Drainage Reports

Abbreveated Water & Sewer Need Reports

Water Study

Wastewater Study

Stormwater Waiver Application

Engineering

Comments on Grading and Drainage (Cycle #2)
LOMAS VERDES ESTATES
(City of Scottsdale Case Number: 5-PP-2017)

The Case Drainage Report should be prepared by following the City of Scottsdale (COS) Design Standards & Policies Manual (DS&PM) and in accordance with the revised City Stormwater Ordinance Chapter 37.

The Engineer has submitted the Case Drainage Report for the first time with the 2nd submittal of this case. Also, instead of having a subdivision with custom lots as per the 1st submittal, this time the Preliminary Grading & Drainage (G&D) Plan shows that it is going to be a mass-graded subdivision where both the pad and the Finished Floor (FF) elevation for the house on each lot has been established. Therefore, a number of additional comments have been generated as a result of the major modifications to the previous concept. However, many of the comments are repetitive from the 1st cycle which didn't get addressed. The Engineer must come to the City for a meeting with the Stormwater reviewer and Stormwater Review Manager prior to resubmittal.

Please address the following drainage comments:

1. The Engineer must submit a CD in the back pocket of each drainage report of the requested two (2) copies of the report to ensure no misplacement of the CD takes place. The CD must contain a PDF file of the complete sealed and signed drainage report as well as all digital HEC-RAS files. [*Reference: COS DSPM: Section 4-1.800 & Section 4-1A*]

2. A Case Drainage Report under the Preliminary Plat (PP) category is typically a 90% to 95% of the Final Drainage Report, in which not only the drainage concept associated with the offsite washes has to be finalized, but also all onsite drainage improvements should be somewhat finalized so that the each lot size is final when subdivided and is not subject to any modifications. The building envelope on each lot must have a minimum developable area as designated by Residential Zoning. [*Reference: COS DSPM: Section 4-1.800 & Section 4-1A*]

3. Label all 1.0 ft. existing contours on the Preliminary G&D plan. Please darken every 5th contour line. Show and label all proposed contours. Use different line types for the existing and the proposed contour lines. [*Reference: COS DSPM: Section 4-1.900 & Section 4-1B*]

4. Create a table in the drainage report and enlist the 100-year existing and the proposed Water Surface Elevation (WSE) for each HEC-RAS cross-section (XS) and the ΔWSE. In the same table, enlist the 100-year existing and the proposed velocity for each HEC-RAS XS and the Δ velocity to demonstrate 'no adverse impact'. [*Reference: COS DSPM: Section 4-1.800*]

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5. Add additional XSs to the current HEC-RAS river reach to go at a minimum of 100 feet beyond the east and west property lines to ensure that the upstream (U/S) and the downstream (D/S) boundary conditions have no influence on the pre- vs. post- HEC-RAS models throughout the property. [*Reference: COS DSPM: Section 4-1.800 & Section 4-1A*]

6. The City requires a minimum of an additional 50 feet of survey topography beyond the property lines. In addition to having and showing 50 feet of survey topography beyond the property lines on the Preliminary G&D plan, the Engineer must obtain digital COS quarter section topography maps (CAD files) from the City's GIS dept. in order to set up and run the HEC-RAS models free from the influences of U/S and D/S boundary conditions. [*Reference: COS DSPM: Section 4-1.900 & Section 4-1B*]

7. The 11"X17" Existing Condition Floodplain Map and Proposed Condition Floodplain Map provided in the drainage report must show the survey topography as well as the supplemented COS topography. Do not show the supplemented COS topography on the Preliminary G&D plans. [*Reference: COS DSPM: Section 4-1.800 & Section 4-1A*]

8. The proposed floodplain goes over the proposed onsite retention basins. This is not allowed. The basins have to be strictly offline and for onsite 100-year, 2-hour full storage only and must be physically isolated from the floodplains of the washes. [*Reference: COS DSPM: Section 4-1.402*]

9. The Engineer must demonstrate how these basins will be drained out. If bleed off pipes are used to drain out these basins into adjacent washes, then such must be shown on the G&D plan. If the basins are retention, then the Engineer must state in the report that a Geotechnical Report will be submitted with the Final Drainage Report showing percolation test in support of drain time which is 36 hours maximum. The maximum basin slope is 4:1. Please label them on the G&D plan. [*Reference: COS DSPM: Section 4-1.402*]

10. In addition to dedicating Drainage Easement (D.E.) around the 50+ cfs washes as well as around the basins, a minimum of 8.0 feet wide Access Easements (A.E.) must be provided from the public Right of Way (R.O.W.) and/or from the private tract to the detention/retention basins D.E. to grant right to the City for access to these basins. [*Reference: COS DSPM: Section 4-1.700*]

11. Show the erosion setback lines on the G&D plan. All graded channels must meet 'channel freeboard' criteria as well as erosion protection criteria against permissible velocities per the FCDMC policies and manuals. Document it in the report. [*Reference: COS DSPM: Section 4-1.700 &Section 4-1.800*]

Please briefly respond to each of the above comments (or check them with markers) and include the responses in the re-submittals.

