DRAINAGE REPORT
for

Chili's at University & Gibson
1700 Miles Road

Prepared by:

Tierra West, LLC
5571 Midway Park Place NE
Albuquerque, New Mexico 87109

Revised December, 2014

I certify that this report was prepared under my supervision, and I am a registered professional engineer in the State of New Mexico in good standing.

Ronald R. Bohannan, PE

RONALD R. BOHANNAN
NEW MEXICO
7868
PROFESSIONAL ENGINEER

Job No 2013086

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Appendix (Calculations)

Plan Attachments

Purpose

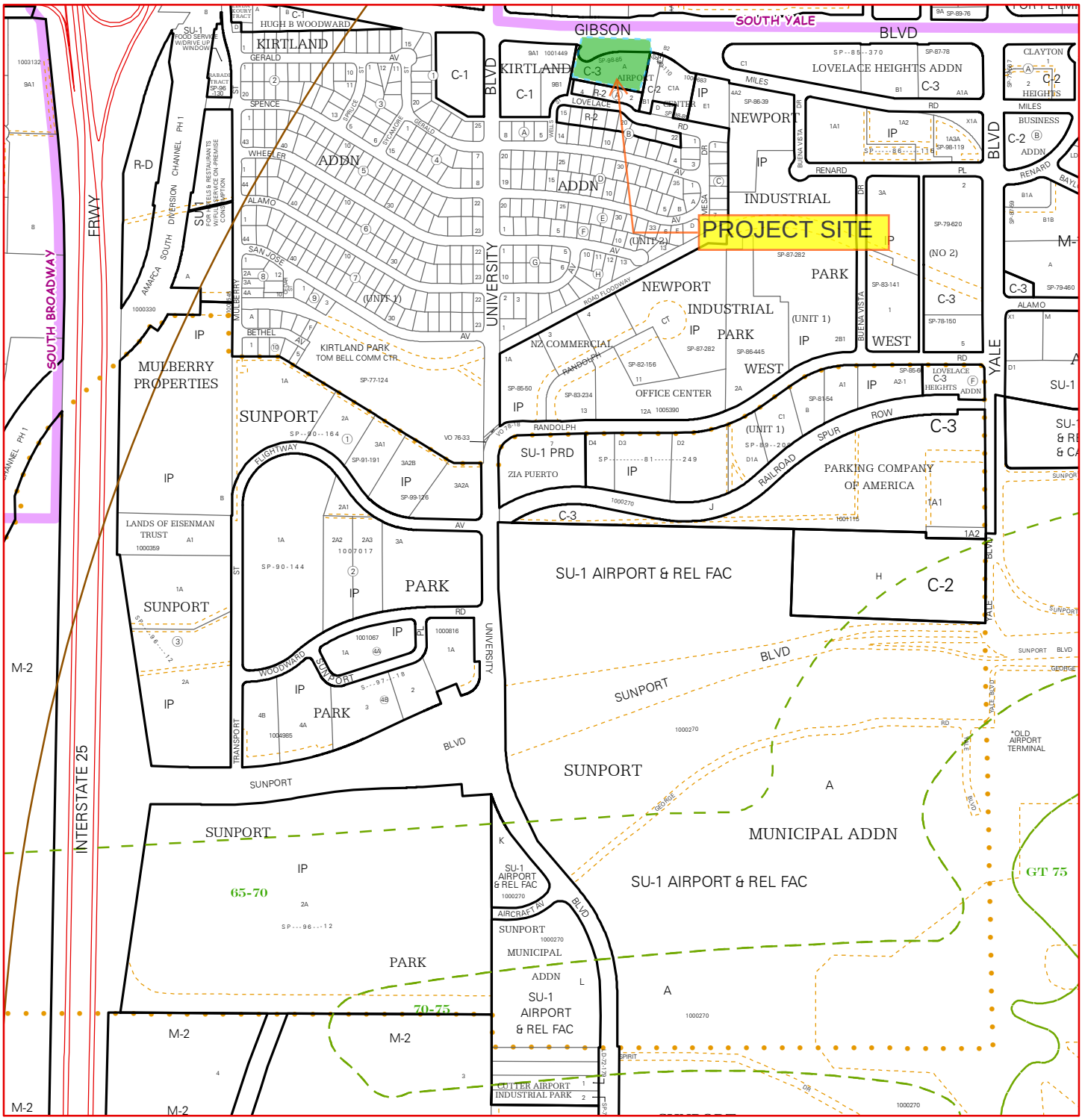
The purpose of this report is to provide the drainage management plan for redevelopment of a property for a Chili's sit-down restaurant to be located on the south side of Miles Road, near the southeast corner of Gibson and University Boulevard in Albuquerque, New Mexico. This plan is in accordance with the City of Albuquerque's Development Process Manual, Chapter 22, Hydrology Section.

Introduction

The subject of this report, as shown on the Exhibit A - Vicinity Map, is an approximately 1.9-acre property located near the westerly terminus of Miles Road. The property address is 1700 Miles Road SE, has a legal description of Tract A of Tracts A-E, Airport Center, and is zoned C-3. The site is vacant, but currently developed with a building and associated parking lot formerly used as a rental car building. A vacation action for the Miles Road public right of way along the property frontage has been approved and the parcel will be re-platted to incorporate the vacated property. A Master Drainage Plan was prepared for this area by Jeff Mortensen & Associates, Inc., updated on March 4, 1997 (JMA Report) and is included for reference in the Appendix.

Flood Plain

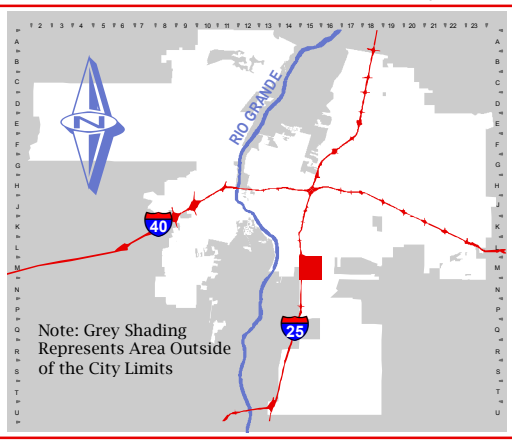
The site is not within a floodplain as shown on FIRM Map 35001C0342G. See Exhibit B for location of site.



For more current information and more details visit: http://www.cabq.gov/gis

Albuquerque Geographic Information System

Map amended through: 1/24/2011



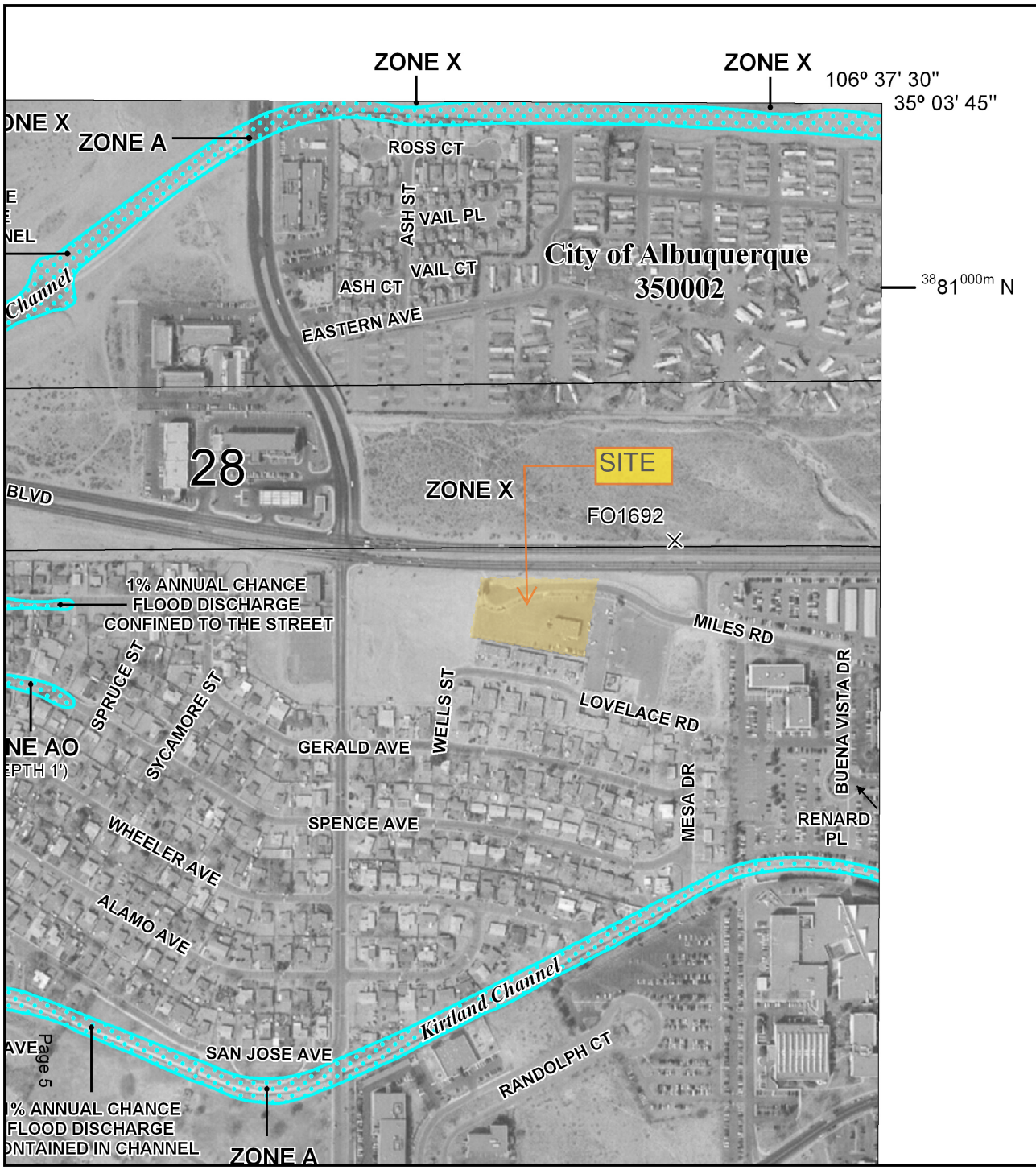
Zone Atlas Page:

M-15-Z

Selected Symbols

- SECTOR PLANS
- Design Overlay Zones
- City Historic Zones
- H-1 Buffer Zone
- Petroglyph Mon.
- Escarpment
- 2 Mile Airport Zone
- Airport Noise Contours
- Wall Overlay Zone

0 750 1,500 Feet



MAP SCALE 1" = 500'

0 500 1000 FEET

NFIP

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 0342G

FIRM

FLOOD INSURANCE RATE MAP

BERNALILLO COUNTY, NEW MEXICO

AND INCORPORATED AREAS

PANEL 342 OF 825

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
ALBUQUERQUE, CITY OF	350002	0342	G
BERNALILLO COUNTY UNINCORPORATED AREAS	350001	0342	G

Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

MAP NUMBER
35001C0342G

MAP REVISED
SEPTEMBER 26, 2008

Federal Emergency Management Agency

Existing Drainage Conditions

Currently the site is developed with an office building and parking lot. The project site is bounded on the north by Gibson Boulevard, on the west by a partially developed property, on the south by a residential subdivision, and to the east by a parking lot. This site is included in the JMA Report (See Appendix), an d analyzed as Drainage Basin C.

The property is steeply sloped from east to west with drainage from the site being conveyed to Miles Road by sheet flow and a concrete rundown, which drains west to a series of drainage inlets at the end of the cul-de-sac. The street grade of Miles Road at the property frontage is approximately 5% and the parking lot grades within the site are approximately 8%. The JMA Report anticipated all developed flows from this site and upstream basins, calculated to be 20.4 cfs, to be conveyed to the end of the Miles Road cul-de-sac.

Storm runoff from the adjacent parking lot to the east (Basin DE-1 per the JMA Report) sheet flows into the subject property at a calculated rate of 5.0 cfs. An additional offsite basin, Basin F-1, is taken into account by the JMA Report and assumed to flow into Miles Road in the ultimate developed condition, although said basin remains undeveloped and flows south- away from Miles Road . The portion of Miles Road which drains to the end of the cul-de-sac is designated as Basin M and contributes 6.0 cfs.

Proposed Drainage Conditions

The proposed development will consist of a 6,000 square foot restaurant building with 117 parking spaces and associated landscape. Surface improvements of Miles Road are proposed to be demolished, as are all private improvements within the project property. A private driveway connection is anticipated to be constructed in the future in conjunction with the commercial development proposed on the adjacent property to the west as a separate project.

Six onsite drainage basins are delineated on the site corresponding to the proposed grading configuration and impervious land treatment for the developed condition. The “onsite” portion of Basin M previously delineated by the JMA Report is accounted for by the proposed onsite basins, while the offsite portion of Basin M and the entirety of the other two offsite basins are accounted for. Although the Drainage Plan in the JMA report suggests flow from that basin being conveyed to Miles Road, this analysis accounts for that flow to be accepted by this development at two points along the easterly boundary consistent with the topography and parking lot improvements. All basins are analyzed under fully developed conditions.

