construction of the basin and embankment. A copy of the final report is provided in Appendix N. The items for which recommendations were made are as follows:

- Maximum embankment slopes based on data from a slope stability analysis;
- Key trench dimensions;
- Soil properties for the embankment fill;
- Compaction requirements for embankments; and
- Compaction requirements for concrete structures.

Three analyses were performed to develop the embankment recommendations. These analyses were: 1) slope stability analysis; 2) settlement analysis, and; 3) seepage analysis. The following paragraphs contain a synopsis of the findings. A more detailed description is contained in the preliminary geotechnical investigation report.

The program used for the slope stability analysis was PCSTABL5. This utilizes the modified Bishop method of analysis. The results of the analysis indicate that the dam embankment will have the following safety factors against failure:

- Steady state conditions: Factor of Safety = 2.27
- Earthquake conditions: Factor of Safety = 1.71

The State Engineers' criteria for a safety factor is a minimum of 1.50 for steady state conditions. The computed factor of safety computed complies with this requirement.

A settlement analysis was performed on the proposed embankment. The results of this analysis indicate that the approximate settlements of the foundation soils will be two inches. The subsurface soils are relatively uniform and, therefore, these settlements are anticipated to be uniform and no cracking of the embankment should take place.

The seepage analysis indicates a coefficient of permeability of approximately 300 feet/year. The estimated horizontal and vertical wetted distance for a retention time of 50 hours would be approximately two feet.

Proposed Improvements

The investigation indicated that the existing soils which are predominantly classified as SM with some SP could be used for a homogenous embankment. There is considerable rubble on the north side of the basin area to a depth of about 15 feet. This material should be wasted or, if used, it should be screened to remove the debris. Continued quality control will need to occur during construction to verify cleanliness of fill and proper gradation.

Prior to construction of the embankment, the area to receive fill will be prewetted to a depth of approximately 5 feet. The extent of the prewetting should be verified by the geotechnical engineer prior to start of the embankment. The dam embankment will be constructed at a slope of 3:1 on the upstream and downstream face. The downstream slope will extend into the embankment on the east side of the lateral. The crest of the embankment will be 15 feet wide at an elevation of 5006.00. This elevation is controlled by the topography of the existing wall of the storage area in close proximity of the dam. The key trench will be constructed on the dam centerline and will be 20 feet wide at the bottom with side slopes of 2:1. The depth will be a minimum of five feet below the bottom of the embankment. The embankment material will be compacted to 95% of maximum density as determined by the ASTM D698 procedure.

The side slopes of the basin will be constructed at a slope of 2.5:1. This side slope is necessary to provide the required storage capacity. These side embankments will be constructed in accordance with the recommendations for the dam embankment where fill is required. In areas of cut, the top one foot of the face of the slope will be compacted to 95% of ASTM D-698 to help mitigate gullying and localized erosion. An access road will be constructed along the north perimeter of the basin. This road will be designed and constructed in accordance with the standard detail presented in the DPM. A perimeter fence will be built along the property line with an access gate at the road entry along the Alameda Lateral.

A low-flow swale will be constructed from the base of the channel drop structure to the trash rack structure at the base of the dam. The low-flow channel will be 12 feet wide and

will serve as an access road in the basin for maintenance personnel. The trash rack structure will have wingwalls to prevent erosion of the embankment near the sides of the structure. The principle spillway will consist of a 54-inch RCP with an 18-inch orifice plate bolted to the entrance to the pipe. This orifice plate will limit discharge to approximately 33 cfs for the 100-Year storm. A detention basin layout plan is presented on Plate 4 in Appendix U, and a cross section of the dam is shown on sheet 9 of the plans accompanying this report.

Basin Hydraulics: 24-Hour, 100-Year Storm

The basin hydraulics were calculated using the flow volumes produced from the AHYMO991 computer program for the 24-Hour, 100-Year rainfall data. This analysis routed the flows through the basin using the "ROUTE HYD" function. Stage-storage-discharge data for the basin was input to the program and the analysis provided time-stage-storage-discharge data for the total hydrograph to the basin. The stage-discharge-storage graph is presented on Plate 6 in Appendix U. The results of the analysis are as follows:

- Total inflow volume = 86.04 acre-feet
- Maximum inflow rate = 1475 cfs
- Maximum storage needed = 65.87 acre-feet
- Maximum outflow rate of the principle spillway = 33 cfs; and
- Maximum elevation of water in basin = 4995.75 feet

The elevation of the emergency spillway was set at 4998.00 which will provide a 2.25 foot freeboard to the emergency spillway for the design storm. Inflow-outflow calculations show that the basin will empty in approximately 47 hours. This time violates the DPM requirement that detention basins drain in less than 24 hours. The time violation results from the requirement that the outflow from the basin to the Alameda Drain not exceed 36 cfs. Therefore, this design will require a variance from the DPM requirement.

Basin Hydraulics: One-Half Probable Maximum Flood (1/2-PMF) Analysis

The area which was considered to contribute to the one-half PMF was the same basin areas as are shown on Plate 1. This analysis assumes that the Princeton Detention Basin will remain in operation during a one-half PMF storm and will continue to discharge into the North Diversion Channel.

The analysis assumes that flows which are not intercepted by the storm drain will move west on Menaul Boulevard. The excess flows are "channelled" to the basin on Broadbent Parkway and are added to flows which reach the basin through the conveyance system. This approach is conservative in that runoff which does not get intercepted by the storm sewer will collect at the intersection of Edith and Menaul Boulevard downstream from the basin.