Stormwater Review By:
Mohammad Rahman, PE, PH, CFM
Phone 480-312-2563 Fax 480-312-7781
e-mail: mrahman@ScottsdaleAZ.gov
Review Cycle #2 Date 5/30/17

Engineering and Environmental Consultants, Inc.
7740 N. 16th Street, Suite 135 | Phoenix, Arizona 85020 | P: 602.248.7702

Stormwater Review By:
Mohammad Rahman, PE, PH, CFM
Phone 480-312-2563 Fax 480-312-7781
e-mail: mrahman@ScottsdaleAZ.gov
Review Cycle #2 Date 5/30/17

DRAINAGE REPORT

FOR

LOMAS VERDES ESTATES

6501 E. Red Bird Road
Scottsdale, Arizona 85266

OWNER:
Lomas Verdes Estates, LLC
7001 E. Main Street; Suite 101
Scottsdale, AZ 85251
Phone: (480) 221-9311

Registered Professional Engineer (Civil)
CERTIFICATE NO 31682
JAMES M. GRIFFIN
Date Signed 5/12/17
ARIZONA, U.S.A.
Expires 9/30/18

January 25, 2017
Revised May 11, 2017

Prepared by:
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5-PP-2017
05/15/17

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WATER DISTRIBUTION SYSTEM

BASIS OF DESIGN REPORT

FOR

LOMAS VERDES ESTATES

6501 E. Red Bird Road
Scottsdale, Arizona 85266

OWNER:
Lomas Verdes Estates, LLC
7001 E. Main Street; Suite 101
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CERTIFICATE NO.
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JAMES M.
GRIFFIN
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Expires 9/30/18

January 25, 2017

PREPARED BY:

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Vicinity Map

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Flow Test Results

Appendix C:
Model Results;

- Average Day
- Maximum Day
- Peak Hour
- Maximum Day with Fire Demand
- Fire Demand at 30 psi

INTRODUCTION

Lomas Verdes Estates is a single family custom and semi-custom residential subdivision to be constructed on approximately 7 acres. The site is located East of 64th Street and South of Red Bird Road. The site is bordered to the North, West and South by existing residential properties. The site lies within the North half of the South half of the Southwest Quarter of Section 34, Township 5 North, Range 4 East of Gila and Salt River Base and Meridian. The Assessor's Parcel Number for this property is 212-10-003F. Based on the information provided on the Maricopa County Assessor's Maps, the site has a Latitude of 33°43'45"N and a Longitude 111°56'33"W at the approximate center of the site. The approximate elevation of the site is 1964.00. See the Appendix for a Vicinity Map.

EXISTING CONDITIONS

The property is currently zoned R1-43 and is approximately 8 acres in size. The slope of the land is generally from northeast to southwest. There is approximately 16-feet of fall from the rear (north) of the site to the front (south) of the site. A horse stable and fencing exist along the southeast corner of property. An existing fence follows the south property line and a portion of the east line. The site consists of native desert with a cleared/dirt area in the southeast corner for horse training. The site is in Flood Zone X, as depicted on the FEMA Flood Insurance Rate Map. A site aerial map has been provided within the Appendix.

There is an existing 12" public water main in 64th street.

PROPOSED CONDITIONS

Lomas Verdes Estates will provide a new public water main connecting to the existing 12" main in 64th Street and extending an 8" water main, via a tapping sleeve and valve, to the end of the site cul-de-sac. The new water main will be located within public right of way to be dedicated as part of this project development. The proposed development will provide one new public fire hydrant near the southeast corner of the site. Additionally, 6 new domestic water meters and a landscape meter are to be provided for the proposed development.

WATER ANALYSIS DATA

Per City of Scottsdale DS&PM manual, Figure 6.1-2 Average Day Water Demands in Gallons per day, this projects Residential Demand per dwelling unit is <2DU/ac = 485.6 gpd