Onsite and offsite flows from Basin M are designed to be conveyed to an existing curb inlet on the north side of Miles Road via curb & gutter and concrete rundowns. This existing curb inlet is designed to be raised in grade and modified to a Type D inlet, identified as Analysis Point #2 (AP#2). Surface flows calculated at 1.11 cfs will still be conveyed to the drainage structures end of the cul-de-sac in the developed condition, whereas the pre-development condition handles 20.4 cfs.

A public storm drain easement will be dedicated by plat to contain flows from the public right of way. Surface flows from the parking lot area and landscaped areas are designed to be captured by private drainage facilities consisting of drainage inlets and storm drain pipe connecting to the existing storm drain system on the southerly portion of the Miles Road cul-de-sac. A small slope area on the westerly portion of the site will sheet flow onto the adjacent property to the west at a calculated rate of 0.4 cfs during the 100-yr storm event. A similar, albeit smaller, tributary area from the existing slope has historically drained to the adjacent site, however, it is impractical to divert flow from this slope area into the existing storm drain system due to topography.

The grading design is configured to maintain historical drainage patterns and accept drainage from the adjacent property consistent with the existing conditions and in substantial conformance with the approved Master Drainage Plan as outlined in the JMA Report. Total peak flows from onsite and offsite basins in the proposed developed condition will not increase from the existing condition (20.37 vs. 20.58 cfs) due to the proposed demolition of the expansive parking lot and road improvements to be replaced with landscape areas meeting current zoning code requirements.

Stormwater Control Measures

Stormwater Control Measures are incorporated in the design to the extent practicable and will provide management of the 90th Percentile Storm. Design measures include passive water harvesting in depressed parking lot islands, a

vegetated gravel-lined swale along the southerly and northerly property boundary, and three small retention ponds. The 90th Percentile Storm was quantified per the Drainage Ordinance requirement of 0.44 inches and reduced by 0.1 inch to account for the initial impervious abstraction as listed in Table A-6 of Section 22 of the DPM. Detailed pond volume design tabulations are included in Appendix A.

As noted above, the proposed use is less intense than the existing and there should be an improvement in stormwater quality due to the proposed design. An Erosion Control and Sediment Control Plan was designed in conjunction with the grading and drainage plan which will implement best management practices during construction activities and is included with this report.

Summary

Following a detailed analysis of existing and proposed drainage conditions guided by DPM Section 22 – Weighted E Method, storm water discharge resulting from the 100-year, 6-hr storm event indicates that the proposed redevelopment and corresponding grading and drainage design will accommodate the proposed development, and correspondingly not increase run-off volumes or alter historic discharge locations. It is therefore recommended that this development be approved for grading and Site Plan Development for Building Permit based upon these findings.

APPENDIX A

HYDROLOGY

EXISTING DRAINAGE BASINS

LEGAL DESCRIPTION

LOT 9, BLOCK A, KIRTLAND ADDITION
LOT 1, BLOCK A, KIRTLAND ADDITION
UNPLATTED PARCELS
TRACT 4-A-1, NEWPORT INDUSTRIAL PARK-WEST, UNIT 2
MILES RD. S.E., WALKER DRIVE S.E., AND PUBLIC ALLEYS

PROJECT BENCHMARK = T.B.M.

CITY OF ALBUQUERQUE BENCHMARK "G-9A".
STATION IS STAMPED "ACS,G-9A, 1984", A 3 1/4" ALUMINUM TABLET SET FLUSH IN THE CONCRETE GUTTER. TO REACH THE STATION FROM I-25, TRAVEL EAST ON GIBSON BLVD. 0.7 MILES. AS SHOWN BELOW.
ELEVATION = 5143.07' (M.S.L.D.)

LEGEND

TC	TOP OF CURB	24	EXISTING CONTOUR
FL	FLOW LINE	24	PROPOSED CONTOUR
NG	NATURAL GROUND	91.93	PROPOSED SPOT ELEVATION
91.93	SPOT ELEVATION		PROPOSED FLOWLINE
	GUY WIRE/ANCHOR	q=	ATTENUATED FLOW RATE
	POWER POLE		BASIN BOUNDARY
	LIGHT POLE		FUTURE STORM DRAIN
	MANHOLE		PROPOSED RETAINING WALL
	CONTROL POINT		

NOTE: THIS IS NOT A BOUNDARY SURVEY. APPARENT PROPERTY CORNERS ARE SHOWN FOR ORIENTATION ONLY.

SCALE: 1" = 40'

40 0 80
20 40

GIBSON BOULEVARD S.E.
(RIGHT-OF-WAY VARIES)

PROJECT BENCHMARK=T.B.M.
FD. BRASS CAP STAMPED "G-9A"
ELEVATION=5143.07

136' RIGHT-OF-WAY
120' RIGHT-OF-WAY

TRAFFIC CONTROL
TWO 15" SIDEWALK CULVERTS PER C.O.A. STD. DWG. 2236
TRAFFIC SIGNAL
48" RCP
42" RCP
TRAFFIC CONTROL

Q_{100} = 2.7 cfs (FUTURE)
Q_{100} = 3.6 cfs
Q_{100} = 20.4cfs TOTAL
Q_{100} = 6.0 cfs
Q_{100} = 5.6 cfs
Q_{100} = 5.0 cfs
Q_{100} = 2.7 cfs
Q_{100} = 3.8 cfs
Q_{100} = 0.4 cfs
Q_{100} = 0.5 cfs
q= 0.8 cfs (INTERIM)
q= 0.8 cfs
q= 0.8 cfs

BASIN A-1
BASIN A-2
BASIN B
BASIN C
BASIN M
BASIN DE-1
BASIN DE-2
BASIN F-1
BASIN F-2
BASIN G

MILES ROAD S.E. (IMPROVED)
C.O.A. PROJECT #5319.90
WALKER DRIVE S.E.
C.O.A. PROJECT #5319.90
MILES ROAD S.E. EXCESS R.O.W. TO BE VACATED
FORMER WALKER DRIVE S.E. (TO BE VACATED)

LOT 1, BLOCK A KIRTLAND ADDITION
LOT 9, BLOCK A KIRTLAND ADDITION
PRIVATE ACCESS ROAD
UNPLATTED PARCEL
UNPLATTED PARCEL
TRACT 4-A-1 NEWPORT INDUSTRIAL PARK - WEST UNIT 2

LOVELACE ROAD S.E. (50' R.O.W.)
LOVELACE ROAD S.E. (60' R.O.W.)
UNIVERSITY BOULEVARD S.E. (80' R.O.W.)
WELLS ST. S.E. (50' R.O.W.)

392.91
MAX. W.S.L. 5092.0
MAX. W.S.L. 5092.0
S 89°43'39" E
710.80'
N 79°07'43" W 356.37'
N 10°52'17" E 100.18'
N 00°04'27" E
N 89°55'33" W 258.46'
S 00°06'19" W 253.86'
N 79°11'11" W 479.40'
S=1%
S=1%
S=1%

LANDSCAPING
PROPOSED BUILDING
FUTURE BUILDING ADDITION
PROPOSED CAR WASH
DETENTION POND
EXISTING OFFICE BUILDING F.F. = 5131.70
END OF CURB
WATER VALVE BOX
SAS MANHOLE
4" CURB PENETRATION PER C.O.A. STD. DWG. 2235

18"x 18" DRAINAGE INLET TG=91.2
6" PVC (SDR-35) STORM DRAIN @ S=0.0200 DAYLIGHTED INTO SIDEWALK CULVERT (INTERIM)
24" SIDEWALK CULVERT PER C.O.A. STD. DWG. 2236 (INTERIM)
FUTURE CONNECTION TO BACK OF INLET
18"x 18" DRAINAGE INLET TG=91.2
6" PVC (SDR-35) STORM DRAIN @ S=0.0200 DAYLIGHTED INTO SIDEWALK CULVERT
24" SIDEWALK CULVERT PER C.O.A. STD. DWG. 2236
FD. REBAR W/ CAP
FD. REBAR W/CAP
20' PUBLIC ALLEY (VACATED BY V-95-7)
26' PUBLIC ALLEY

KEYED NOTES:

1. EXISTING 5' UTILITY AND DRAINAGE EASEMENT RESERVED BY COVENANTS, CONDITIONS, RESTRICTIONS, TERMS, PROVISIONS AND EASEMENTS RECORDED IN BOOK D255, PAGE 312, RECORDS OF BERNALILLO COUNTY, NEW MEXICO, TO BE VACATED.
2. EXISTING DOWN GUY EASEMENT PER UNRECORDED SURVEY PREPARED BY ANDREWS, ASBURY & ROBERT, INC. DATED JANUARY 02, 1979.

DETENTION TIME CALCULATIONS

A. BASIN B
$q_{avg} = 1/2\ q_{100} = (1/2)(0.8) = 0.4$ cfs
$V_{100} = 7{,}880$
TIME TO DRAIN = $\frac{(7{,}880\ cf)}{(0.4\ cfs)}$ (1 min./60 sec.)(1 hr./60 min.) = 5.5 hours

B. BASIN F-2
$q_{avg} = 1/2\ q_{100} = (1/2)(0.8) = 0.4$ cfs
$V_{100} = 4{,}440$ cf
TIME TO DRAIN = $\frac{(4{,}440\ cf)}{(0.4\ cfs)}$ (1 min./60 sec.)(1 hr./60 min.) = 3.1 hours

THIS PLAN REPLACES THE PREVIOUS PLAN DATED 02-15-1996.

JEFFREY G. MORTENSEN NEW MEXICO 8547 REGISTERED PROFESSIONAL ENGINEER
03-04-97

JEFF MORTENSEN & ASSOCIATES, INC.
6010-B MIDWAY PARK BLVD. N.E.
ALBUQUERQUE NEW MEXICO 87109
ENGINEERS SURVEYORS (505) 345-4250

MASTER DRAINAGE PLAN
MILES ROAD S.E. / KIRTLAND ADDITION

DESIGNED BY G.M./G.R.B.
DRAWN BY T.N.T.
APPROVED BY J.G.M.

NO.	DATE	BY	REVISIONS

JOB NO. 950925
DATE 03-1997
SHEET 1 OF 2

C:\DATA\ACADWIN\950925\950925M4 Mon Mar 3 15:05:32 1997 -TNT

POST-DEVELOPMENT DRAINAGE BASINS

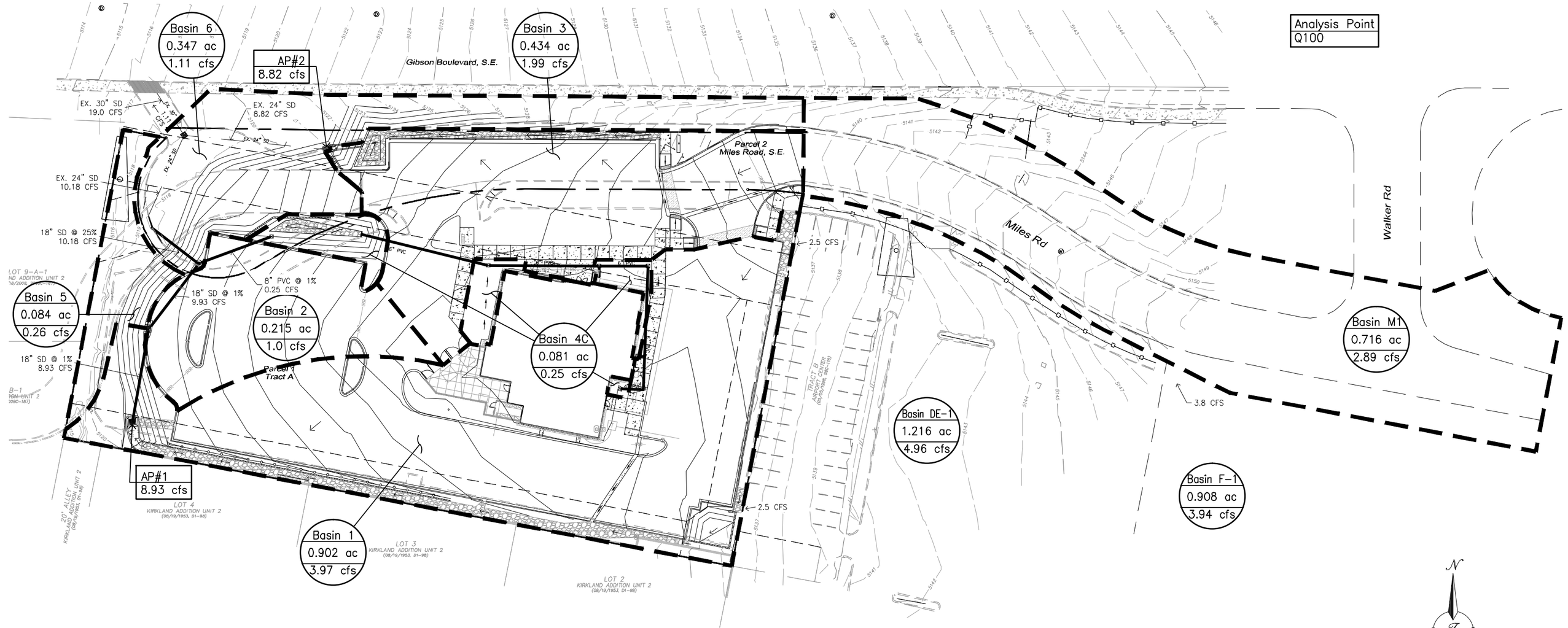
LEGEND

DRAINAGE BASIN

FLOW DIRECTION

Basin ID
Basin Area
Q_{100} (cfs)