An Inflow-Elevation-Volume-Outflow table was produced to determine the maximum flow rate for which the emergency overflow spillway (weir) would need to be designed in order to convey the 1/2-PMF flows while maintaining two feet minimum freeboard. The calculations indicate that an 80-foot wide spillway flowing a maximum of 5.58 feet deep would be needed to discharge a maximum of 3934 cfs and maintain a maximum water surface elevation of 5003.58. This design would provide a 2.5 foot freeboard to the crest of the dam and the volume of the basin at this water surface elevation would be 108.82 acre-feet. The proposed weir geometry is a trapezoidal shape with a top width of 0.67 feet. The slope of the upstream and downstream faces will be 2:1. The capacity of the weir was calculated using the following equation from King & Brater, 6th Ed:

$$Q = C \times L \times H^{1.5}$$

where, Q = Flow rate, cfs;

C = Weir coefficient (varies with depth of head);

L = Length of weir, feet; and

H = Head, feet.

The weir coefficients were taken from the values in King & Brater, 6th Ed, Table 5-9, Chapter 5.

The apron downstream from the weir is set at a slope of 0.00189 foot-per-foot, is 80 feet in length, and has a one foot high sill at the end with slots to accommodate passage of low flows.

Principle Spillway

The principle spillway will be a 54-inch RCP with an 18-inch diameter orifice plate attached to the pipe entrance. This pipe will be an extension of the existing 54-inch outfall pipe and will have anti-seepage collars set at approximately 15 foot intervals. These collars will be approximately 9 feet by 9 feet by 0.5 feet thick. The 54-inch pipe terminates at an existing J-box located just east of Broadway Boulevard. Three parallel 48-inch RCP's exit this J-box and outfall to the Alameda Drain at Second Street. The Broadway J-box will be modified so the principle spillway pipe from the detention basin discharges to the middle and north 48-inch pipe. The south 48-inch pipe will be plugged at this point, and the outfall for the Claremont detention basin will discharge to this pipe downstream. The Alameda Lateral will discharge to the principle spillway at manhole S132 adjacent to the lateral.

VIII. ANALYSIS OF EXISTING PARALLEL 48-INCH STORM DRAIN SYSTEM

The existing system was hydraulically modelled using the hydrology software cited in the previous sections. The control for determining the maximum flow rate for the system was the existing invert of the connector pipe which drains the Alameda Lateral. This pipe ties into the manhole located at the upstream end of Pipe 13 in the analysis. The three parallel 48-inch pipes were modelled as a single pipe. The equivalent pipe is a single 83-inch diameter pipe with a Manning's value of 0.0185. The manholes located on Franciscan, Arno, and Commercial have 15-inch waterlines passing through them, with the invert at approximately eight inches above the floor of the manhole.

The junction loss coefficients used for the analysis were 0.12 for free surface flow, 0.3 at the manholes where the 15-inch water main passes through, and 1.24 at the J-box.

The results of the analysis showed that the maximum pressure flow rate which the system can take in at the existing dam outlet pipe is 301 cfs (gravity capacity of the 54-inch outfall pipe is 79 cfs). The head at the existing dam was ten feet. The model indicated that flows will move out of the system at the lateral inflow point prior to surcharging through the manholes. The analysis is contained in Appendix R.

S-SEC personnel conducted a site visit to Franciscan Street and Arno Street on September 3, 1991. A maintenance crew was cleaning out the manholes at this time. The objects which were removed from the manholes included a tire, a stump approximately 1.5 feet in diameter and 2 feet long, a 6 foot length of 4-inch diameter metal pipe, an eight-foot rug rolled to a diameter of about 1.5 feet, and a metal trash can. There were several rags and miscellaneous smaller items which had caught up on the larger items. A tree trunk about 1.5 feet in diameter and 6 feet long was also observed in the south manhole on Franciscan Street. A chainsaw was required to remove the tree trunk from the manhole.

These objects are a collection of trash which washed into the storm drain system at the existing detention basin outfall pipe. They are the most likely cause of the surcharging of the storm drain system in the past. The trash rack, in combination with the fencing of the

basin, which will prevent illegal dumping of trash into the basin, should alleviate this problem.

The geotechnical investigation included an effort to determine if any adverse effects occurred to the soils around the manholes where surcharging has taken place in the past. Samples from the soil borings had moisture contents which would be expected for the clayey, sandy material located at the sites. The blow counts around the manholes were two blows-per-foot. This is very low and indicates loose soils. The most plausible cause for the low blow counts is that the soils were not properly compacted when the manholes were constructed. If corrective action is considered, the geotechnical firm has recommended either removing and recompacting the soils around the manhole, or pressure grouting around them. By limiting the flows through the pipes in the system, and constructing the improvements, it may be less imperative that any action be taken as the system should not flow under pressure.

IX. PRELIMINARY OPINION OF PROBABLE CONSTRUCTION COST

The preliminary opinion of probable construction cost of the improvements proposed in this report is $ 2,549,910. This includes a ten percent contingency and taxes. As directed by the COA project manager, a portion of the unit prices listed in the opinion were developed from the contract prices for the Lomas Storm Drain project and the Geneivas Arroyo project. The unit prices which are marked with an asterisk were taken from the COA Estimated Unit Price listing for Spring 1990. The detailed breakdown of the estimate is provided in Appendix S.

X. EASEMENT ACQUISITION

The proposed improvements will require acquisition of easements from property owners in the area. The acquisitions for the parallel 72-inch pipes from Princeton/Phoenix to the 10-foot × 16-foot CMPA culvert are as follows:

Description	Area (square feet)
Menaul School Addition No. 1 (Union 76)	13,700
Parcel "3" Menaul Development Area	4,990
Parcel "4" Menaul Development Area	2,250
Tract A Menaul Development Area	4,375
Tract B-1-A Menaul Development Area	6,450

The acquisitions for the parallel 30-inch drains from Menaul Road to Menaul Boulevard are as follows:

Description	Area (square feet)
Menaul School Addition No.1 (Union 76)	2,800
6.5774 Acre Tract (FINA)	6,400
Industrial Subdivision	2,600

The total area required of easement acquisition is 43,565 square feet (1.0001 acres).