Average Day Demand = 485.6 gpd x 6 dwellings = 2,913.6 gpd or 2.02 gpm

Maximum Day Demand = Average Day Demand x 2 = 5,827.20 gpd or 4.05 gpm

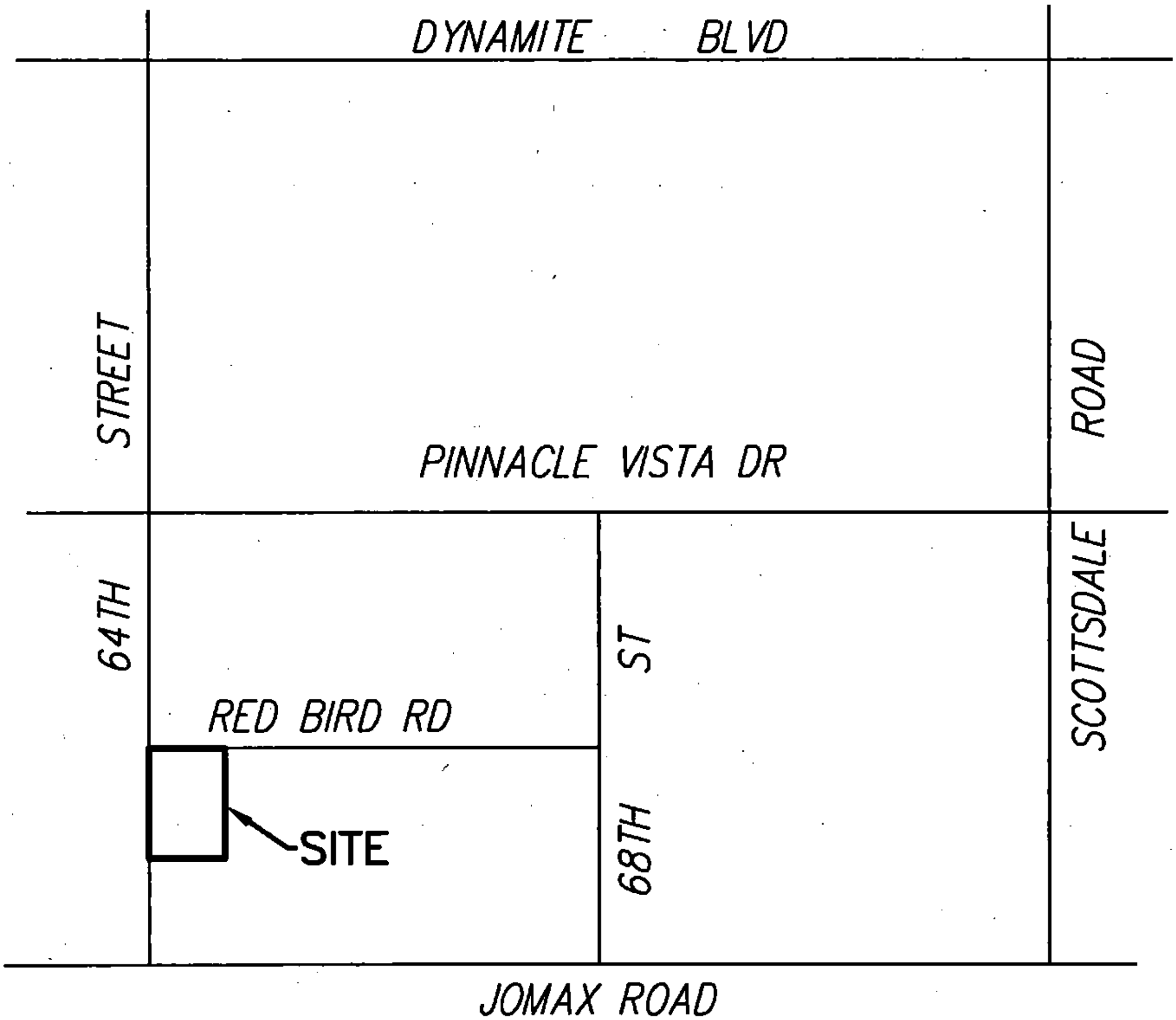
Peak Hour = Maximum Day Demand x 3.5 = 10,197.60 gpd or 7.08 gpm

Fire Flow Demand = 500 gpm with 30 psi residual

Maximum Day with Fire Demand = 507.08 gpm

Based on the Fire Hydrant Flow Test Results, the existing 12-inch waterline and the new 8-inch waterline are adequately sized to provide water supply for the proposed demand and intended use.

APPENDIX "A"



VICINITY MAP

APPENDIX "B"

E•J | Flow Test Summary

Project Name:	EJFT 17018
Project Address:	26697-26891 N 64th St, Scottsdale, AZ 85266
Date of Flow Test:	2017-02-01
Time of Flow Test:	8:15 AM
Data Reliable Until:	2017-08-01
Conducted By:	Eder Cueva & Matt Young (EJ Flow Tests) 602.999.7637
Witnessed By:	Jim Tunnell (City of Scottsdale) 602.819.7718
City Forces Contacted:	City of Scottsdale
Permit Number:	C52492

Note Max Static Pressure of 72 PSI utilized as a safety factor

Raw Flow Test Data

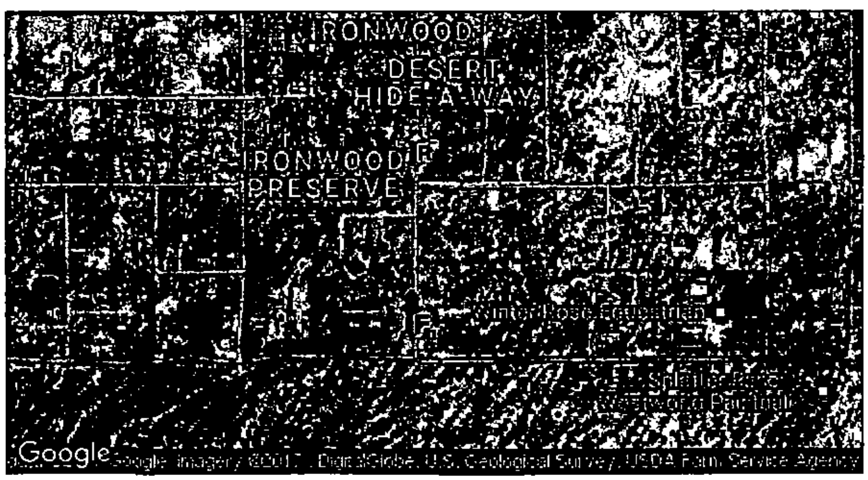
Static Pressure:	106.0 PSI
Residual Pressure:	66.0 PSI
Flowing GPM:	2,176
GPM @ 20 PSI:	3,289