Analysis Point
Q100



AP#1=[Basin 1]+[Basin DE-1]=8.93 cfs
AP#2=[Basin 3]+[Basin M1 (Offsite)]+[Basin F-1]=8.82 cfs

N

NTS

Weighted E Method

Albuquerque Chili's University & Gibson (Miles Road). - Zone #2

Pre-Development Basins

Basin Description												100-Year, 6-Hr			10-Year, 6-Hr			2-Year, 6-Hr		
Basin	Area (sf)	Area (acres)	Area (sq miles)	Treatment A %	Treatment A (acres)	Treatment B %	Treatment B (acres)	Treatment C %	Treatment C (acres)	Treatment D %	Treatment D (acres)	Weighted E (ac-ft)	Volume (ac-ft)	Flow cfs	Weighted E (ac-ft)	Volume (ac-ft)	Flow cfs	Weighted E (ac-ft)	Volume (ac-ft)	Flow cfs
C	59,100	1.357	0.00212	0%	0	13%	0.176	16%	0.21708	71%	0.963	1.787	0.202	5.61	1.071	0.121	3.56	0.588	0.066	1.94
M	58,550	1.344	0.00210	0%	0	0%	0.000	12%	0.161295	88%	1.183	2.001	0.224	6.07	1.242	0.139	3.99	0.713	0.080	2.30
DE-1	52,950	1.216	0.00190	0%	0	14%	0.170	18%	0.218802	68%	0.827	1.754	0.178	4.96	1.044	0.106	3.13	0.567	0.057	1.68
F-1	39,550	0.908	0.00142	0%	0	0%	0.000	23%	0.208827	77%	0.699	1.892	0.143	3.94	1.151	0.087	2.55	0.643	0.049	1.43
Total	**210,150**	**4.824**	**0.00754**										**0.747**	**20.58**		**0.453**	**13.24**		**0.252**	**7.34**

Post-Development Basins

Basin Description												100-Year, 6-Hr			10-Year, 6-Hr			2-Year, 6-Hr		
Basin	Area (sf)	Area (acres)	Area (sq miles)	Treatment A %	Treatment A (acres)	Treatment B %	Treatment B (acres)	Treatment C %	Treatment C (acres)	Treatment D %	Treatment D (acres)	Weighted E (ac-ft)	Volume (ac-ft)	Flow cfs	Weighted E (ac-ft)	Volume (ac-ft)	Flow cfs	Weighted E (ac-ft)	Volume (ac-ft)	Flow cfs
1	39,283	0.902	0.00141	0%	0	0%	0.000	19%	0.171345	81%	0.730	1.932	0.145	3.97	1.184	0.089	2.59	0.668	0.050	1.46
2	9,360	0.215	0.00034	0%	0	0%	0.000	4%	0.008595	96%	0.206	2.080	0.037	1.00	1.307	0.023	0.66	0.764	0.014	0.39
3	18,908	0.434	0.00068	0%	0	0%	0.000	7%	0.030385	93%	0.404	2.051	0.074	1.99	1.283	0.046	1.32	0.745	0.027	0.77
4	3,508	0.081	0.00013	0%	0	0%	0.000	100%	0.080533	0%	0.000	1.130	0.008	0.25	0.520	0.003	0.14	0.150	0.001	0.05
5	3,647	0.084	0.00013	0%	0	0%	0.000	100%	0.083724	0%	0.000	1.130	0.008	0.26	0.520	0.004	0.14	0.150	0.001	0.05
6	15,094	0.347	0.00054	0%	0	0%	0.000	96%	0.33265	4%	0.014	1.170	0.034	1.11	0.553	0.016	0.61	0.176	0.005	0.23
M1 (offsite)	31,190	0.716	0.00112	0%	0	0%	0.000	43%	0.30789	57%	0.408	1.694	0.101	2.89	0.987	0.059	1.81	0.515	0.031	0.94
DE-1	52,950	1.216	0.00190	0%	0	14%	0.170	18%	0.218802	68%	0.827	1.754	0.178	4.96	1.044	0.106	3.13	0.567	0.057	1.68
F-1	39,550	0.908	0.00142	0%	0	0%	0.000	23%	0.208827	77%	0.699	1.892	0.143	3.94	1.151	0.087	2.55	0.643	0.049	1.43
Total	**213,490**	**4.901**	**0.00766**										**0.728**	**20.37**		**0.434**	**12.95**		**0.235**	**7.00**

Equations:

Weighted E = E_a*A_a + E_b*A_b + E_c*A_c + E_d*A_d / (Total Area)

Volume = Weighted D * Total Area

Flow = Q_a * A_a + Q_b * A_b + Q_c * A_c + Q_d * A_d

First Flush and Pond Volume Calculations

North Swale Pond

ELEVATION	AREA SF	INCREMENT VOL, CF	CUM VOL CF	CUM VOL AC-FT
26	8			
27	100	54	54	0.0012
27.5	191	73	**127**	0.0029

Landscape Island
west

ELEVATION	AREA SF	INCREMENT VOL, CF	CUM VOL CF	CUM VOL AC-FT
25.7	50			
26.7	224	137	**137**	0.0031

Landscape Island
east

ELEVATION	AREA SF	INCREMENT VOL, CF	CUM VOL CF	CUM VOL AC-FT
29.1	50			
30.1	224	137	**137**	0.0031

West POND

ELEVATION	AREA SF	INCREMENT VOL, CF	CUM VOL CF	CUM VOL AC-FT
25	26			
26	288	157	157	0.0036
27	654	471	**628**	0.0144

Southeast POND

ELEVATION	AREA SF	INCREMENT VOL, CF	CUM VOL CF	CUM VOL AC-FT
34	217			
35	313	265	265	0.0061
36	424	369	**634**	0.0145

Total Vol Provided= **1662** cubic feet

Volume Required = A_d * (0.44in-0.1in)

A_d=1.317 acres = 57368 cubic feet

Vol Req'd= 1625 cubic feet< Vol Provided, Therefore **OK**

APPENDIX B

HYDRAULIC CALCULATIONS

Worksheet for Triangular Channel - South Rock Swale AP#1

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.033	
Channel Slope	0.01500	ft/ft
Left Side Slope	3.00	ft/ft (H:V)
Right Side Slope	3.00	ft/ft (H:V)
Discharge	8.93	ft³/s

Results

Normal Depth	0.96	ft	<1', therefore OK.
Flow Area	2.74	ft²	
Wetted Perimeter	6.05	ft	
Hydraulic Radius	0.45	ft	
Top Width	5.74	ft	
Critical Depth	0.89	ft	
Critical Slope	0.02232	ft/ft	
Velocity	3.26	ft/s	
Velocity Head	0.16	ft	
Specific Energy	1.12	ft	
Froude Number	0.83		
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	0.96	ft
Critical Depth	0.89	ft
Channel Slope	0.01500	ft/ft
Critical Slope	0.02232	ft/ft

Worksheet for Rectangular Channel - 2' wide AP#2

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.013	
Channel Slope	0.04000	ft/ft
Bottom Width	2.00	ft
Discharge	8.82	ft³/s

Results

Normal Depth	0.43	ft	<0.5', therefore OK.
Flow Area	0.86	ft²	
Wetted Perimeter	2.86	ft	
Hydraulic Radius	0.30	ft	
Top Width	2.00	ft	
Critical Depth	0.85	ft	
Critical Slope	0.00590	ft/ft	
Velocity	10.25	ft/s	
Velocity Head	1.63	ft	
Specific Energy	2.06	ft	
Froude Number	2.76		
Flow Type	Supercritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	0.43	ft
Critical Depth	0.85	ft
Channel Slope	0.04000	ft/ft
Critical Slope	0.00590	ft/ft

CAPACITY OF SINGLE 'C' STORM DI @ BASIN 2

Capacity of the grate:

L = 47.375" - 2(6" $_{ends}$) - 14(½" $_{middle\ bars}$)
= 28.375"
= 2.365'

W = 30" - 13(½" $_{middle\ bars}$)
= 23.5"
= 1.958'

Area = 2.365' x 1.958'
= 4.63 ft^2

Effective Area = 4.63 – 4.63 (0.5 $_{clogging\ factor}$)
= 2.3 ft^2 at the grate

Orifice Equation

Q = CA sqrt(2gH)
Q = 0.6*2.3*sqrt(2*32.2*0.67)
Q = 9.06 cfs

Capacity of the throat:

L = 47-⅜"
= 3.948'

Weir Equation

Q = CLH^(3/2)
Q = 2.95 * 3.948 * 0.67^(3/2)
Q = 6.39 cfs

Total Capacity:

$Q = 9.06_{grate} + 6.39_{throat}$
Q = 15.45 cfs

Q (CAPACITY)=15.45 CFS > Q (REQUIRED)=1.0 CFS, THEREFORE OK.

SINGLE ‘D’ TYPE STORM DRAIN INLET @ AP#1

SINGLE ‘D’:

Area at the grate:

L = 38.375" - 7 (1/2" middle bars)
= 34.875"
= 2.906'

W = 25.5" - 13 (1/2 middle bars)
= 19"
= 1.583'

Area = 1.583' x 2.906'
= 4.601 ft^2

Effective Area = 4.601 - 0.5 (4.601)
= 2.30 ft^2

Effective Area = 2.30 ft^2

Orifice Equation

Q = CA sqrt(2gH)
Q = 0.6*2.3*sqrt(2*32.2*1.0)
Q = 11.07 cfs

Q (CAPACITY)=8.93 CFS > Q (REQUIRED)=2.83 CFS, THEREFORE OK.

SINGLE ‘D’ TYPE STORM DRAIN INLET @ AP#2

SINGLE ‘D’:

Area at the grate:

L = 38.375" - 7 (1/2" middle bars)
= 34.875"
= 2.906'

W = 25.5" - 13 (1/2 middle bars)
= 19"
= 1.583'

Area = 1.583' x 2.906'
= 4.601 ft^2

Effective Area = 4.601 - 0.5 (4.601)
= 2.30 ft^2

Effective Area = 2.30 ft^2

Orifice Equation

Q = CA sqrt(2gH)
Q = 0.6*2.3*sqrt(2*32.2*1.0)
Q = 11.07 cfs

Q (CAPACITY)=11.07 CFS > Q (REQUIRED)=8.82 CFS, THEREFORE OK.

Worksheet for Circular Pipe - 18in, Capacity @1% slope

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.013		
Channel Slope	0.01000	ft/ft	
Diameter	18	in	
Discharge	10.18	ft³/s	=Q required

Results

Normal Depth	1.19	ft	
Flow Area	1.50	ft²	
Wetted Perimeter	3.30	ft	
Hydraulic Radius	0.46	ft	
Top Width	1.22	ft	
Critical Depth	1.23	ft	
Percent Full	79.3	%	
Critical Slope	0.00940	ft/ft	
Velocity	6.77	ft/s	
Velocity Head	0.71	ft	
Specific Energy	1.90	ft	
Froude Number	1.07		
Maximum Discharge	11.30	ft³/s	
Discharge Full	10.50	ft³/s	> Q required, therefore OK.
Slope Full	0.00939	ft/ft	
Flow Type	SuperCritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Average End Depth Over Rise	0.00	%
Normal Depth Over Rise	79.31	%
Downstream Velocity	Infinity	ft/s

Worksheet for Circular Pipe - 6in PVC capacity

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.010		
Channel Slope	0.01000	ft/ft	
Diameter	6	in	
Discharge	0.25	ft³/s	= Q required

Results

Normal Depth	0.20	ft	
Flow Area	0.07	ft²	
Wetted Perimeter	0.69	ft	
Hydraulic Radius	0.11	ft	
Top Width	0.49	ft	
Critical Depth	0.25	ft	
Percent Full	40.3	%	
Critical Slope	0.00459	ft/ft	
Velocity	3.37	ft/s	
Velocity Head	0.18	ft	
Specific Energy	0.38	ft	
Froude Number	1.53		
Maximum Discharge	0.78	ft³/s	
Discharge Full	0.73	ft³/s	> Q required, therefore OK.
Slope Full	0.00117	ft/ft	
Flow Type	SuperCritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Average End Depth Over Rise	0.00	%
Normal Depth Over Rise	40.34	%
Downstream Velocity	Infinity	ft/s

APPENDIX C

Master Drainage Plan by Plan Jeff Mortensen & Associates, Inc., dated 03/04/97 (JMA Report)

SUPPLEMENTARY INFORMATION

FOR

MILES ROAD S.E./KIRTLAND ADDITION

MASTER DRAINAGE PLAN

(REVISED MARCH, 1997)