XI. PLAN SHEETS AND SPECIFICATIONS

A list of the anticipated plan sheets and the specifications for the project is presented in Appendix T. In addition to the standard COA Earthwork specifications, S-SEC will provide a supplemental earthwork section that pertains to construction and material requirements of the embankment. A preliminary draft of the special specifications is contained in the geotechnical report in Appendix N.

S-SEC

PROJECT MENAUL STORM DRAIN SHEET NO. 1 OF

SUBJECT HYDROLOGY PROJECT NO.

BY PJR DATE 7/30/91 CHECKED BY DATE 1/1

BASIN 1

SUB-BASIN	AREA, Mi^2	LAND USE A	B	C	D	T_p, hr (UPLAND)
1,1	0.01837	—	—	22	78	0.165
1,2	0.00617	—	—	10	90	0.051 *
1,3	0.00761	—	—	10	90	0.072 *
1,4	0.00093	—	—	10	90	0.026 *
1,5	0.00488	—	—	10	90	0.037 *
1,6	0.00761	—	—	18	72	0.061 *
1,7	0.00875	—	—	19	81	0.058 *
1,8	0.02181	—	100	—	—	0.225

* USED 0.1333 AS A MINIMUM

Smith-Scheuch Engineering Company
6400 Uptown Boulevard, N.E. - Suite 500E Albuquerque, New Mexico 87110
505/884-0700

CONTRACT CONTROL FORM

PRELIMINARY REVIEW

Contact Person Thuan
Phone No. 768-2505

H16-0109

Project # S244.01
C.I.P. #
New or Ext. #

Type of Agreement: Drainage Covenant

Description/Project: Far West Inns. Inc
Public Works/Div: Eng / Design
Developer: Far West Inns Inc

Contract Amount $ -0-
Contract Amount $

SIA Contract Period: ___ - ___
SW'S Contract Period: ___ - ___
Project Completed Date ___

DRAFT CONTRACT:

Recd by Legal: ___ Rejected/Returned to Dept: ___/___
Returned to Legal: ___/___ Approved: ___ Initials: ___

FINANCIAL GUARANTY:

Letter of Credit No. ___ Date ___ Attached: Yes ___ No ___ Initial ___
Other: Type ___ No. ___ Attached: Yes ___ No ___ Initial ___

FINAL CONTRACT REVIEW

APPROVALS REQUIRED:

	Date Delivered	Returned to Dept	Approved by	Approval Date
Purchasing:	___/___	___/___		
Asst. City Attorney:	___/___	___/___		
CIP:	___/___	___/___		
City Attorney:	___/___	___/___		
CAO:	___/___	___/___		
~~Other:~~ Hydrology	8-15-95 ITL	___/___	JPC	8-15-95
Council:	EC No: ___	Approved: ___		Date: ___

DISTRIBUTION:

	Date:	By:
User Department	___	___
Vendor	___	___
City Clerk	___	___
Treasury	___	___
___	___	___

Revised 7/93
Doc. 1930/1vb

D. Mark Goodwin & Associates, P.A.
Consulting Engineers and Surveyors
P.O. BOX 90606, ALBUQUERQUE, NM 87199
(505) 345-2010

August 8, 1995

Mr. John Curtin
Hydrology Division
City of Albuquerque
P.O. Box 1293
Albuquerque, NM 87103

Re: Shoney's Suites (H16/D109)

Dear Mr. Curtin:

This letter is per your request to explain the emergency spillway operation for the referenced project.

As the pond fills to its maximum water surface elevation of 95.72. The 100 year-10 day volume is retained within the pond. Since the rim of the pond is at 03.00, the pond will never overflow.

The outfall or spillway for the pond as shown on sheet 18 of the calculations shows that when the pond fills to the elevations at the asphalt curb across the channel, the 24" RCP will be full and the only discharge will be over the asphalt curb weir. The weir as shown can spill the required Q of 10.32 cfs, which is the 100 year value.

With this information, please issue a letter of approval for Building Permit. Your expedient assistance is greatly appreciated.

Very Truly Yours,

MARK GOODWIN & ASSOCIATES, P.A.

for Gregory J. Krenik, P.E.
Project Manager

GJK: smr

City of Albuquerque

P.O. BOX 1293 ALBUQUERQUE, NEW MEXICO 87103

July 28, 1995

Gregory J. Krenik, PE
Mark Goodwin & Assoc
P.O. Box 90606
Albuquerque, NM 87199

RE: GRADING & DRAINAGE PLAN FOR SHONEY'S SUITES (H16-D109)
RECEIVED JULY 18, 1995 FOR BUILDING PERMIT
ENGINEER'S STAMP DATED 7-10-95

Dear Mr. Krenik:

Based on the information included in the submittal referenced above, City Hydrology has the following comments:

City Legal will not accept a Drainage Covenant that has been modified. A clean form is attached. Execute the unmodified form and submit it to City Hydrology for processing.

Provide an emergency spillway to protect the berm on the south side of the pond from breaching. An alternative would be t o eliminate the berm and meet the existing grade at about elevation 5101.

If you have any questions about this project, You may contact me at 768-2727.

Sincerely,

John P. Curtin

John P. Curtin, PE
Civil Engineer/Hydrology

c: Andrew Garcia
Tom Morgan, ShoLodge, Inc, 600 W Main St, Gallatin, TN 37066

City of Albuquerque

P.O. BOX 1293 ALBUQUERQUE, NEW MEXICO 87103

August 9, 1995

Gregory J. Krenik, PE
Mark Goodwin & Assoc
P.O. Box 90606
Albuquerque, NM 87199

RE: GRADING & DRAINAGE PLAN FOR SHONEY'S SUITES (H16-D109)
RECEIVED AUGUST 7, 1995 FOR BUILDING PERMIT
ENGINEER'S STAMP DATED 8/7/95

Dear Mr. Krenik:

Based on the information included in the submittal referenced above, City Hydrology accepts the Grading & Drainage Plan for Building Permit.