Data with a 34 PSI Safety Factor

Static Pressure:	72.0 PSI
Residual Pressure:	32.0 PSI
Flowing GPM:	2,176
GPM @ 20 PSI:	2,507

Hydrant F_1

Pitot Pressure (1):	42	PSI
Coefficient of Discharge (1):	0.9	
Hydrant Orifice Diameter (1):	2.5	inches
Pitot Pressure (2):	42	PSI
Coefficient of Discharge (2):	0.9	
Hydrant Orifice Diameter (2):	2.5	inches



Static-Residual Hydrant

Flow Hydrant

Distance Between F_1 and R
1271 ft (measured linearly)

Static-Residual Elevation
1969 ft (above sea level)

Flow Hydrant (F_1) Elevation
1948 ft (above sea level)

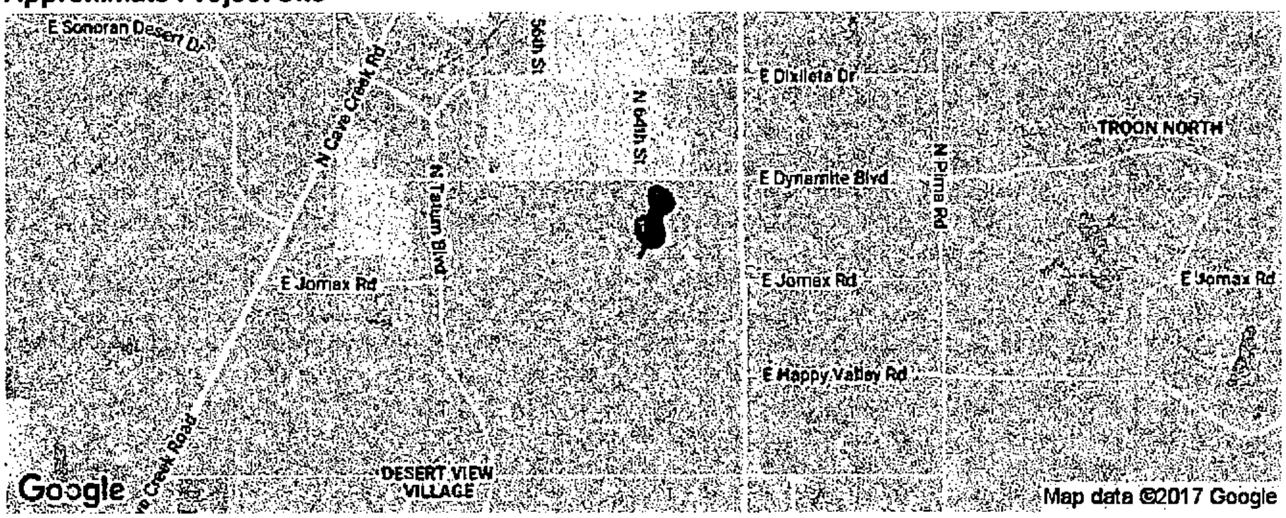
Elevation & distance values are approximate