Revised to Account for New Alignment of Mesita Rd. w/ Cul-de-Sac KO

TABLE OF CONTENTS

JEFFREY G. MORTENSEN NEW MEXICO REGISTERED PROFESSIONAL ENGINEER

03-04-97

JMA No. 950925

BASIN A-1 CALCULATIONS

Site Characteristics

1. Precipitation Zone = 2
2. $P_{6,100} = P_{360}$ = 2.35 in.
3. Total Area (A_T) 39,835/0.91
4. Existing Land Treatment

Treatment	Area (sf/ac)	%
C	39,835/0.91	100

5. Developed Land Treatment

Treatment	Area (sf/ac)	%
B	11,555/0.26	29
D	28,280/0.65	71

Existing Condition

1. Volume

$$E_W = (E_A A_A + E_B A_B + E_C A_C + E_D A_D)/A_T$$
$$E_W = 1.13 \text{ in.}$$
$$V_{100} = (E_W/12) A_T$$
$$V_{100} = (1.13/12)(39,835) = 3,750 \text{ cf}$$

2. Peak Discharge

$$Q_p = Q_{PA}A_A + Q_{PB}A_B + Q_{PC}A_C + Q_{PD}A_D$$
$$Q_p = Q_{100} = (3.14)(0.91) = 2.9 \text{ cfs}$$

Developed Condition

1. Volume

$$E_W = (E_A A_A + E_B A_B + E_C A_C + E_D A_D)/A_T$$
$$E_W = [(0.78)(0.26) + (2.12)(0.65)]/(0.91) = 1.74 \text{ in.}$$
$$V_{100} = (E_W/12) A_T$$
$$V_{100} = (1.74/12)(39,835) = 5,780 \text{ cf}$$

2. Peak Discharge

$$Q_p = Q_{PA}A_A + Q_{PB}A_B + Q_{PC}A_C + Q_{PD}A_D$$
$$Q_p = Q_{100} = (2.28)(0.26) + (4.70)(0.65) = 3.6 \text{ cfs}$$

Comparison

1. $\Delta V_{100} = 5,780 - 3,750 = 2,030$ cf (increase)
2. $\Delta Q_{100} = 3.6 - 2.9 = 0.7$ cfs (increase)

BASIN A-2 CALCULATIONS

Site Characteristics

1. Precipitation Zone = 2
2. $P_{6,100} = P_{360}$ = 2.35 in.
3. Total Area (A_T) 37,500/0.86
4. Existing Land Treatment

Treatment	Area (sf/ac)	%
C	37,500/0.86	100

5. Developed Land Treatment

Treatment	Area (sf/ac)	%
B	10,875/0.25	29
D	26,625/0.61	71

Existing Condition

1. Volume

$E_W = (E_A A_A + E_B A_B + E_C A_C + E_D A_D)/A_T$
E_W = 1.13 in.
$V_{100} = (E_W/12)A_T$
$V_{100} = (1.13/12)(37,500)$ = 3,530 cf

2. Peak Discharge

$Q_p = Q_{PA}A_A + Q_{PB}A_B + Q_{PC}A_C + Q_{PD}A_D$
$Q_p = Q_{100} = (3.14)(0.86)$ = 2.7 cfs

Developed Condition

1. Volume

$E_W = (E_A A_A + E_B A_B + E_C A_C + E_D A_D)/A_T$
$E_W = [(0.78)(0.25) + (2.12)(0.61)]/(0.86)$ = 1.73 in.
$V_{100} = (E_W/12)A_T$
$V_{100} = (1.73/12)(37,500)$ = 5,410 cf

2. Peak Discharge

$Q_p = Q_{PA}A_A + Q_{PB}A_B + Q_{PC}A_C + Q_{PD}A_D$
$Q_p = Q_{100} = (2.28)(0.25) + (4.70)(0.61)$ = 3.4 cfs

Comparison

1. ΔV_{100} = 5,410 - 3,530 = 1,880 cf (increase)
2. ΔQ_{100} = 3.4 - 2.7 = 0.7 cfs (increase)

BASIN B CALCULATIONS

Site Characteristics

1. Precipitation Zone = 2
2. $P_{6,100} = P_{360}$ = 2.35 in.
3. Total Area (A_T) 59,410/1.42
4. Existing Land Treatment

Treatment	Area (sf/ac)	%
C	59,410/1.42	100

5. Developed Land Treatment

Treatment	Area (sf/ac)	%
B	16,040/0.38	27
D	43,370/1.04	73

Existing Condition

1. Volume

$$E_W = (E_A A_A + E_B A_B + E_C A_C + E_D A_D)/A_T$$
$$E_W = 1.13 \text{ in.}$$
$$V_{100} = (E_W/12)A_T$$
$$V_{100} = (1.13/12)(59,410) = 5,590 \text{ cf}$$

2. Peak Discharge

$$Q_p = Q_{PA}A_A + Q_{PB}A_B + Q_{PC}A_C + Q_{PD}A_D$$
$$Q_p = Q_{100} = (3.14)(1.42) = 4.5 \text{ cfs}$$

Developed Condition

1. Volume

$$E_W = (E_A A_A + E_B A_B + E_C A_C + E_D A_D)/A_T$$
$$E_W = [(0.78)(0.38) + (2.12)(1.04)]/(1.42) = 1.76 \text{ in.}$$
$$V_{100} = (E_W/12)A_T$$
$$V_{100} = (1.76/12)(59,410) = 8,710 \text{ cf}$$

2. Peak Discharge

$$Q_p = Q_{PA}A_A + Q_{PB}A_B + Q_{PC}A_C + Q_{PD}A_D$$
$$Q_p = Q_{100} = (2.28)(0.38) + (4.70)(1.04) = 5.8 \text{ cfs}$$

Comparison

1. $\Delta V_{100} = 8,710 - 5,590 = 3,120$ cf (increase)
2. $\Delta Q_{100} = 5.8 - 4.5 = 1.3$ cfs (increase)

BASIN C CALCULATIONS

Site Characteristics

1. Precipitation Zone = 2
2. $P_{6,100} = P_{360}$ = 2.35 in.
3. Total Area (A_T) 59,100/1.36
4. Existing Land Treatment

Treatment	Area (sf/ac)	%
C	57,330/1.32	97
D	1,770/0.04	03

5. Developed Land Treatment

Treatment	Area (sf/ac)	%
B	7,680/0.18	13
C	9,460/0.22	16
D	41,960/0.96	71

Existing Condition

1. Volume

$$E_W = (E_A A_A + E_B A_B + E_C A_C + E_D A_D)/A_T$$
$$E_W = [(1.13)(1.32) + (2.12)(0.04)]/(1.36) = 1.16 \text{ in.}$$
$$V_{100} = (E_W/12)A_T$$
$$V_{100} = (1.16/12)(59,100) = 5,710 \text{ cf}$$

2. Peak Discharge

$$Q_p = Q_{PA}A_A + Q_{PB}A_B + Q_{PC}A_C + Q_{PD}A_D$$
$$Q_p = Q_{100} = (3.14)(1.32) + (4.70)(0.04) = 4.3 \text{ cfs}$$

Developed Condition

1. Volume

$$E_W = (E_A A_A + E_B A_B + E_C A_C + E_D A_D)/A_T$$
$$E_W = [(0.78)(0.18) + (1.13)(0.22) + (2.12)(0.96)]/(1.36) = 1.78 \text{ in.}$$
$$V_{100} = (E_W/12)A_T$$
$$V_{100} = (1.78/12)(59,100) = 8,770 \text{ cf}$$

2. Peak Discharge

$$Q_p = Q_{PA}A_A + Q_{PB}A_B + Q_{PC}A_C + Q_{PD}A_D$$
$$Q_p = Q_{100} = (2.28)(0.18) + (3.14)(0.22) + (4.70)(0.96) = 5.6 \text{ cfs}$$

Comparison

1. $\Delta V_{100} = 8,770 - 5,710 = 3,060$ cf (increase)
2. $\Delta Q_{100} = 5.6 - 4.3 = 1.3$ cfs (increase)

BASIN DE-1 CALCULATIONS

Site Characteristics

1. Precipitation Zone = 2
2. $P_{6,100} = P_{360}$ = 2.35 in.
3. Total Area (A_T) 52,950/1.21
4. Existing Land Treatment

Treatment	Area (sf/ac)	%
C	52,950/1.21	100

5. Developed Land Treatment (Assume will develop like Tract C)

Treatment	Area (sf/ac)	%
B	7,420/0.16	14
C	9,530/0.22	18
D	36,000/0.83	68

Existing Condition

1. Volume

$E_W = (E_A A_A + E_B A_B + E_C A_C + E_D A_D)/A_T$
$E_W = 1.13$ in.
$V_{100} = (E_W/12)A_T$
$V_{100} = (1.13/12)(52,950) = 4,990$ cf

2. Peak Discharge

$Q_p = Q_{PA}A_A + Q_{PB}A_B + Q_{PC}A_C + Q_{PD}A_D$
$Q_p = Q_{100} = (3.14)(1.21) = 3.8$ cfs

Developed Condition

1. Volume

$E_W = (E_A A_A + E_B A_B + E_C A_C + E_D A_D)/A_T$
$E_W = [(0.78)(0.16)+(1.13)(0.22)+(2.12)(0.83)]/(1.21) = 1.76$ in.
$V_{100} = (E_W/12)A_T$
$V_{100} = (1.76/12)(52,950) = 7,770$ cf

2. Peak Discharge

$Q_p = Q_{PA}A_A + Q_{PB}A_B + Q_{PC}A_C + Q_{PD}A_D$
$Q_p = Q_{100} = (2.28)(0.16)+(3.14)(0.22)+(4.70)(0.83) = 5.0$ cfs

Comparison

1. $\Delta V_{100} = 7,770 - 4,990 = 2,780$ cf (increase)
2. $\Delta Q_{100} = 5.0 - 3.8 = 1.2$ cfs (increase)

BASIN DE-2 CALCULATIONS

Site Characteristics

1. Precipitation Zone = 2
2. $P_{6,100} = P_{360}$ = 2.35 in.
3. Total Area (A_T) 5,000/0.11
4. Existing Land Treatment

Treatment	Area (sf/ac)	%
C	5,000/0.11	100

5. Developed Land Treatment

Treatment	Area (sf/ac)	%
D	5,000/0.11	100

Existing Condition

1. Volume

$$E_W = (E_A A_A + E_B A_B + E_C A_C + E_D A_D)/A_T$$
$$E_W = 1.13 \text{ in.}$$
$$V_{100} = (E_W/12)A_T$$
$$V_{100} = (1.13/12)(5,000) = 470 \text{ cf}$$

2. Peak Discharge

$$Q_p = Q_{PA}A_A + Q_{PB}A_B + Q_{PC}A_C + Q_{PD}A_D$$
$$Q_p = Q_{100} = (3.14)(0.11) = 0.3 \text{ cfs}$$

Developed Condition

1. Volume

$$E_W = (E_A A_A + E_B A_B + E_C A_C + E_D A_D)/A_T$$
$$E_W = 2.12 \text{ in.}$$
$$V_{100} = (E_W/12)A_T$$
$$V_{100} = (2.12/12)(5,000) = 880 \text{ cf}$$

2. Peak Discharge

$$Q_p = Q_{PA}A_A + Q_{PB}A_B + Q_{PC}A_C + Q_{PD}A_D$$
$$Q_p = Q_{100} = (4.70)(0.11) = 0.5 \text{ cfs}$$

Comparison

1. $\Delta V_{100} = 880 - 470 = 410$ cf (increase)
2. $\Delta Q_{100} = 0.5 - 0.3 = 0.2$ cfs (increase)

BASIN F-1 CALCULATIONS

Site Characteristics

1. Precipitation Zone = 2
2. $P_{6,100} = P_{360}$ = 2.35 in.
3. Total Area (A_T) 39,550/0.91
4. Existing Land Treatment

Treatment	Area (sf/ac)	%
C	39,550/0.91	100

5. Developed Land Treatment (Same land treatments as M15/D22, IP Use)

Treatment	Area (sf/ac)	%
B	9,100/0.21	23
D	30,450/0.70	77

Existing Condition

1. Volume

$$E_W = (E_A A_A + E_B A_B + E_C A_C + E_D A_D)/A_T$$
$$E_W = 1.13 \text{ in.}$$
$$V_{100} = (E_W/12)A_T$$
$$V_{100} = (1.13/12)(39,550) = 3,720 \text{ cf}$$

2. Peak Discharge

$$Q_p = Q_{PA}A_A + Q_{PB}A_B + Q_{PC}A_C + Q_{PD}A_D$$
$$Q_p = Q_{100} = (3.14)(0.91) = 2.9 \text{ cfs}$$

Developed Condition

1. Volume

$$E_W = (E_A A_A + E_B A_B + E_C A_C + E_D A_D)/A_T$$
$$E_W = [(0.78)(0.21)+(2.12)(0.70)]/(0.91) = 1.81 \text{ in.}$$
$$V_{100} = (E_W/12)A_T$$
$$V_{100} = (1.81/12)(39,550) = 5,970 \text{ cf}$$

2. Peak Discharge

$$Q_p = Q_{PA}A_A + Q_{PB}A_B + Q_{PC}A_C + Q_{PD}A_D$$
$$Q_p = Q_{100} = (2.28)(0.21)+(4.70)(0.70) = 3.8 \text{ cfs}$$

Comparison

1. ΔV_{100} = 5,97 - 3,720 = 2,250 cf (increase)
2. ΔQ_{100} = 3.8 - 2.9 = 0.9 cfs (increase)