Include a copy of the Grading & Drainage Plan, dated 8/7/95, in each set of construction documents that will be submitted to Code Administration for the Building Permit.

A separate permit is required for construction of private drainage facilities within the City right of way. A copy of this letter must be on hand when applying for the excavation permit.

Engineer's Certification of grading & drainage per DPM checklist must be approved before any Certificate of Occupancy will be released.

If you have any questions about this project, You may contact me at 768-2727.

Sincerely,

John P. Curtin

John P. Curtin, PE
Civil Engineer/Hydrology

c: Andrew Garcia
Arlene Portillo
Tom Morgan, ShoLodge, Inc, 600 W Main St, Gallatin, TN 37066

DRAINAGE INFORMATION SHEET

PROJECT TITLE: Shoney's Suites ZONE ATLAS/DRNG,FILE#: H-16/D109

DRB #: _____ EPC #: _____ WORK ORDER #: _____

LEGAL DESCRIPTION: Tract A, Sandia Foundation - AMAFCA Subdivision

CITY ADDRESS: _____

ENGINEERING FIRM: Mark Goodwin & Associates, PA CONTACT: Gregory J. Krenik, PE

ADDRESS: PO Box 90606 PHONE: 345-2010

OWNER: ShoLodge, Inc. CONTACT: Tom Morgan

ADDRESS: 600 W Main St, Gallatin, TN 37066 PHONE: _____

ARCHITECT: Lindsay, Ponder, Brayfield CONTACT: Mr. Brayfield

ADDRESS: 270 Langley Dr, Lawrenceville, GA 30245 PHONE: 404/963-8989

SURVEYOR: _____ CONTACT: _____

ADDRESS: _____ PHONE: _____

CONTRACTOR: _____ CONTACT: _____

ADDRESS: _____ PHONE: _____

TYPE OF SUBMITTAL:

_____ DRAINAGE REPORT
X DRAINAGE PLAN
_____ CONCEPTUAL GRADING & DRAINAGE PLAN
X GRADING PLAN
_____ EROSION CONTROL PLAN
_____ ENGINEER'S CERTIFICATION
X OTHER Drainage Covenant submitted to Theresa 8-8-95 JPC

PRE-DESIGN MEETING:

_____ YES
X NO
_____ COPY PROVIDED

CHECK TYPE OF APPROVAL SOUGHT:

_____ SKETCH PLAT APPROVAL
_____ PRELIMINARY PLAT APPROVAL
_____ S. DEV. PLAN FOR SUB'D APPROVAL
_____ S. DEV. PLAN FOR BLDG PERMIT APPROVAL
_____ SECTOR PLAN APPROVAL
_____ FINAL PLAT APPROVAL
_____ FOUNDATION PERMIT APPROVAL
X BUILDING PERMIT APPROVAL
_____ CERTIFICATION OF OCCUPANCY APPROVAL
_____ GRADING PERMIT APPROVAL
_____ PAVING PERMIT APPROVAL
_____ S.A.D. DRAINAGE REPORT
_____ DRAINAGE REQUIREMENTS
_____ OTHER _____ (Specify)

DATE SUBMITTED: 8/7/95

BY: _____
Gregory J. Krenik, PE

RECEIVED
AUG 7 1995
HYDROLOGY

Albuquerque
Metropolitan
Arroyo
Flood
Control
Authority

2600 PROSPECT N.E. ALBUQUERQUE N.M. 87107

TELEPHONE (505) 884-2215
FAX 884-0214

TO WHOM IT MAY CONCERN:

Please be advised that :

Name: SPARLING CONSTRUCTION, WORKING FOR SHONEY SUITES -

Address: 6619 CORONA NE, ALB NM 87113

Telephone: 821-1034 FAX 821-1537

Signature:

has permission from the Albuquerque Metropolitan Arroyo Flood Control Authority to: ENTER THE AMAFCA RIGHT OF WAY ADJACENT TO THE NORTH DIVERSION CHANNEL

for the purpose of: PLACING FILL UP TO ELEVATION 5103.0 & RECREATING RESERVOIR IN SUCH AREA.

This permission is valid starting AUGUST 7 1995 through SEPT 30, 1995.

The person whose signature appears above hereby agrees to indemnify and hold harmless the Authority, its officers, agents and employees from all claims for property damage, personal injury, or wrongful death arising from or connected with permission to access AMAFC property.

Permission granted by:

John Kelly
Field Engineer

Date: AUG 4, 1995

RECEIVED
AUG 7 1995
DIVISION

D. Mark Goodwin & Associates, P.A.
Consulting Engineers and Surveyors
P.O. BOX 90606, ALBUQUERQUE, NM 87199
(505) 345-2010

August 8, 1995

Mr. John Curtin
Hydrology Division
City of Albuquerque
P.O. Box 1293
Albuquerque, NM 87103

Re: Shoney's Suites (H16/D109)

Dear Mr. Curtin:

This letter is per your request to explain the emergency spillway operation for the referenced project.

As the pond fills to its maximum water surface elevation of 95.72. The 100 year-10 day volume is retained within the pond. Since the rim of the pond is at 03.00, the pond will never overflow.

The outfall or spillway for the pond as shown on sheet 18 of the calculations shows that when the pond fills to the elevations at the asphalt curb across the channel, the 24" RCP will be full and the only discharge will be over the asphalt curb weir. The weir as shown can spill the required Q of 10.32 cfs, which is the 100 year value.

With this information, please issue a letter of approval for Building Permit. Your expedient assistance is greatly appreciated.