E•J | Flow Test Summary

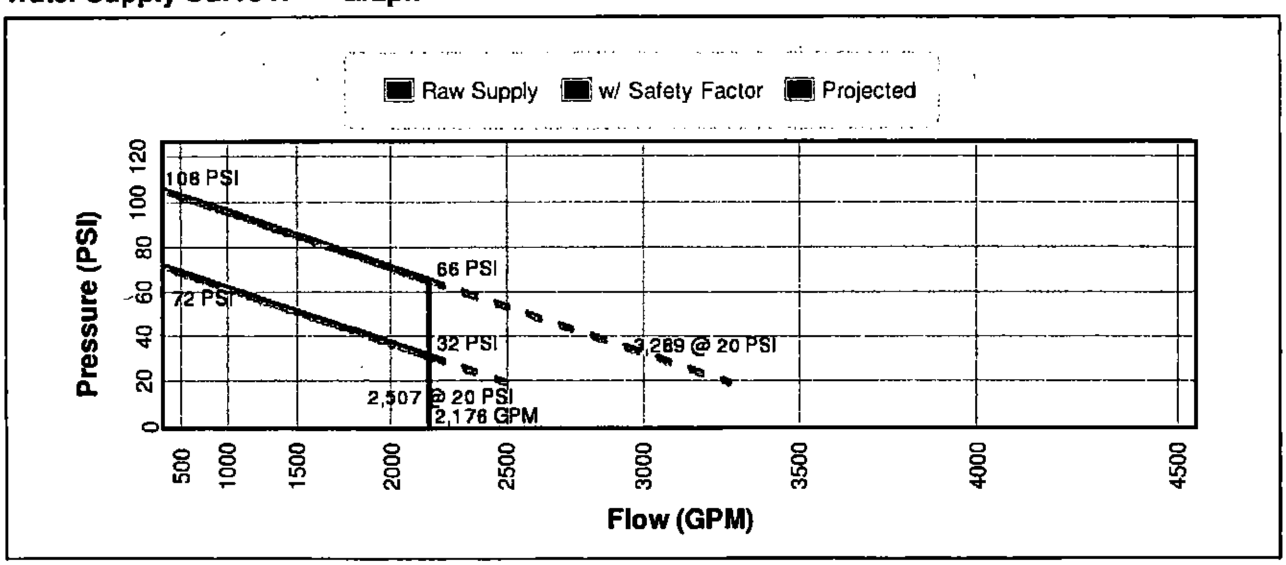
Static-Residual Hydrant

Flow Hydrant (only hydrant F1 shown for clarity)

Approximate Project Site



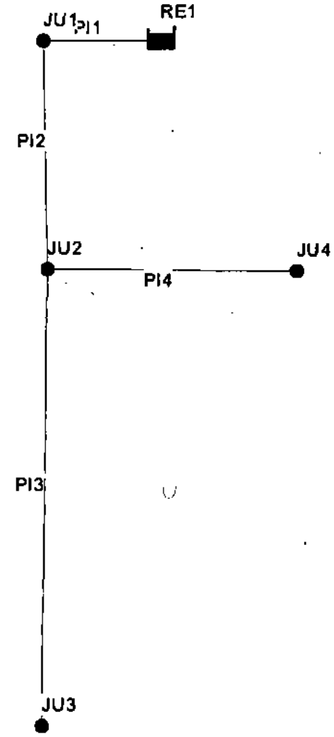
Water Supply Curve $N^{1.85}$ Graph



APPENDIX "C"

LOMAS VERDES ESTATES - WATER MODEL

Day 1



```
**********************************************************************
*                           E P A N E T                              *
*                  Hydraulic and Water Quality                       *
*                  Analysis for Pipe Networks                        *
*                          Version 2.0                               *
**********************************************************************
```

Input File: 16534.net

AVERAGE DAY DEMAND

Link - Node Table:

Link ID	Start Node	End Node	Length ft	Diameter in
PI1	RE1	JU1	1000	24
PI2	JU2	JU1	245	12
PI3	JU3	JU2	1026	12
PI4	JU2	JU4	298	8

Node Results:

Node ID	Demand GPM	Head ft	Pressure psi	Quality	
JU1	0.00	2131.17	72.00	0.00	
JU2	0.00	2131.17	73.73	0.00	
JU3	0.00	2131.17	81.10	0.00	
JU4	2.02	2131.17	71.13	0.00	
RE1	-2.02	2131.17	0.00	0.00	Reservoir

Link Results:

Link ID	Flow GPM	Velocity fps	Unit Headloss ft/Kft	Status
PI1	2.02	0.00	0.00	Open
PI2	-2.02	0.01	0.00	Open
PI3	0.00	0.00	0.00	Open
PI4	2.02	0.01	0.00	Open

```
**********************************************************************
*                            E P A N E T                             *
*                    Hydraulic and Water Quality                     *
*                    Analysis for Pipe Networks                      *
*                           Version 2.0                              *
**********************************************************************
```

Input File: 16534.net

MAXIMUM DAY DEMAND

Link - Node Table:

Link ID	Start Node	End Node	Length ft	Diameter in
PI1	RE1	JU1	1000	24
PI2	JU2	JU1	245	12
PI3	JU3	JU2	1026	12
PI4	JU2	JU4	298	8

Node Results:

Node ID	Demand GPM	Head ft	Pressure psi	Quality	
JU1	0.00	2131.17	72.00	0.00	
JU2	0.00	2131.17	73.73	0.00	
JU3	0.00	2131.17	81.10	0.00	
JU4	4.05	2131.17	71.13	0.00	
RE1	-4.05	2131.17	0.00	0.00	Reservoir

Link Results:

Link ID	Flow GPM	Velocity fps	Unit Headloss ft/Kft	Status
PI1	4.05	0.00	0.00	Open
PI2	-4.05	0.01	0.00	Open
PI3	0.00	0.00	0.00	Open
PI4	4.05	0.03	0.00	Open

```
**********************************************************************
*                                E P A N E T                         *
*                        Hydraulic and Water Quality                 *
*                        Analysis for Pipe Networks                  *
*                              Version 2.0                           *
**********************************************************************
```

Input File: 16534.net

MAXIMUM DAY DEMAND PLUS FIRE DEMAND

Link - Node Table:

Link ID	Start Node	End Node	Length ft	Diameter in
PI1	RE1	JU1	1000	24
PI2	JU2	JU1	245	12
PI3	JU3	JU2	1026	12
PI4	JU2	JU4	298	8

Node Results:

Node ID	Demand GPM	Head ft	Pressure psi	Quality	
JU1	0.00	2131.15	71.99	0.00	
JU2	0.00	2130.97	73.65	0.00	
JU3	0.00	2130.97	81.01	0.00	
JU4	507.08	2129.42	70.38	0.00	
RE1	-507.08	2131.17	0.00	0.00	Reservoir

Link Results:

Link ID	Flow GPM	Velocity fps	Unit Headloss ft/Kft	Status
PI1	507.08	0.36	0.02	Open
PI2	-507.08	1.44	0.72	Open
PI3	0.00	0.00	0.00	Open
PI4	507.08	3.24	5.19	Open

```
**********************************************************************
*                             E P A N E T                            *
*                    Hydraulic and Water Quality                     *
*                    Analysis for Pipe Networks                      *
*                           Version 2.0                              *
**********************************************************************
```

Input File: 16534.net

FIRE FLOW DEMAND @ 30 PSI

Link - Node Table:

Link ID	Start Node	End Node	Length ft	Diameter in
PI1	RE1	JU1	1000	24
PI2	JU2	JU1	245	12
PI3	JU3	JU2	1026	12
PI4	JU2	JU4	298	8

Node Results:

Node ID	Demand GPM	Head ft	Pressure psi	Quality	
JU1	0.00	2129.83	71.42	0.00	
JU2	0.00	2120.25	69.00	0.00	
JU3	0.00	2120.25	76.37	0.00	
JU4	4382.00	2036.26	30.01	0.00	
RE1	-4382.00	2131.17	0.00	0.00	Reservoir

Link Results:

Link ID	Flow GPM	Velocity fps	Unit Headloss ft/Kft	Status
PI1	4382.00	3.11	1.34	Open
PI2	-4382.00	12.43	39.11	Open
PI3	0.00	0.00	0.00	Open
PI4	4382.00	27.97	281.85	Open

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WASTEWATER DISTRIBUTION SYSTEM

BASIS OF DESIGN REPORT

FOR

LOMAS VERDES ESTATES

6501 E. Red Bird Road
Scottsdale, Arizona 85266

OWNER:
Lomas Verdes Estates, LLC
7001 E. Main Street; Suite 101
Scottsdale, AZ 85251
Phone: (480) 221-9311

Registered Professional Engineer (Civil)
CERTIFICATE NO.
31682
JAMES M.
GRIFFIN
Date Signed 1/26/17
ARIZONA, U.S.A.
Expires 9/30/18

January 25, 2017

PREPARED BY:

ENGINEERING AND ENVIRONMENTAL CONSULTANTS, INC.
7740 N. 16TH Street
Phoenix, AZ 85020
PH: 602-248-7702
FAX: 602-248-7851

INTRODUCTION

Lomas Verdes Estates is a single family custom and semi-custom residential subdivision to be constructed on approximately 7 acres. The site is located east of 64th Street and south of Red Bird Road. The site is bordered to the North, West and South by existing residential properties. The site lies within the North half of the South half of the Southwest Quarter of Section 34, Township 5 North, Range 4 East of Gila and Salt River Base and Meridian. The Assessor's Parcel Number for this property is 212-10-003F. Based on the information provided on the Maricopa County Assessor's Maps, the site has a Latitude of 33°43'45"N and a Longitude 111°56'33"W at the approximate center of the site. The approximate elevation of the site is 1964.00. See the Appendix for a Vicinity Map.

EXISTING CONDITIONS

The property is currently zoned R1-43 and is approximately 8 acres in size. The slope of the land is generally from northeast to southwest. There are approximately 16-feet of fall from the rear (north) of the site to the front (south) of the site. A horse stable and fencing exist along the southeast corner of property. An existing fence follows the south property line and a portion of the east line. The site consists of native desert with a cleared/dirt area in the southeast corner for horse training. The site is in Flood Zone X, as depicted on the FEMA Flood Insurance Rate Map.

There is currently no city owned and operated gravity sewer service to the project area.

PROPOSED CONDITIONS

Lomas Verdes Estates will provide a new public dry sewer main from a predetermined location within 64th Street to the roadway cul-de-sac within the subdivision. The dry sewer will provide individual sewer taps to each lot for future connection to public sewer. Temporary individual septic systems will provide residential sanitary sewer disposal until public service is available.

WASTEWATER ANALYSIS

Per City of Scottsdale DS&PM manual, Section 7-1.403, Average Day Wastewater Demand

Residential densities = 2.5 persons per dwelling unit with 100 gpcpd with a peaking factor of 4.

Average Day Wastewater Demand for the 6 lot subdivision =

6 dwellings x 2.5 persons x 100 gpcpd = 1,500 gpd or 1.04 gpm

Peak Demand = 4 x 1,500 gpd = 6,000 gpd or 4.17 gpm

See Appendix "A" for capacity analysis.