BASIN F-2 CALCULATIONS

Site Characteristics

1. Precipitation Zone = 2
2. $P_{6,100} = P_{360}$ = 2.35 in.
3. Total Area (A_T) 29,250/0.67
4. Existing Land Treatment

Treatment	Area (sf/ac)	%
C	29,250/0.67	100

5. Developed Land Treatment (Same land treatments as M15/D22, IP Use)

Treatment	Area (sf/ac)	%
B	6,730/0.15	23
D	22,520/0.52	77

Existing Condition

1. Volume

$$E_W = (E_A A_A + E_B A_B + E_C A_C + E_D A_D)/A_T$$
$$E_W = 1.13 \text{ in.}$$
$$V_{100} = (E_W/12)A_T$$
$$V_{100} = (1.13/12)(29,250) = 2,750 \text{ cf}$$

2. Peak Discharge

$$Q_p = Q_{PA}A_A + Q_{PB}A_B + Q_{PC}A_C + Q_{PD}A_D$$
$$Q_p = Q_{100} = (3.14)(0.67) = 2.1 \text{ cfs}$$

Developed Condition

1. Volume

$$E_W = (E_A A_A + E_B A_B + E_C A_C + E_D A_D)/A_T$$
$$E_W = [(0.78)(0.15) + (2.12)(0.52)]/(0.67) = 1.82 \text{ in.}$$
$$V_{100} = (E_W/12)A_T$$
$$V_{100} = (1.82/12)(29,250) = 4,440 \text{ cf}$$

2. Peak Discharge

$$Q_p = Q_{PA}A_A + Q_{PB}A_B + Q_{PC}A_C + Q_{PD}A_D$$
$$Q_p = Q_{100} = (2.28)(0.15) + (4.70)(0.52) = 2.8 \text{ cfs}$$

Comparison

1. $\Delta V_{100} = 4,440 - 2,750 = 1,690$ cf (increase)
2. $\Delta Q_{100} = 2.8 - 2.1 = 0.7$ cfs (increase)

BASIN M CALCULATIONS

Site Characteristics

1. Precipitation Zone = 2
2. $P_{6,100} = P_{360}$ = 2.35 in.
3. Total Area (A_T) 58,550/1.34
4. Existing Land Treatment

Treatment	Area (sf/ac)	%
C	58,550/1.34	100

5. Developed Land Treatment

Treatment	Area (sf/ac)	%
B	7,025/0.16	12
D	51,525/1.18	88

Existing Condition

1. Volume

$E_W = (E_A A_A + E_B A_B + E_C A_C + E_D A_D)/A_T$
E_W = 1.13 in.
$V_{100} = (E_W/12)A_T$
$V_{100} = (1.13/12)(58,550)$ = 5,510 cf

2. Peak Discharge

$Q_p = Q_{PA}A_A + Q_{PB}A_B + Q_{PC}A_C + Q_{PD}A_D$
$Q_p = Q_{100} = (3.14)(1.34)$ = 4.2 cfs

Developed Condition

1. Volume

$E_W = (E_A A_A + E_B A_B + E_C A_C + E_D A_D)/A_T$
$E_W = [(1.13)(0.16)+(2.12)(1.18)]/(1.34)$ = 2.00 in.
$V_{100} = (E_W/12)A_T$
$V_{100} = (2.00/12)(58,550)$ = 9,760 cf

2. Peak Discharge

$Q_p = Q_{PA}A_A + Q_{PB}A_B + Q_{PC}A_C + Q_{PD}A_D$
$Q_p = Q_{100} = (3.14)(0.16)+(4.70)(1.18)$ = 6.0 cfs

Comparison

1. ΔV_{100} = 9,760 - 5,510 = 4,250 cf (increase)
2. ΔQ_{100} = 6.0 - 4.2 = 1.8 cfs (increase)

ALLEY CALCULATIONS

Site Characteristics

1. Precipitation Zone = 2
2. $P_{6,100} = P_{360}$ = 2.35 in.
3. Total Area (A_T) 3,850/0.09
4. Existing Land Treatment

Treatment	Area (sf/ac)	%
C	3,850/0.09	100

5. Developed Land Treatment

Treatment	Area (sf/ac)	%
D	3,850/0.09	100

Existing Condition

1. Volume

$E_W = (E_A A_A + E_B A_B + E_C A_C + E_D A_D)/A_T$
$E_W = 1.13$ in.
$V_{100} = (E_W/12)A_T$
$V_{100} = (1.13/12)(3,850) = 360$ cf

2. Peak Discharge

$Q_p = Q_{PA}A_A + Q_{PB}A_B + Q_{PC}A_C + Q_{PD}A_D$
$Q_p = Q_{100} = (3.14)(0.09) = 0.3$ cfs

Developed Condition

1. Volume

$E_W = (E_A A_A + E_B A_B + E_C A_C + E_D A_D)/A_T$
$E_W = 2.12$ in.
$V_{100} = (E_W/12)A_T$
$V_{100} = (2.12/12)(3,850) = 680$ cf

2. Peak Discharge

$Q_p = Q_{PA}A_A + Q_{PB}A_B + Q_{PC}A_C + Q_{PD}A_D$
$Q_p = Q_{100} = (4.70)(0.09) = 0.4$ cfs

Comparison

1. $\Delta V_{100} = 680 - 360 = 320$ cf (increase)
2. $\Delta Q_{100} = 0.4 - 0.3 = 0.1$ cfs (increase)

OFFSITE BASIN CALCULATIONS

Site Characteristics

1. Precipitation Zone = 2
2. $P_{6,100} = P_{360}$ = 2.35 in.
3. Total Area (A_T) 27,300/0.63
4. Developed Land Treatment

Treatment	Area (sf/ac)	%
C	6,800/0.16	25
D	20,500/0.47	75

Developed Condition

1. Volume

$$E_W = (E_A A_A + E_B A_B + E_C A_C + E_D A_D)/A_T$$
$$E_W = [(1.13)(0.16) + (2.12)(0.47)]/(0.63) = 1.87 \text{ in.}$$
$$V_{100} = (E_W/12)A_T$$
$$V_{100} = (1.87/12)(27,300) = 4,250 \text{ cf}$$

2. Peak Discharge

$$Q_p = Q_{PA}A_A + Q_{PB}A_B + Q_{PC}A_C + Q_{PD}A_D$$
$$Q_p = Q_{100} = (3.14)(0.16) + (4.70)(0.47) = 2.7 \text{ cfs}$$

HYDRAULIC CALCULATIONS
TEMPORARY RUN-DOWN

1. Curb Opening Width

$Q = CLH^{3/2}$

Let:
C = 2.7
Q = 20.4 cfs
H = 0.67 ft (8" curb height)

Therefore: L = 13.77 ft.
Use 16.0 foot design width for future inlet construction

2. Minimum Channel Width
Using Manning's Equation

Let:
n = 0.013
Q = 20.4 cfs
s = 0.0850 ft/ft
d = 0.67 ft (8" curb height)

Therefore: W = 1.74 ft.
Use 10' design width to satisfy C.O.A. requirements

3. Velocity Multiplied by Depth Calculation
Use Manning's Equation

A. 10-Year Storm Event

Let: $Q_{10} = 0.67\ Q_{100} = 0.67(20.4) = 13.7$ cfs

W = 10.0 ft.
S = 0.0850 ft/ft
n = 0.013

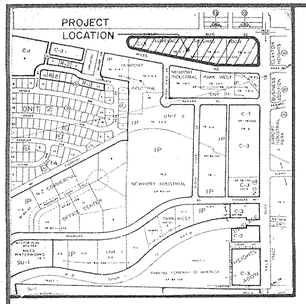
Therefore:
d = 0.15 ft
V = 9.1 ft/s
Vd = 1.36 < 6.5 (per D.P.M.)

B. 100-Year Storm Event

Let: $Q = Q_{100} = 20.4$ cfs

W = 10.0 ft.
S = 0.0850 ft/ft
n = 0.013

Therefore:
d = 0.19 ft
V = 10.7 ft/s
Vd = 2.03 < 6.5 (per D.P.M.)



VICINITY MAP M-15

SCALE: 1" = 800'

PROJECT BENCHMARK

AC 3 1-M16, BRASS CAP SET FLUSH IN PAVEMENT 16 FEET SOUTH OF THE INTERSECTION OF YALE BLVD. & GIBSON BLVD.
ELEVATION: 5189.85 FEET. (M.S.L.D.)

T.B.M.

CHISELED "U" ON S.W. CORNER OF CONCRETE SLAB AS SHOWN HEREON.
ELEVATION: 5168.33 FEET (M.S.L.D.)

LEGAL DESCRIPTION

TRACTS A-1, B-1, & C-1
LOVELACE HEIGHTS ADDITION

LEGEND

- EXISTING SPOT ELEVATION
- PROPOSED SPOT ELEVATION
- 5155 — EXISTING CONTOUR
- 56 — PROPOSED CONTOUR
- EXISTING CONCRETE
- PROPOSED CONCRETE
- PROPOSED ASPHALT
- F.L. FLOW LINE
- T.C. TOP OF CURB
- (TC 83.53) FUTURE TOP OF CURB ELEV. (PROJECT NO. 2834)
- PROPERTY LINE
- FUTURE CURB + GUTTER (PROJECT NO. 2834)
- EXISTING SWALE
- PROPOSED SWALE
- PROPOSED CHAIN LINK FENCE
- BASIN BOUNDARY

DRAINAGE PLAN

The following items concerning the U.S.A. Park Parking Lot Drainage Plan are contained hereon:

1. Vicinity Map
2. Grading Plan
3. Calculations

As shown by the Vicinity Map, this site is located on the southwest corner of the intersection of Yale Boulevard S.E. and Gibson Boulevard S.E. As shown by Plate H-15 of the Albuquerque Master Drainage Study, (AMDS), this site does not lie within a designated Flood Hazard Zone, and downstream flooding is not apparent and does not appear to be a problem. At present, the majority of the site flows from east to west to an existing 24" RCP pipe extension which also connects to an existing storm inlet located in Gibson Boulevard S.E. The storm inlet drains directly onto private property located on the north side of Gibson Boulevard S.E. A portion of the site flows from east to west onto Gibson Boulevard S.E. No offsite flows enter the site along the north and east property lines since the adjacent streets route runoff away from the project site. No offsite flows enter the site along the west property line since the adjacent lot is graded in a manner which will route runoff away from the project site. No offsite flows enter the site along the south property line since the existing bar ditch along the north side of the Miles Road S.E. right-of-way routes runoff away from the project site. Also, when the future Miles Road S.E. is constructed (City Project No. 2834), the runoff generated will be routed away from the project site by the proposed street improvements.

The Grading Plan shows 1) existing and proposed grades indicated by spot elevations and contours at 1'0" intervals, 2) continuity between existing and proposed grades, and 3) the limit and character of the proposed improvements. As shown by this Plan, the proposed improvements consist of the construction of a new parking lot along with adjacent landscaping. Flows generated by the proposed improvements will be routed from east to west and discharged onto Gibson Boulevard S.E. via a series of multiple sidewalk culverts. From that point, the runoff will flow west along the south edge of Gibson Boulevard S.E. to an existing concrete rundown located above the South Diversion Channel which is located roughly about 3500 feet to the west of the project site which is the outfall for this site. A previous submittal by Fred C. Arfman has substantiated the capacity of this existing downstream facility. Based upon the fact that this site is an infill site, is located at the lower end of the watershed, and the proximity of downstream facilities, the free discharge of runoff from this site is appropriate. Furthermore, the proposed drainage pattern will eliminate the problem of directly discharging to the aforementioned storm inlet and hence will improve the existing drainage condition by decreasing the amount of runoff discharged onto the private property. Also, there would be no benefit in ponding and discharging at a controlled rate since this would only increase the amount of runoff discharged into an existing storm inlet located roughly about 340 feet to the west of the project site by extending the duration of low-flow runoff. This inlet also discharges onto private property. Some of the runoff generated by this site and discharged to the street may be captured by the existing downstream inlet. However, this is an existing inlet in an existing problem (i.e., the discharge of public runoff onto private property). The proposed parking lot will improve these conditions by bypassing the first inlet altogether.

Also, there is already sufficient flows within Gibson Boulevard S.E. such that the increase in runoff from this site will have no affect on the downstream inlet capacity (i.e. the inlet can only accept so much runoff). Based upon DPM Plate 22.3D-6, the inlet capacity is only about 18 cfs. The upstream contribution is approximately 284 cfs as determined by the AMDS update (draft only). Consequently, the proposed paving will not affect the inlet capacity, nor will it increase the runoff presently being discharged onto private property.

The Calculations which appear hereon analyze both the existing and developed conditions for the 100-year, 6-hour rainfall event. The Rational Method has been used to quantify the peak rate of discharge and the SCS Method has been used to quantify the volume of runoff. Both methods have been used in accordance with the City of Albuquerque Development Process Manual, Volume II and the Mayor's Emergency Rule adopted January 14, 1986. As shown by these Calculations, the proposed improvements will increase the total discharge from this site by approximately 18.2 cfs.