Very Truly Yours,

MARK GOODWIN & ASSOCIATES, P.A.

for Gregory J. Krenik, P.E.
Project Manager

GJK: smr

City of Albuquerque

P.O. BOX 1293 ALBUQUERQUE, NEW MEXICO 87103

July 28, 1995

Gregory J. Krenik, PE
Mark Goodwin & Assoc
P.O. Box 90606
Albuquerque, NM 87199

RE: GRADING & DRAINAGE PLAN FOR SHONEY'S SUITES (H16-D109)
RECEIVED JULY 18, 1995 FOR BUILDING PERMIT
ENGINEER'S STAMP DATED 7-10-95

Dear Mr. Krenik:

Based on the information included in the submittal referenced above, City Hydrology has the following comments:

City Legal will not accept a Drainage Covenant that has been modified. A clean form is attached. Execute the unmodified form and submit it to City Hydrology for processing.

Provide an emergency spillway to protect the berm on the south side of the pond from breaching. An alternative would be t o eliminate the berm and meet the existing grade at about elevation 5101.

If you have any questions about this project, You may contact me at 768-2727.

Sincerely,

John P. Curtin, PE
Civil Engineer/Hydrology

c: Andrew Garcia
Tom Morgan, ShoLodge, Inc, 600 W Main St, Gallatin, TN 37066

DRAINAGE INFORMATION SHEET

PROJECT TITLE: Shoney's Suites ZONE ATLAS/DRNG,FILE#: H-16 / D / 89

DRB #: _____ EPC #: _____ WORK ORDER #: _____

LEGAL DESCRIPTION: Tract A, Sandia Foundation - AMAFCA Subdivision

CITY ADDRESS: _____

ENGINEERING FIRM: Mark Goodwin & Associates, PA CONTACT: Gregory J. Krenik, PE

ADDRESS: PO Box 90606 PHONE: 345-2010

OWNER: ShoLodge, Inc. CONTACT: Tom Morgan

ADDRESS: 600 W Main St, Gallatin, TN 37066 PHONE: _____

ARCHITECT: Lindsay, Ponder, Brayfield CONTACT: Mr. Brayfield

ADDRESS: 270 Langley Dr, Lawrenceville, GA 30245 PHONE: 404/963-8989

SURVEYOR: _____ CONTACT: _____

ADDRESS: _____ PHONE: _____

CONTRACTOR: _____ CONTACT: _____

ADDRESS: _____ PHONE: _____

TYPE OF SUBMITTAL:

_____ DRAINAGE REPORT
X DRAINAGE PLAN
_____ CONCEPTUAL GRADING & DRAINAGE PLAN
X GRADING PLAN
_____ EROSION CONTROL PLAN
_____ ENGINEER'S CERTIFICATION
X OTHER

PRE-DESIGN MEETING:

_____ YES
X NO
_____ COPY PROVIDED

CHECK TYPE OF APPROVAL SOUGHT:

_____ SKETCH PLAT APPROVAL
_____ PRELIMINARY PLAT APPROVAL
_____ S. DEV. PLAN FOR SUB'D APPROVAL
_____ S. DEV. PLAN FOR BLDG PERMIT APPROVAL
_____ SECTOR PLAN APPROVAL
_____ FINAL PLAT APPROVAL
_____ FOUNDATION PERMIT APPROVAL
X BUILDING PERMIT APPROVAL
_____ CERTIFICATION OF OCCUPANCY APPROVAL
_____ GRADING PERMIT APPROVAL
_____ PAVING PERMIT APPROVAL
_____ S.A.D. DRAINAGE REPORT
_____ DRAINAGE REQUIREMENTS
_____ OTHER _____ (Specify)

DATE SUBMITTED: 7-17-95

BY: _____
Gregory J. Krenik, PE

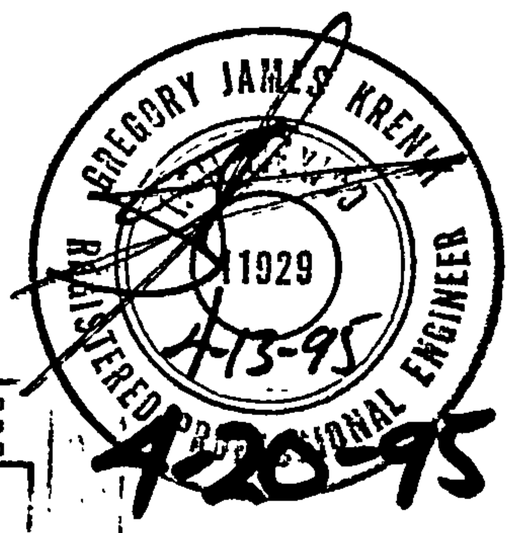
RECEIVED
JUL 18 1995
HYDROLOGY D

DRAINAGE CALCULATIONS
for
SHONEY'S SUITES

Prepared For

ShoLodge Inc.
600 W. Main Street
Gallatin, TN 37066

April 1995



4-13-95
4-20-95
5-19-95
7-17-95

D. Mark Goodwin & Associates, P.A.
Consulting Engineers and Surveyors

PROJECT SHONEY'S HOTEL
SUBJECT DRAINAGE CALCS
BY GJK DATE 4-20-95
CHECKED _____ DATE _____
SHEET 11 OF ____
REVISED 5-19-95 7-17-95

- SIZE TEMPORARY RETENTION POND

ELEVATION	AREA (SF)	STORAGE (CF)
91	950 ~~1750~~	0
92	2156 ~~2606~~	~~3178~~ 1553
93	3604 ~~3896~~	~~5254~~ 4433
94	5014 ~~4879~~	~~9316.5~~ 8742
95	6496 ~~5695~~	~~14,493.5~~ 14,497
96	8050 ~~6883~~	~~20,242.5~~ 21,770