CONCLUSION

8" Capacity at minimum slope = 0.874 cfs

Site Demand = 0.009 cfs

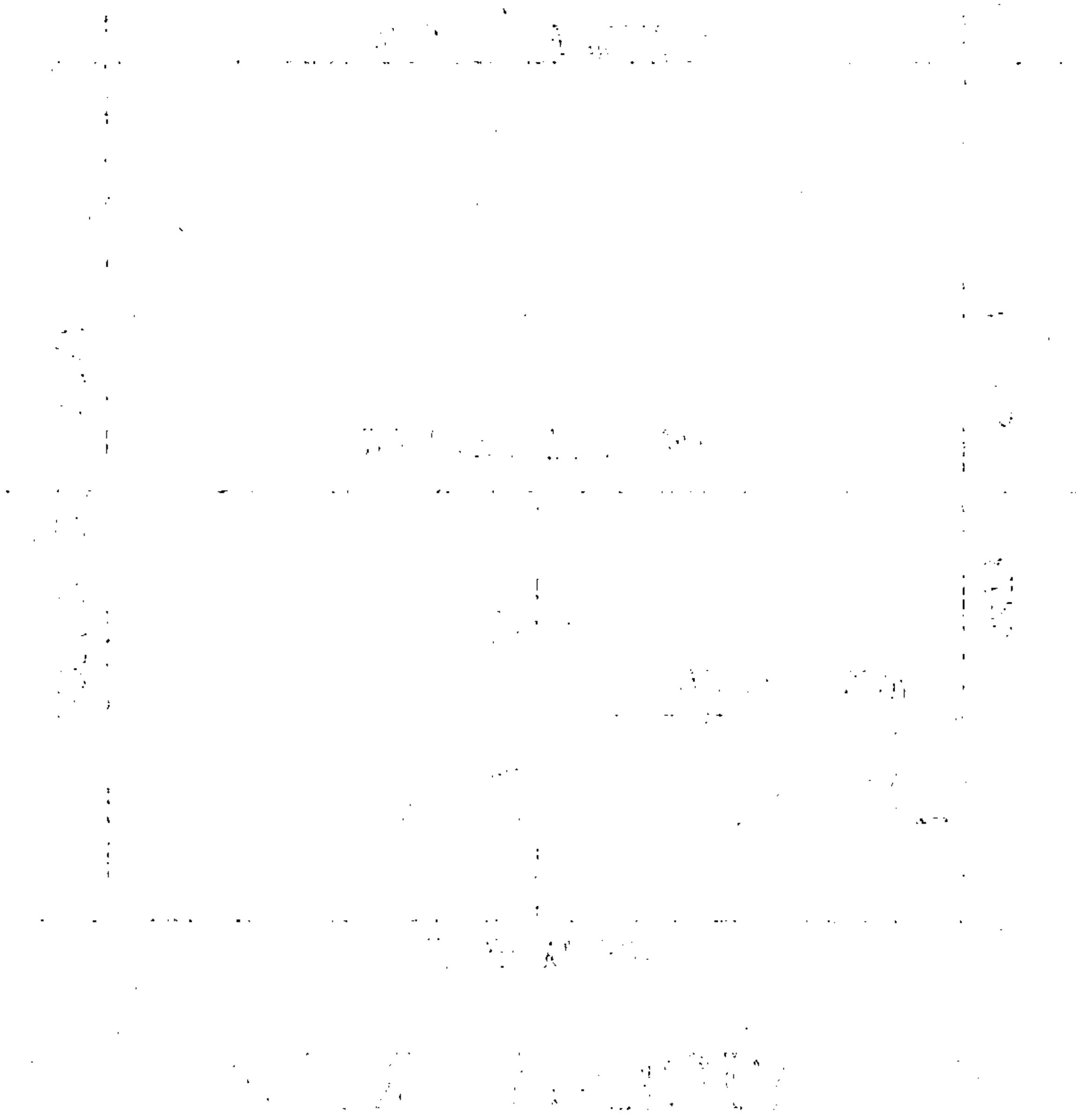
Proposed pipe size provides adequate capacity for the proposed onsite development.

APPENDIX "A"

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	CAPACITY (GPD)	CAPACITY (CFS)	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

PEAK FLOWS VS. PIPE CAPACITY ANALYSIS

PIPE (MH ? - MH ?)	PIPE SIZE (IN)	[illegible]	POPULATION	AVE. DAILY FLOW (GPD)	PEAKING FACTOR	[illegible]	[illegible]	[illegible]
[illegible]	8	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	8	4	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]



APPROVED
M. Rahman
11/17/17

DRAINAGE REPORT

FOR

LOMAS VERDES ESTATES

6501 E. Red Bird Road
Scottsdale, Arizona 85266

OWNER:
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Registered Professional Engineer (Civil)
CERTIFICATE NO
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January 25, 2017
Revised May 11, 2017
Revised July 7, 2017
Revised Oct 4, 2017

Prepared by:
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5-PP-2017
10/26/2017

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FIGURES:

1.0 INTRODUCTION/PURPOSE

The purpose of this report is to provide a drainage narrative of the onsite and offsite drainage considerations for this proposed residential subdivision located at 6501 E. Red Bird Road situated within Scottsdale, Arizona. The site is in Flood Zone X, as depicted on the FEMA Flood Insurance Rate Map. The site is located on the east side of 64th Street and the south side of Red Bird Road, just north of Jomax Road.