CALCULATIONS

Ground Cover Information

From SCS Bernalillo County Soil Survey,
Plate 31: WaB, Wink fine sandy loam
Hydrologic Soil Group: B
Existing Pervious CN = 70 (DPM Plate 22.2 C-2
Developed Pervious CN = 61 (DPM Plate 22.2 C-2)

Time of Concentration/Time to Peak

$T_c = 0.0078\ l^{0.77}/s^{0.385}$ (Kirpich Equation)

$T_p = T_c = 10$ min.

Point Rainfall

$P_6 = 2.3$ in. (DPM Plate 22.2 D-1)

Rational Method (100 YEAR RAINFALL)

Discharge: Q = CiA

where C varies
$i = P_6\ (6.84)\ T_c^{-0.51} = 4.86$ in/hr
$P_6 = 2.3$ in (DPM Plate 22.2D-1)
$T_c = 10$ min (minimum)
A = area, acres

SCS Method

Volume: V = 3630(DRO) A

Where DRO = Direct runoff in inches
A = area, acres

Existing Condition

Atotal = 312,865 sf = 7.18 Ac
C = 0.40 (Weighted average per Emergency Rule, 1/14/86)
$Q_{100} = CiA = (0.40)(4.86)(7.18) = 14.0$ cfs
Aimp = -0- sf; % impervious = -0- %
Composite CN = 70 (DPM Plate 22.2 C-3)
DRO = 0.4 in (DPM Plate 22.2 C-4)
$V_{100} = 3630\ (DRO)A = 10,425$ cf

Developed Condition

Basin 1
Atotal = 214,600 sf = 4.92 Ac
Roof area = 3200 sf (0.01)
Paved area = 205,400 sf (0.96)
Landscaped area = 6,000 sf (0.03)
C = 0.88 (Weighted average per Emergency Rule, 1/14/86)
$Q_{100} = CiA = (0.88)(4.86)(4.12) = 21.0$ cfs
Aimp = 208,600 sf; % impervious = 97 %
Composite CN = 97 (DPM Plate 22.2 C-3)
DRO = 1.9 in (DPM Plate 22.2 C-4)
$V_{100} = 3630\ (DRO)\ A = 33,930$ cf

Developed Condition

Basin 2
Atotal = 46,075 sf = 1.06 Ac
Paved area = 46,075 sf (1.0)
Landscaped area = -0- sf (0.00)
C = 0.95 Pasture or Range Land: fair condition)
$Q_{100} = CiA = (0.95)(4.86)(1.06) = 4.9$ cfs
Aimp = 46,075 sf; % impervious = 100 %
Composite CN = 98 (DPM Plate 22.2 C-3)
DRO = 2.2 in (DPM Plate 22.2 C-4)
$V_{100} = 3630\ (DRO)\ A = 8465$ cf

Developed Condition

Basin 3
Atotal = 59,040 sf = 1.36 Ac
Paved area = 59,040 sf (1.0)
Landscaped area = -0- sf (0.00)
C = 0.95 (Weighted average per Emergency Rule, 1/14/86)
$Q_{100} = CiA = (0.95)(4.86)(1.36) = 6.3$ cfs
Aimp = 59,040 sf; % impervious = 100 %
Composite CN = 98 (DPM Plate 22.2 C-3)
DRO = 2.2 in (DPM Plate 22.2 C-4)
$V_{100} = 3630\ (DRO)\ A = 10,860$ cf

Channel Cpacity (2'-0" curb opening) Weir Equation

$Q = CLH^{3/2} = 3.4$ cfs
Where C = 3.09
L = 2'-0"
H = 0.67 feet

REVISED SIDEWALK CULVERT SIZING

Rational Method

Discharge: Q = CiA

where C varies
$i = P_6\ (6.84)\ T_c^{-0.51} = 3.19$ in/hr
$P_6 = 1.51$ in (DPM Plate 22.2D-1 - 10 Year Rainfall)
$T_c = 10$ min (minimum)
A = area, acres

Developed Condition (10 Year Rainfall)

Basin 1
C = 0.93 (Weighted average per Emergency Rule, 1/14/86)
$Q_{100} = CiA = (0.93)(3.19)(4.92) = 14.5$ cfs

Basin 2
C = 0.95 (Weighted average per Emergency Rule, 1/14/86)
$Q_{100} = CiA = (0.95)(3.19)(1.06) = 3.2$ cfs

Basin 3
C = 0.95 (Weighted average per Emergency Rule, 1/14/86)
$Q_{100} = CiA = (0.95)(3.19)(1.36) = 4.1$ cfs

Therefore, use 4 - 2'0" and 1 - 1'0" sidewalk culverts (Basin 1)
1 - 2'0" sidewalk culvert (Basin 2)
2 - 2'0" sidewalk culverts (Basin 3)

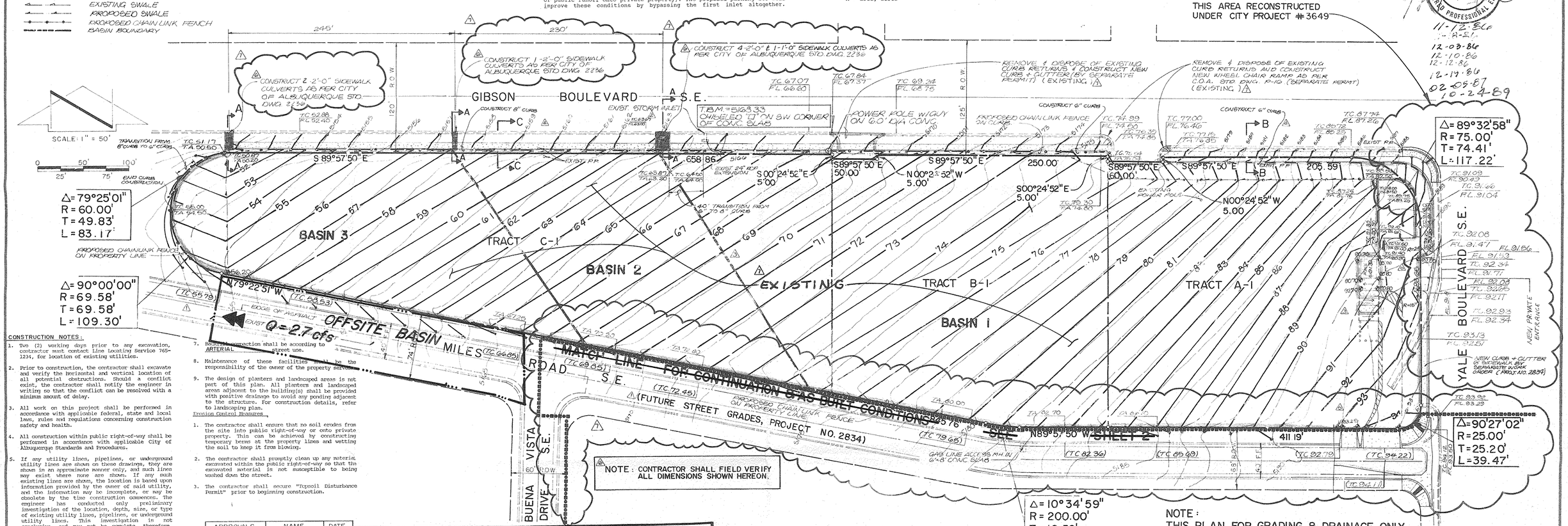
Comparison

$\Delta Q_{100} = (21.0 + 4.9 + 6.3) - 14.0 = 18.2$ cfs (increase)
$\Delta V_{100} = (33,930 + 8465 + 10,860) - 10,425 = 42,830$ cf (increase)

JEFFREY G. MORTENSEN NEW MEXICO 8547 REGISTERED PROFESSIONAL ENGINEER

11-12-86
12-03-86
12-10-86
12-12-86
12-19-86
02-05-87
10-24-89

THIS AREA RECONSTRUCTED UNDER CITY PROJECT #3649



CONSTRUCTION NOTES:

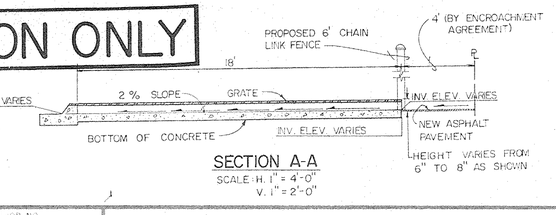
1. Two (2) working days prior to any excavation, contractor must contact Line Locating Service 765-1234, for location of existing utilities.
2. Prior to construction, the contractor shall excavate and verify the horizontal and vertical location of all potential obstructions. Should a conflict exist, the contractor shall notify the engineer in writing so that the conflict can be resolved with a minimum amount of delay.
3. All work on this project shall be performed in accordance with applicable federal, state and local laws, rules and regulations concerning construction safety and health.
4. All construction within public right-of-way shall be performed in accordance with applicable City of Albuquerque Standards and Procedures.
5. If any utility lines, pipelines, or underground utility lines are shown on these drawings, they are shown in an approximate manner only, and such lines may exist where none are shown. If any such existing lines are shown, the location is based upon information provided by the owner of said utility, and the information may be incomplete, or may be obsolete by the time construction commences. The engineer has conducted only preliminary investigation of the location, depth, size, or type of existing utility lines, pipelines, or underground utility lines. This investigation is not conclusive, and may not be complete, therefore, makes no representation pertaining thereto, and assumes no responsibility or liability therefor. The contractor shall inform itself of the location of any utility line, pipeline, or underground utility line in or near the area of the work in advance of and during excavation work. The contractor is fully responsible for any and all damage caused by its failure to locate, identify and preserve any and all existing utilities, pipelines, and underground utility lines. In planning and conducting excavation, the contractor shall comply with state statutes, municipal and local ordinances, rules and regulations, if any, pertaining to the location of these lines and facilities.
6. An Excavation/Construction Permit will be required before beginning any work within city right-of-way. An approved copy of these plans must be submitted at the time of application for this permit.
7. Street construction shall be according to ARTERIAL street use.
8. Maintenance of these facilities shall be the responsibility of the owner of the property served.
9. The design of planters and landscaped areas is not part of this plan. All planters and landscaped areas adjacent to the building(s) shall be provided with positive drainage to avoid any ponding adjacent to the structure. For construction details, refer to landscaping plan.

Erosion Control Measures

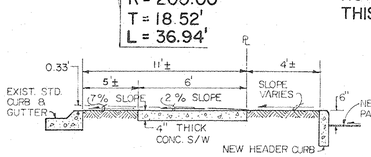
1. The contractor shall ensure that no soil erodes from the site into public right-of-way or onto private property. This can be achieved by constructing temporary berms at the property lines and wetting the soil to keep it from blowing.
2. The contractor shall promptly clean up any material excavated within the public right-of-way so that the excavated material is not susceptible to being washed down the street.
3. The contractor shall secure "Topsoil Disturbance Permit" prior to beginning construction.

APPROVALS	NAME	DATE
A.C.E. / DESIGN		
INSPECTOR		
A.C.E. / FIELD		

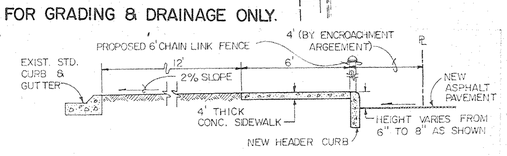
FOR INFORMATION ONLY



SECTION A-A
SCALE: H. 1" = 4'-0"
V. 1" = 2'-0"



SECTION B-B
SCALE: V. 1" = 3'-0"
H. 1" = 4'-0"



SECTION C-C
SCALE: V. 1" = 3'-0"
H. 1" = 4'-0"

NOTE:
THIS PLAN FOR GRADING & DRAINAGE ONLY.

NO	DATE	BY	REVISIONS
5	10/89	JGM	UPDATE FOR PARKING LOT EXPANSION; S.O. #19 APPROVAL
4	01/87	LPU	SIZE NO. OF SIDEWALK CULVERTS NEEDED FOR ALL BASINS BASED ON A 10 YEAR RAINFALL + SECTION B-B
3	12/86	LPU	REVISED ENTRANCES TO PARKING LOT
2	12/86	LPU	REVISE CURB, SIDEWALK & DRIVEPAD LOCATIONS.
1	12/86	LPU	REVISED GRADES, TEXT, CALCULATIONS & ENTRANCES
	11/86	LPU	SHOW FUTURE CURB ELEVATIONS ON MILES RD.
	1/86	LPU	REMOVED PROPOSED STORM INLET + REVISED CALCS.

JEFF MORTENSEN & ASSOCIATES, INC.
811 DALLAS, N.E. - ALBUQUERQUE, NM 87110
ENGINEERS - TELEPHONE (505) 265-5611

DESIGNED BY L.P.U.
DRAWN BY R.A.R.
APPROVED J.G.M.
JOB NO. 60157
DATE 10-89

GRADING AND DRAINAGE PLAN
EXPRESS AIRPORT PARKING

FILE NO.
SHEET 1 OF

LEGAL DESCRIPTION

LOT 9, BLOCK A, KIRTLAND ADDITION
LOT 1, BLOCK A, KIRTLAND ADDITION
UNPLATTED PARCELS
TRACT 4-A-1, NEWPORT INDUSTRIAL PARK-WEST, UNIT 2
MILES RD. S.E., WALKER DRIVE S.E., AND PUBLIC ALLEYS

PROJECT BENCHMARK = T.B.M.