- DETERMINE DIRECT DISCHARGE

WEST HALF OF DEVELOPED SIDE

AREA = 1.08137 AC
TYPE B = 0.28361 AC = 26.23 %
TYPE D = 0.79776 AC = 73.77 %

FROM HYMO OUTPUT SHEETS 12-14
Q = 4.37 CFS
V = 0.1553 AC-FT

4.37 CFS < 7.06 CFS OK

TOTAL SITE UNDEVELOPED

TYPE A = 100 %

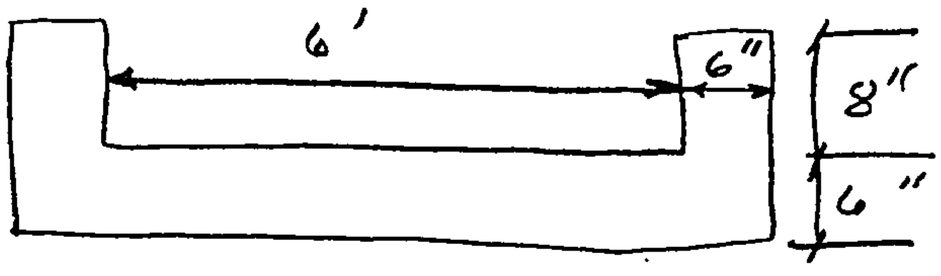
FROM HYMO OUTPUT SHEETS 15-17

Q = 7.06 CFS
V = 0.2001 AC-FT

- SIZE CHANNEL TO MENAUL

THIS SHOULD BE SIZED FOR BUILDOUT

Q = 20.25 - 4.37 = 15.88 CFS



S = 1% n = 0.013

d = 0.5
wp = 7.0
A = 3.0
V = 6.50 F/S
Q = 19.49 CFS OK

D. Mark Goodwin & Associates, P.A.
Consulting Engineers and Surveyors

PROJECT SHONEYS HOTEL
SUBJECT DRAINAGE CALCS
BY GJK DATE 4-20-95
CHECKED _____ DATE _____
SHEET 18 OF ____
Revised 7-17-95

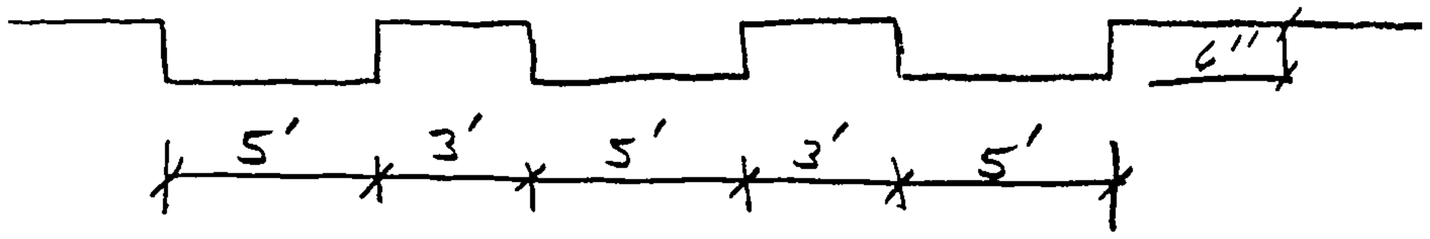
- SIZE WIER OPENING FOR CHANNEL

$Q = 2.95 L H^{1.5}$

$H = 0.5'$

$L = 14.49'$ $\quad Q = 15.1125$

SAY 3 - 5' OPENINGS IN CURB.



- DETERMINE VOLUME TO GO INTO POND

FROM SHEET 1 & SHEET 11

$V = 0.326 + 0.1594 - 0.1553$
$= 0.3301$ AC-FT

$V_{10\,day} = 0.3301 + \frac{0.89426(3.95 - 2.3)}{12}$

$= 0.453$ AC-FT
$= 19,736$ CF $<$ ~~20,742.5~~ 21,220 OK SEE SHEET 11

- DETERMINE IF DOUBLE O INLET IN SUMP IS OKAY

$\frac{Q}{P} = 3.0 H^{3/2}$ $\quad$ P = Perimeter = 12 - SIDES DO NOT COUNT
ASSUME 25% CLOGGED.

$P = 12 \times .75 = 9$
$Q = 9.20 + 5.49 - 4.37$
$= 10.32$ CFS

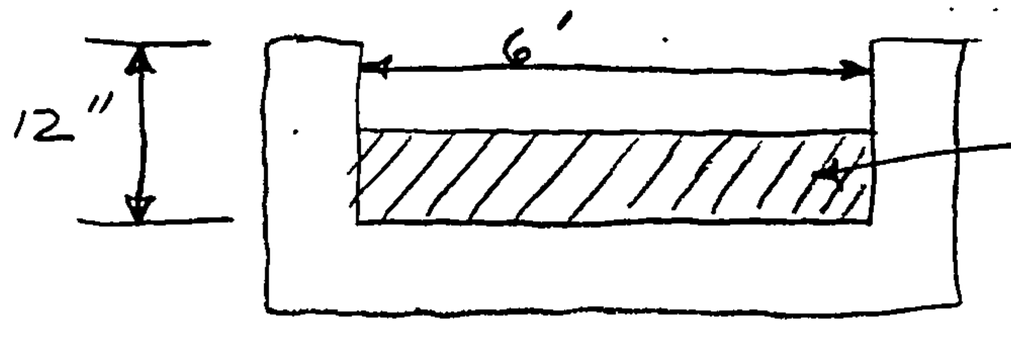
$H = 0.53 < .67$ OK

~~18" RCP AT 1% 30 YRS; HGS~~
24" RCP AT 0.5% 30 YRS 16 CFS
OK

- SIZE SPILLWAY

CONCRETE CHANNEL WILL ACT AS SPILLWAY

WIER WILL BE OVER ASPHALT CURB



$d = 0.34$ $\quad V = 5.1837$ F/S
$w_p = 6.68$
$A = 2.04$

$Q = 10.57$ CFS ≈ 10.32
OK

City of Albuquerque

P.O. BOX 1293 ALBUQUERQUE, NEW MEXICO 87103

June 19, 1995

Gregory J. Krenik, PE
Mark Goodwin & Assoc
P.O. Box 90606
Albuquerque, NM 87199

RE: GRADING & DRAINAGE PLAN FOR SHONEY'S SUITES (H16-D109)
RECEIVED MAY 22, 1995 FOR BUILDING PERMIT
ENGINEER'S STAMP DATED 5-22-95

Dear Mr. Krenik:

Based on the information included in the submittal referenced above, City Hydrology accepts the Grading & Drainage Plan for Building Permit.