CITY OF ALBUQUERQUE BENCHMARK "G-9A".
STATION IS STAMPED "ACS,G-9A, 1984", A 3 1/4" ALUMINUM TABLET SET FLUSH IN THE CONCRETE GUTTER. TO REACH THE STATION FROM I-25, TRAVEL EAST ON GIBSON BLVD. 0.7 MILES. AS SHOWN BELOW.
ELEVATION = 5143.07' (M.S.L.D.)

LEGEND

TC TOP OF CURB
FL FLOW LINE
NG NATURAL GROUND
SPOT ELEVATION
GUY WIRE/ANCHOR
POWER POLE
LIGHT POLE
MANHOLE
CONTROL POINT

EXISTING CONTOUR
PROPOSED CONTOUR
PROPOSED SPOT ELEVATION
PROPOSED FLOWLINE
ATTENUATED FLOW RATE
BASIN BOUNDARY
FUTURE STORM DRAIN
PROPOSED RETAINING WALL

NOTE: THIS IS NOT A BOUNDARY SURVEY. APPARENT PROPERTY CORNERS ARE SHOWN FOR ORIENTATION ONLY.

SCALE: 1" = 40'

PROJECT BENCHMARK=T.B.M.
FD. BRASS CAP STAMPED "G-9A"
ELEVATION=5143.07

GIBSON BOULEVARD S.E.
(RIGHT-OF-WAY VARIES)

136' RIGHT-OF-WAY
120' RIGHT-OF-WAY
TWO 15" SIDEWALK CULVERTS PER C.O.A. STD. DWG. 2236
48" RCP
42" RCP
TRAFFIC CONTROL
TRAFFIC SIGNAL
CONCRETE PAVING
EDGE OF CONCRETE
TRAFFIC SIGNAL W/MAST ARM
TEL M.H.
FD. REBAR W/ CAP
FUTURE CONNECTION TO BACK OF INLET
18"x 18" DRAINAGE INLET TG=91.2
6" PVC (SDR-35) STORM DRAIN @ S=0.0200 DAYLIGHTED INTO SIDEWALK CULVERT (INTERIM)
24" SIDEWALK CULVERT PER C.O.A. STD. DWG. 2236 (INTERIM)
392.91'
MAX. W.S.L. 5092.0
PROPOSED BUILDING
FUTURE BUILDING ADDITION
LANDSCAPING
PROPOSED CAR WASH
PRIVATE ACCESS ROAD
18"x 18" DRAINAGE INLET TG=91.2
N 00°04'27" E
24" SIDEWALK CULVERT PER C.O.A. STD. DWG. 2236
6" PVC (SDR-35) STORM DRAIN @ S=0.0200 DAYLIGHTED INTO SIDEWALK CULVERT
N 89°55'33" W 258.46'
20' PUBLIC ALLEY (1)
FD. REBAR W/CAP
UNIVERSITY BOULEVARD S.E. (80' R.O.W.)
WELLS ST. S.E. (50' R.O.W.)
LOVELACE ROAD S.E. (50' R.O.W.)

Q_{100} = 2.7 cfs (FUTURE)
Q_{100} = 3.6 cfs
Q_{100} = 20.4cfs TOTAL
Q_{100} = 6.0 cfs
Q_{100} = 5.6 cfs
Q_{100} = 5.0 cfs
Q_{100} = 2.7 cfs
Q_{100} = 3.8 cfs
Q_{100} = 2.7 cfs
Q_{100} = 0.4 cfs ALLEY
q= 0.8 cfs (INTERIM)
q= 0.8 cfs
Q_{100} = 0.5 cfs
q= 0.8 cfs

BASIN A-1
BASIN A-2
BASIN B
BASIN C
BASIN M
BASIN DE-1
BASIN DE-2
BASIN F-1
BASIN F-2
BASIN G

LOT 1, BLOCK A KIRTLAND ADDITION
LOT 9, BLOCK A KIRTLAND ADDITION
MILES ROAD S.E. (IMPROVED)
C.O.A. PROJECT #5319.90
WALKER DRIVE S.E.
MILES ROAD S.E. EXCESS R.O.W. TO BE VACATED
EXISTING OFFICE BUILDING F.F. = 5131.70
UNPLATTED PARCEL
UNPLATTED PARCEL
FORMER WALKER DRIVE S.E. (TO BE VACATED)
20' PUBLIC ALLEY VACATED BY V-95-6
S 89°43'39" E 710.80'
N 79°07'43" W 356.37'
N 10°52'17" E 100.18'
N 79°11'11" W 479.40'
S 00°06'19" W 253.86'
TRACT 4-A-1 NEWPORT INDUSTRIAL PARK - WEST UNIT 2
DETENTION POND
END OF CURB
WATER VALVE BOX
SAS MANHOLE
4" CURB PENETRATION PER C.O.A. STD. DWG. 2235
CURB HEIGHT CHANGE
6' CHAINLINK FENCE
ASPHALT PAVING SERVICE POLE WITH LIGHT

KEYED NOTES:

(1) EXISTING 5' UTILITY AND DRAINAGE EASEMENT RESERVED BY COVENANTS, CONDITIONS, RESTRICTIONS, TERMS, PROVISIONS AND EASEMENTS RECORDED IN BOOK D255, PAGE 312, RECORDS OF BERNALILLO COUNTY, NEW MEXICO, TO BE VACATED.

(2) EXISTING DOWN GUY EASEMENT PER UNRECORDED SURVEY PREPARED BY ANDREWS, ASBURY & ROBERT, INC. DATED JANUARY 02, 1979.

DETENTION TIME CALCULATIONS

A. BASIN B
$q_{avg} = 1/2\ q_{100} = (1/2)(0.8) = 0.4$ cfs
$V_{100} = 7{,}880$
TIME TO DRAIN = $\frac{(7{,}880\ cf)}{(0.4\ cfs)}$ (1 min./60 sec.)(1 hr./60 min.) = 5.5 hours

B. BASIN F-2
$q_{avg} = 1/2\ q_{100} = (1/2)(0.8) = 0.4$ cfs
$V_{100} = 4{,}440$ cf
TIME TO DRAIN = $\frac{(4{,}440\ cf)}{(0.4\ cfs)}$ (1 min./60 sec.)(1 hr./60 min.) = 3.1 hours

THIS PLAN REPLACES THE PREVIOUS PLAN DATED 02-15-1996.

JEFFREY G. MORTENSEN NEW MEXICO 8547 REGISTERED PROFESSIONAL ENGINEER
03-04-97

JEFF MORTENSEN & ASSOCIATES, INC.
6010-B MIDWAY PARK BLVD. N.E.
ALBUQUERQUE NEW MEXICO 87109
ENGINEERS SURVEYORS (505) 345-4250

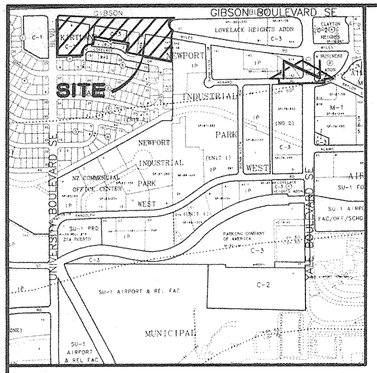
MASTER DRAINAGE PLAN
MILES ROAD S.E. / KIRTLAND ADDITION

DESIGNED BY G.M./G.R.B.
DRAWN BY T.N.T.
APPROVED BY J.G.M.

NO.	DATE	BY	REVISIONS

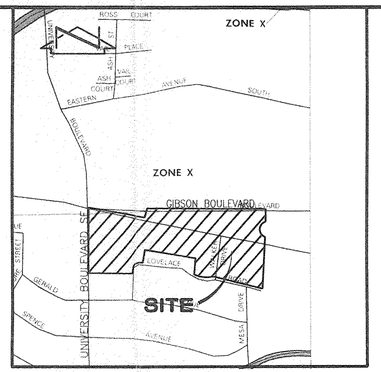
JOB NO. 950925
DATE 03-1997
SHEET 1 OF 2

C:\DATA\ACADDWG\950925\950925M4 Mon Mar 3 15:05:32 1997 -TNT



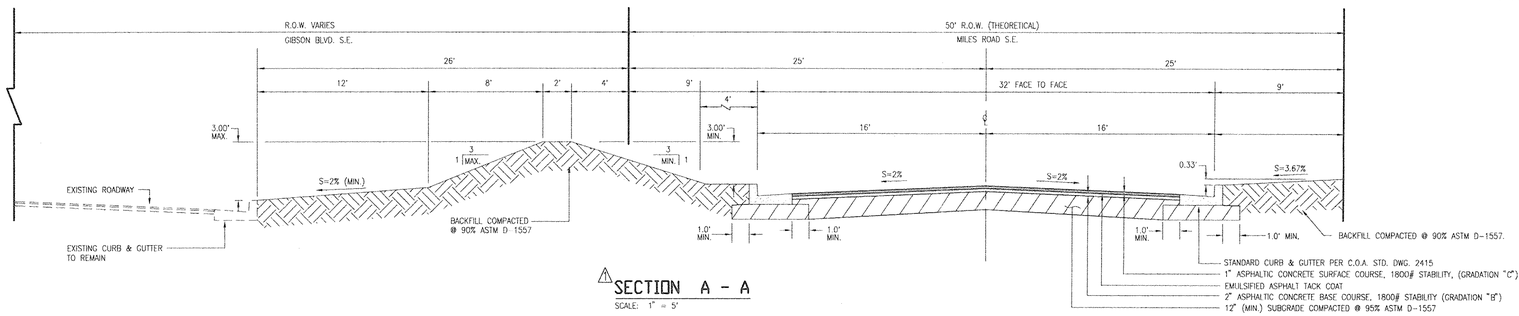
VICINITY MAP M-15

SCALE: 1" = 750'



F.I.R.M. PANEL 342 OF 825

SCALE: 1" = 500'



⚠ SECTION A - A

SCALE: 1" = 5'

STREET CAPACITY CALCULATIONS

USING D.P.M. PLATE 22.3 D-2.
40' F-F
60' R.O.W.
S=0.049 ft/ft
y=0.67'

$Q_{(half\ street)} \approx 75$ cfs

$Q_{FULL\ STREET} = 2(75) = 150$ cfs $>> Q_{100}$

RUNDOWN CALCULATIONS

1. Curb Opening

$Q = CLH^{3/2}$

C = 2.7
Q = 23.5 cfs (Per M.D.P.)
H = 0.67 ft (8" curb height)

Therefore: L = 15.87 ft.
Use 16.0 ft. design width

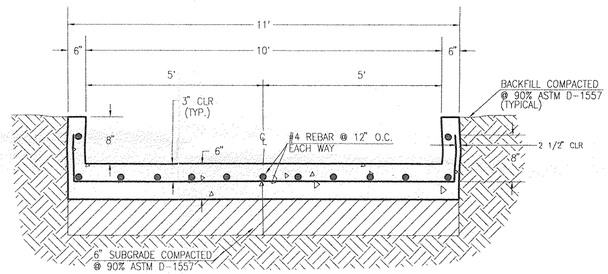
2. Channel Width
Using Manning's Equation

Let: n = 0.013
s = 0.0850 ft/ft
Q = 23.5 cfs (Per M.D.P.)
h = 0.67 (8" curb height)
Therefore: W = 1.94 ft.

Therefore:

3. Velocity times Depth per D.P.M.,
vd < 6.5 @ Q_{10}

Using Manning's Equation
Let: n = 0.013
s = 0.0850 ft/ft
$Q_{10} = 0.67\ Q_{100} = 0.67(23.5) = 15.7$ cfs
W = 10.0 ft.
Therefore: v = 9.7 fps
d = 0.16 ft
vd = 1.6 << 6.5



SECTION B - B

SCALE: 1"=2'-0"

DRAINAGE BASIN SUMMARY TABLES (EXISTING AND DEVELOPED CONDITIONS)

BASIN SUMMARY (OBSERVED AT UNIVERSITY AND GERALD)

	BASIN	AREA (ac)	V_{100} (Exist.) (cf)	V_{100} (Developed) (cf)	ΔV_{100} (cf)	Q_{100} (Exist) (cfs)	Q_{100} (developed) (cfs)	ΔQ_{100} (cfs)
⚠	A-1	0.91	3,760	0	(3,750)[C]	3.6	0.0	(3.6)[C]
⚠	A-2	0.86	3,530	5,410[A]	1,880	3.4	0.8[A,B]	(2.6)[C]
⚠	B	1.42	5,590	8,710	3,120	4.5	0.8[B]	(3.7)[C]
⚠	C	1.36	5,710	0	(5,710)[C]	4.3	0.0	(4.3)[C]
	DE-1	1.21	4,990	0	(4,990)[C]	3.8	0.0	(3.8)[C]
	DE-2	0.11	470	880	410	0.3	0.5	0.2
⚠	F-1	0.91	3,720	0	(3,720)[C]	2.9	0.0	(2.9)[C]
	F-2	0.67	2,750	4,440	1,690	2.1	0.8[B]	(1.3)[C]
⚠	M	1.34	5,510	0	(5,510)[C]	4.2	0.0	(4.2)[C]
⚠	Alley	0.09	360	680	320	0.3	0.4	0.1
	Offsite	0.63	4,250	0	(4,250)[C]	2.7	0.0	(2.7)[C]
⚠	TOTAL:	9.51	40,630	20,120	(20,510)[C]	32.1	3.3	(28.8)[C]

BASIN SUMMARY (OBSERVED AT GIBSON)

	BASIN	AREA (ac)	V_{100} (Exist.) (cf)	V_{100} (Developed) (cf)	ΔV_{100} (cf)	Q_{100} (Exist (cfs)	Q_{100} (developed) (cfs)	ΔQ_{100} (cfs)
⚠	A-1	0.91	0	5,780	5,780	0	3.6	3.6
⚠	A-2	0.86	0	5,410[A]	5,410[A]	0	3.4	3.4
⚠	B	1.42	0	0	0	0	0	0
⚠	C	1.36	0	8,770	8,770	0	5.6	5.6
	DE-1	1.21	0	7,700	7,700	0	5.0	5.0
	DE-2	0.11	0	0	0	0	0	0
⚠	F-1	0.91	0	5,970	5,970	0	3.8	3.8
	F-2	0.67	0	0	0	0	0	0
⚠	M	1.34	0	9,760	9,760	0	6.0	6.0
⚠	Alley	0.09	0	0	0	0	0	0
	Offsite	0.63	0	4,250	4,250	0	2.7	2.7
⚠	TOTAL:	9.51	0	47,640	47,640	0	30.1	30.1

TOTAL DEVELOPED AREA DRAINING TO GIBSON (BEFORE C.O.A. PROJ. NO. 4850.90): 6.85 ac (72%)
TOTAL DEVELOPED AREA DRAINING TO GIBSON (AFTER C.O.A. PROJ. NO. 4850.90): 7.19 ac (75%)

A. FLOW TO BE DIVERTED TO GIBSON STORM DRAIN UPON COMPLETION OF C.I.P.
B. FLOW RATE OF 0.8 CFS ACHIEVED THROUGH DETENTION
C. NUMBERS IN PARENTHESES ARE NEGATIVE THEREBY REPRESENTING A DECREASE.

MASTER DRAINAGE PLAN

MASTER DRAINAGE PLAN

The following items concerning the Tracts A-G, Airport Center Master Drainage Plan are contained hereon:

1. Vicinity Map
2. F.I.R.M.
3. Basin Summary Tables
4. Conceptual Grading Plan
5. Future Gibson Blvd. S.E. Storm Drain

As shown by the Vicinity Map, the site is located at the southeast corner of Gibson Boulevard S.E. and University Boulevard S.E. More particularly, the site is bounded by University Boulevard S.E. on the west, Gibson Boulevard S.E. on the north, existing developments on the east (M15/D22 and M15/D21A) and Lovelace Road S.E., an unpaved public alley, and a small apartment complex on the south. Miles Road S.E. crosses through the site. At present, Miles Road S.E. is unimproved.

As shown by Panel 342 of 825 of the National Flood Insurance Program Flood Insurance Rate Maps published by F.E.M.A. for Bernalillo County, New Mexico dated September 20, 1996, this site does not lie within a designated flood hazard area. The site does, however, contribute runoff to a flood hazard area which lies downstream of the site along Mulberry Street S.E. Currently the site is undeveloped with the exception of Tract C which contains an office building and paved parking lot.

The Conceptual Grading Plan shows: 1) existing and proposed grades indicated by spot elevations and contours at 1'0" intervals, 2) the limit and character of the existing improvements, 3) the limit and character of the proposed improvements, 4) continuity between existing and proposed grades, and 5) proposed pond locations.

It is the intent of the Master Drainage Plan to outline the probable development scenario of the properties and obtain approval of the drainage characteristics to serve the properties. This plan will also facilitate approval of the drainage requirements for vacation of Miles Road S.E., Walker Drive S.E., and public alleys in anticipation of Lot 1, Block A, Kirtland Addition replatting action.

As indicated by the existing topography, the site slopes from east to west toward University Blvd. S.E. Consequently, offsite flows do not enter from the west. Offsite flows do not enter from the north or south because those areas are topographically lower. The two developed parcels to the east discharge to Gibson (M15/D21A) and to the Kirtland Park Channel (M15/D22), respectively. Minor offsite flows are generated by that portion of Miles Road between Buena Vista Drive S.E. and the east end of the site. That area to the east contributes flows calculated to be 2.7 cfs. These flows will be accepted by the proposed Miles Road and carried by the street section to Gibson Blvd. S.E. In the existing condition, the majority of the site drains to University Blvd. S.E. where runoff will flow south to be intercepted by the Kirtland Addition, Unit 1, street system. These streets drain west to Mulberry Street S.E. where a flood zone is designated between Gerald S.E. and Wheeler S.E.

The Developed Drainage Basins shown hereon have been assumed to have the following future uses per information from the current property owners: Basin A1 - gas station, convenience store, fast food restaurant, police substation; Basin B - commercial/retail development; Basin C - currently utilized as a car rental facility; Basins DE - commercial/retail development; Basin F - industrial park use. Basin G contains a PNM facility that will remain.

In recognition of downstream flooding conditions on Mulberry Street S.E., and considering the proposed Gibson Boulevard S.E. Reconstruction/Rehabilitation, University Boulevard to Jackson Street (C.O.A. Project #4850.90), this Plan will divert as much developed runoff to Gibson as is physically possible. As determined from the Basin Summary Table, 75% of the developed area will ultimately drain to Gibson. Of the private areas which will continue to drain to University Blvd., only Basin DE-2 will have free discharge. This is because Basin DE-2 is physically incapable of draining to Gibson Blvd., is very small, and is oddly shaped and will most likely be developed as landscaping, although it was conservatively analyzed as Land Treatment "D" in the Calculations. For Basins B and F-2 it is proposed to limit the discharge from each tract to the flow rate which is delivered by a 6" drain pipe. This is the smallest discharge possible without utilizing onsite retention which is prohibited by Ordinance. These two Basins will utilize the concept of onsite detention ponding via a 6" storm drain discharging to historic points on public right-of-way. Individual grading and drainage plans will be required for the development of each tract as a condition for permit approvals.

The detained discharge rate (q) from Basins A-2, B and F-2 have been conservatively calculated using the Orifice Equation with an average head of 4 feet which was the head calculated in the Grading and Drainage Plan previously submitted for the current Lot 1, Kirtland Addition Grading and Drainage Plan (City of Albuquerque Hydrology File No. M15/D32; Engineer's Stamp 1/26/96). This yields a peak flow rate of 0.8 cfs from each Basin. The public alley and Basin DE-2 will discharge freely to Lovelace Road S.E. A total peak flow rate of 3.3 cfs will discharge to University Blvd. S.E. which is significantly less than the current rate of 30.6 cfs. There will also be a significant reduction in volume of 100-year runoff from 40,630 cf to 20,120 cf. It is proposed that the detention ponding be accommodated in the paved parking areas similar to the previously mentioned site specific plan (M15/D32, 1/26/96). In the interim, Basin A-2 should discharge its developed runoff at a controlled rate to University Boulevard S.E. This is proposed as an interim solution. The ultimate solution, as shown by this Master Drainage Plan, is to construct a private storm drain connection into the back of a public storm drain inlet proposed as part of the Gibson Boulevard project. It is further proposed that this line should be installed as a "dry" line at the time that Tract A is developed and that a temporary plug be placed in the line to render it "dry". At such time as the Gibson Boulevard storm drain and associated inlets are constructed, the plug can be removed and inserted into the temporary discharge line thereby diverting flows to Gibson. When the Basin A-2 diversion is completed there will be a further reduction in the flow rate to University by 0.8 cfs and a reduction in the volume of runoff by 5,410 cubic feet. The future discharge line for Basin A-2 should be sized for the free discharge of 3.4 cfs, and should be installed as shallow as possible to prevent tie-in problems in case the storm drain system depth changes during construction.

In the preparation of this plan, the Preliminary Drainage Report for the "Gibson Boulevard Reconstruction/Rehabilitation, University Boulevard to Jackson Street" (COA Project No. 4850.90) prepared by Avid Engineering, Inc., dated August 1995 has been reviewed. It is noted that the majority of this site lies within Basin 70 as identified on the Basin Map appearing therein. The Gibson Boulevard Report conservatively calculates that all of Basin 70 will drain into Gibson Boulevard S.E. Closer review of the topographic data presented reveals that most of the area comprising Tracts A-G, Airport Center drain to Lovelace Road S.E., or University Boulevard S.E. The AHYMO Model created by Avid Engineering for the developed conditions shows a discharge rate of 4.43 cfs per acre for Basin 70. As can be determined from the Basin Summary Table for Gibson Blvd., Basins A-1, A-2, C, DE-1, M-1, F-1, and the Offsite Basin will discharge 30.1 cfs/7.37 acres = 4.08 cfs per acre. These Basins will all surface drain to Gibson Blvd. either directly, or via the temporary run-down to be constructed from Miles Rd. S.E. to Gibson Blvd.

This site lies within Reach 1 as designated in the Avid report which extends from Yale Boulevard to University Boulevard and the South Diversion Channel. The storm drain improvements in this reach are intended to a) collect runoff from areas east and south of the Yale intersection, b) to eliminate runoff to the unimproved arroyos/private property along the north side of Gibson Boulevard S.E. and to c) reduce surface flows through the University Boulevard intersection in accordance with City design criteria. Discussions with Mr. Rick Beltramo of Avid Engineering indicate that the system is being constructed in order to allow for free discharge from Basin 70. The diversion of that runoff from a University Boulevard discharge to a Gibson Boulevard discharge will be advantageous so as to lessen downstream impacts. The future alignment for the Gibson Boulevard storm drain on this plan along with preliminary inlets locations. The final design and analysis of the future storm drain and inlet locations is being coordinated between this office and Avid Engineering, Inc.

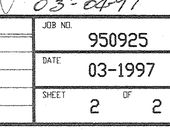
It appears likely that Miles Road will develop before the Proposed Gibson Blvd. Storm Drain will be constructed. In the interim, there will be free discharge of additional developed runoff to Gibson Boulevard. This is appropriate because this development is programmed toward the ultimate drainage scenario, the proposed construction lies in close proximity to the bottom of the Gibson Watershed and the existing facilities which convey flows into the South Diversion Channel. Carlos Montoya, C.O.A. Hydrology Section, indicated via phone conversation that the existing facilities located at the bottom of Gibson Blvd. have sufficient capacity to convey Gibson flows into the South Diversion Channel.

The Miles Road S.E. Street Section will convey all runoff from Basins C, DE-1, F-1, M-1, and the offsite flows directly to Gibson Blvd. S.E. via a temporary rundown. The temporary rundown is provided in lieu of permanent storm drain improvements to provide maximum flexibility in the design of the Gibson Blvd. S.E. improvements. The rundown is 10' wide and constructed with concrete. Bollards will be installed at the inlet and outlet of the rundown to discourage traffic use. Per the D.P.M., the maximum value for the runoff depth multiplied by runoff velocity crossing driving lanes is 6.5, for the 10-year storm event. As shown by the Calculations, the depth multiplied by the velocity is equal to 1.57, which is much less than 6.5 for the 10-year event. The 100-year event increases the value to 2.35, which is still less than 6.5, the D.P.M. requirement.

Runoff from Basin A-1 will drain directly to Gibson Blvd. S.E. either via drivepad and/or private drainage improvements with the exact method to be addressed by separate submittal for Building Permit. Basin A-2 will ultimately drain to University Blvd. S.E. via private storm drainage improvements within public right-of-way. Paving (street) improvements alone are sufficient to accept and convey the developed runoff analyzed hereon; no public drainage infrastructure is required.

The Calculations which appear hereon analyze both the existing and developed conditions for the 100-year, 6-hour rainfall event. The Procedure for 40-acre and Smaller Basins, as set forth in the Revision of Section 22.2, Hydrology of the Development Process Manual, Volume 2, Design Criteria, dated January, 1993, has been used to quantify the peak rate of discharge and volume of runoff generated. As shown by these calculations, there is a net increase in volume and peak flow rates generated for each tract, but by redirecting much of the runoff to Gibson Blvd. N.E., and through the use of detention ponding, the peak flow rate and volume of runoff discharging to University will decrease significantly when compared to the existing flow rates. This reduction in flow rate and volume will lessen the impact on downstream flooding observed during significant rainfall events.

THIS PLAN REPLACES THE PREVIOUS PLAN DATED 02-15-1996.



3-03-0497

JEFF MORTENSEN & ASSOCIATES, INC.
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VICINITY MAP, DRAINAGE PLAN, CALCULATIONS, SECTIONS AND F.I.R.M.
MASTER DRAINAGE PLAN
MILES ROAD S.E. / KIRTLAND ADDITION

DESIGNED BY G.M./G.R.B.
DRAWN BY S.G.H.
APPROVED BY J.G.M.

NO.	DATE	BY	REVISIONS
⚠	2/97	GRB	REVISE TO ACCOUNT FOR NEW ALIGNMENT OF MILES ROAD S.E.

JOB NO. 950925
DATE 03-1997
SHEET 2 OF 2

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