A Drainage Covenant for the temporary retention pond must be executed and submitted to City Hydrology for processing.

A separate permit is required for construction of private drainage facilities within the City right of way. A copy of this letter must be on hand when applying for the excavation permit.

Engineer's Certification of grading & drainage per DPM checklist must be approved before any Certificate of Occupancy will be released.

If you have any questions about this project, You may contact me at 768-2727.

Sincerely,

John P. Curtin, PE
Civil Engineer/Hydrology

c: Andrew Garcia
Arlene Portillo
Tom Morgan, ShoLodge, Inc, 600 W Main St, Gallatin, TN 37066

DRAINAGE INFORMATION SHEET

PROJECT TITLE: Shoney's Suites ZONE ATLAS/DRNG,FILE#: H-16/D109

DRB #: _____ EPC #: _____ WORK ORDER #: _____

LEGAL DESCRIPTION: Tract A, Sandia Foundation - AMAFCA Subdivision

CITY ADDRESS: _____

ENGINEERING FIRM: Mark Goodwin & Associates, PA CONTACT: Gregory J. Krenik, PE

ADDRESS: PO Box 90606 PHONE: 345-2010

OWNER: ShoLodge, Inc. CONTACT: Tom Morgan

ADDRESS: 600 W Main St, Gallatin, TN 37066 PHONE: _____

ARCHITECT: Lindsay, Ponder, Brayfield CONTACT: Mr. Brayfield

ADDRESS: 270 Langley Dr, Lawrenceville, GA 30245 PHONE: 404/963-8989

SURVEYOR: _____ CONTACT: _____

ADDRESS: _____ PHONE: _____

CONTRACTOR: _____ CONTACT: _____

ADDRESS: _____ PHONE: _____

TYPE OF SUBMITTAL:

_____ DRAINAGE REPORT
X DRAINAGE PLAN
_____ CONCEPTUAL GRADING & DRAINAGE PLAN
X GRADING PLAN
_____ EROSION CONTROL PLAN
_____ ENGINEER'S CERTIFICATION
X OTHER

PRE-DESIGN MEETING:

_____ YES
X NO
_____ COPY PROVIDED

CHECK TYPE OF APPROVAL SOUGHT:

_____ SKETCH PLAT APPROVAL
_____ PRELIMINARY PLAT APPROVAL
_____ S. DEV. PLAN FOR SUB'D APPROVAL
_____ S. DEV. PLAN FOR BLDG PERMIT APPROVAL
_____ SECTOR PLAN APPROVAL
_____ FINAL PLAT APPROVAL
_____ FOUNDATION PERMIT APPROVAL
X BUILDING PERMIT APPROVAL
_____ CERTIFICATION OF OCCUPANCY APPROVAL
_____ GRADING PERMIT APPROVAL
_____ PAVING PERMIT APPROVAL
_____ S.A.D. DRAINAGE REPORT
_____ DRAINAGE REQUIREMENTS
_____ OTHER _____ (Specify)

DATE SUBMITTED: 5-19-95

BY: _____
Gregory J. Krenik, PE

D. Mark Goodwin & Associates, P.A.
Consulting Engineers and Surveyors
P.O. BOX 90606, ALBUQUERQUE, NM 87199
(505) 345-2010

May 19, 1995

Mr. John Curtin
Hydrology Division
City of Albuquerque
P.O. Box 1293
Albuquerque, NM 87103

Re: Shoney's Suites (H16-D109)

Dear Mr. Curtin:

This is an itemized response to your letter of May 10, 1995.

I've included the text portion of the Menaul Detention Basin and Storm Drain Report and excerpts that pertain to our particular basin. Also sheet 4 of 29 of the MDBSR plans has been included in the report.

Since the downstream improvements will occur after the hotel construction I've included a temporary retention pond which will retain the developed runoff. We will discharge just over 4 cfs from the site but less than the 7 cfs that is generated in the undeveloped state.

We've added the SO-19 and the note about filling the east side to 5100.00

Very Truly Yours,

MARK GOODWIN & ASSOCIATES, P.A.

Gregory J. Krenik, P.E.
Project Manager

GJK: smr

City of Albuquerque

P.O. BOX 1293 ALBUQUERQUE, NEW MEXICO 87103

May 10, 1995

Gregory J. Krenik, PE
Mark Goodwin & Assoc
P.O. Box 90606
Albuquerque, NM 87199

RE: GRADING & DRAINAGE PLAN FOR SHONEY'S SUITES (H16-D109)
RECEIVED APRIL 14, 1995 FOR BUILDING PERMIT
ENGINEER'S STAMP DATED 4-20-95

Dear Mr. Krenik:

Based on the information included in the submittal referenced above, City Hydrology has the following comments that must be addressed before the Grading & Drainage Plan will be accepted for Building Permit:

Document predesign with Pat Conley & Carlos Montoya. Provide excerpts from "Menaul Detention Basin Report" that support your conclusions. What interim measures are proposed until downstream improvements are completed. Include a copy of Sheet 4 of 29, "Menaul Detention Basin and Storm Drain" in the next submittal. Do any off-site flows enter the site.

The sidewalk culverts require an S.O.19. Add the necessary notes and provide the signature blocks for approvals per the DPM, chapter 22, page 116. Note that eastern area must be filled to elevation 5100.00. Proposed contours do not clearly indicate this. Retention Pond should be labeled Desiltation Pond.

If you have any questions about this project, You may contact me at 768-2727.

Sincerely,

John P. Curtin

John P. Curtin, P.E.
Civil Engineer/Hydrology

c: Andrew Garcia
Tom Morgan, ShoLodge, Inc, 600 W Main St, Gallatin, TN 37066

DRAINAGE INFORMATION SHEET

PROJECT TITLE: Shoney's Suites ZONE ATLAS/DRNG,FILE#: H-16

DRB #: _____ EPC #: _____ WORK ORDER #: _____

LEGAL DESCRIPTION: Tract A, Sandia Foundation - AMAFCA Subdivision

CITY ADDRESS: _____

ENGINEERING FIRM: Mark Goodwin & Associates, PA CONTACT: Gregory J. Krenik, PE

ADDRESS: PO Box 90606 PHONE: 345-2010

OWNER: ShoLodge, Inc. CONTACT: Tom Morgan

ADDRESS: 600 W Main St, Gallatin, TN 37066 PHONE: _____

ARCHITECT: Lindsay, Ponder, Brayfield CONTACT: Mr. Brayfield

ADDRESS: 270 Langley Dr, Lawrenceville, GA 30245 PHONE: 404/963-8989

SURVEYOR: _____ CONTACT: _____

ADDRESS: _____ PHONE: _____

CONTRACTOR: _____ CONTACT: _____

ADDRESS: _____ PHONE: _____

TYPE OF SUBMITTAL:

_____ DRAINAGE REPORT
X DRAINAGE PLAN
_____ CONCEPTUAL GRADING & DRAINAGE PLAN
X GRADING PLAN
_____ EROSION CONTROL PLAN
_____ ENGINEER'S CERTIFICATION
X OTHER

PRE-DESIGN MEETING:

_____ YES
X NO
_____ COPY PROVIDED

CHECK TYPE OF APPROVAL SOUGHT:

_____ SKETCH PLAT APPROVAL
_____ PRELIMINARY PLAT APPROVAL
_____ S. DEV. PLAN FOR SUB'D APPROVAL
_____ S. DEV. PLAN FOR BLDG PERMIT APPROVAL
_____ SECTOR PLAN APPROVAL
_____ FINAL PLAT APPROVAL
_____ FOUNDATION PERMIT APPROVAL
X BUILDING PERMIT APPROVAL
_____ CERTIFICATION OF OCCUPANCY APPROVAL
_____ GRADING PERMIT APPROVAL
_____ PAVING PERMIT APPROVAL
_____ S.A.D. DRAINAGE REPORT
_____ DRAINAGE REQUIREMENTS
_____ OTHER _____ (Specify)

DATE SUBMITTED: 4-21-95

BY: _____
Gregory J. Krenik, PE

RECEIVED
APR 14 1995
HYDROLOGY DIVISION

3

DRAINAGE INFORMATION SHEET

109
H-19

PROJECT TITLE: Shoney's Suites ZONE ATLAS/DRNG.FILE#: H-16

DRB #: _____ EPC #: _____ WORK ORDER #: _____

LEGAL DESCRIPTION: Tract A, Sandia Foundation - AMAFCA Subdivision

CITY ADDRESS: 2500 Menaul N.E.

ENGINEERING FIRM: Mark Goodwin & Associates, PA CONTACT: Gregory J. Krenik, PE

ADDRESS: PO Box 90606 PHONE: 345-2010

OWNER: ShoLodge, Inc. CONTACT: Tom Morgan

ADDRESS: 600 W Main St, Gallatin, TN 37066 PHONE: _____

ARCHITECT: Lindsay, Ponder, Brayfield CONTACT: Mr. Brayfield

ADDRESS: 270 Langley Dr, Lawrenceville, GA 30245 PHONE: 404/963-8989

SURVEYOR: _____ CONTACT: _____

ADDRESS: _____ PHONE: _____

CONTRACTOR: _____ CONTACT: _____

ADDRESS: _____ PHONE: _____

TYPE OF SUBMITTAL:

_____ DRAINAGE REPORT
_____ DRAINAGE PLAN
_____ CONCEPTUAL GRADING & DRAINAGE PLAN
_____ GRADING PLAN
_____ EROSION CONTROL PLAN
X ENGINEER'S CERTIFICATION
_____ OTHER

PRE-DESIGN MEETING:

_____ YES
_____ NO
_____ COPY PROVIDED

CHECK TYPE OF APPROVAL SOUGHT:

_____ SKETCH PLAT APPROVAL
_____ PRELIMINARY PLAT APPROVAL
_____ S. DEV. PLAN FOR SUB'D APPROVAL
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_____ FINAL PLAT APPROVAL
_____ FOUNDATION PERMIT APPROVAL
_____ BUILDING PERMIT APPROVAL
X CERTIFICATION OF OCCUPANCY APPROVAL
_____ GRADING PERMIT APPROVAL
_____ PAVING PERMIT APPROVAL
_____ S.A.D. DRAINAGE REPORT
_____ DRAINAGE REQUIREMENTS
_____ OTHER _____ (Specify)

30 day Temp C.O. on 4-18-96 Ag.

DATE SUBMITTED: 4-18-96

BY: _____
Gregory J. Krenik, PE

M Goodwin.
828-2200

RECEIVED
APR 18 1996
HYDROLOGY DIV

Greg Krenik
Mark Goodwin & Associates

VERBAL 5/16/96

H16-D109

PM

1. Copy of Executed Covenant
2. Concurrence for sidewalk culverts
3. Concurrence from AMAFCA for placement of fill.

5/16/96 M. Goodwin.

828-2000
828 2200

Greg Krenik
Mark Goodwin & Associates
P.O. Box 90606
Albuquerque, New Mexico

RE: Engineer Certification For Shoney's Suites (H16-D105)
Certification Statement dated 4/18/96

Dear Mr. Krenik:

Based on the information provided on your April 18 1996 submittal, Engineer Certification for the above referenced site is acceptable.

If I can be of further assistance, please feel free to contact me at 768-2667

Sincerely

Bernie Montoya P.E.
Engineering Associate

C. Andrew Garcia