2.0 LOCATION

This site lies within the North Half of the South Half of the Southwest Quarter of Section 34, Township 5 North, Range 4 East of the Gila and Salt River Base & Meridian, Maricopa County Arizona. The legal description for the property is as follows: **The West 528 feet of the North half of the South half of the Southwest quarter of Section 34, Township 5 North, Range 4 East of the Gila and Salt River Base and Meridian, Maricopa County, Arizona.** The site is bordered to the north by Red Bird Road, to the east by an existing residential property, to the south by land that has been recently subdivided as single family residences and to the west by 64th Street. The Assessor's Parcel Number for this property is 212-10-003F. Based on the information provided on the Maricopa County Assessor's Maps, the site has a Latitude of 33.7273°N and a Longitude 111.9425°W at the approximate center of the site. The approximate elevation of the site is 1965.00. A vicinity map is provided as FIGURE 1 in the "FIGURES" section of this report.

3.0 SITE DESCRIPTION

The property is currently zoned R1-43 single family and is approximately 332,998 square feet (net), or 7.64 acres. The gross area of the property is approximately 348,834 square feet, or 8.01 acres. The slope of the land is generally from northeast to southwest. There is approximately 16-feet of fall from the northeast corner of the site to the southwest corner of the site providing a slope of just under 2% towards the southwest. The property is primarily native desert with the exception of some grading that occurred near the southeast corner of the site for what appears to be some type of non-permitted horse arena with stables. There are two notable washes that flow from northeast to southwest through the southeast portion of the site. Based on preliminary calculations and investigation, it appears as though these washes convey peak discharges in excess of 50 cfs. The site is in Flood Zone X, as depicted on the FEMA Flood Insurance Rate Map. A site aerial map has been provided as FIGURE 2 within the "FIGURES" section of this report.

Red Bird Road exists along the north side of the site. This roadway is within City of Scottsdale right of way and is currently a well compacted un-paved roadway providing ingress/egress to several subdivisions to the east of this parcel. 64th Street exists along the west side of the site. This roadway is also within City of Scottsdale right of way and is a two way paved roadway that is maintained by the City of Scottsdale.

4.0 FEMA FLOODPLAN CLASSIFICATION

The site lies within Zone "X" (not shaded) as indicated on the Flood Insurance Rate Map (FIRM) for Maricopa County, Arizona, Map Number 04013C 1305L, dated October 16, 2013. Zone "X" (not shaded) is defined as "areas determined to be outside the 0.2% annual chance of floodplain". A copy of the FIRM is provided as FIGURE 3 in the "FIGURES" section of this report.

5.0 OFFSITE DRAINAGE DESCRIPTION

As previously discussed, the site is in Flood Zone X, defined as areas determined to be outside the 0.2% annual chance of floodplain. There are two significant existing washes that enter the site along the east property line that convey offsite runoff through the southeast portion of the site prior to intercepting one another and then exiting the site near the southwest corner of the site where they cross 64th Street as an existing "wet crossing". The northern of the two washes has a 100-year peak discharge of approximately 161 cfs and the southern of the two washes has a 100-year peak discharge of approximately 87 cfs. The two washes combine near the southwest corner of the site with an approximate 100-year peak discharge of 197 cfs. It should be noted that the 100-year peak discharge at the southwest corner of the site (197 cfs) is lower than the combined discharges of the two washes (161 cfs and 87 cfs) due to lags in the hydrographs related to time of concentration and times to peak and also as a result of the washes having miscellaneous storage throughout their lengths. The existing Flo2D models have been reviewed and it is our conclusion that the estimated 100-year peak discharges provided within the calculations are accurate, however, the City of Scottsdale is requiring that we utilized information from a more recent Flo2D model that has published discharges or 215 cfs, 114 cfs and 273 cfs. Additionally, the city is requiring that we use a multiplying factor of 1.5 for these flows to provide more conservative results.

An exhibit depicting proposed drainage easements has been provided as FIGURE 6 within the "Figures" section of this report. This exhibit depicts the location of the drainage easements that will be required to convey the peak discharges through this site. Proposed building pads will be established outside of the easement locations. An exhibit depicting the existing flood limits is also provided herein for reference.

With the exception of some very minor local washes collecting onsite runoff, there are no other significant washes impacting this site.

6.0 ONSITE DRAINAGE DESCRIPTION

In an effort to create more usable building pads, provide natural NAOS desert landscape corridors and protect the native desert surroundings within this area, the proposed subdivision lots will require some very minor re-routing of the washes within Lots 4 and 5. The washes will be re-routed in such a manner that the existing/historical entrance location and exit location will be undisturbed.

The washes will be designed to convey the 100-year peak discharges while maintaining near historical flow velocities and depths. Where necessary, the wash design will incorporate native angular rip-rap to assist in providing erosion protection along the banks and reducing flow velocities. Scour protection/bank protection calculations are provided herein.

In addition to allowing offsite drainage to pass through the site, the development will provide onsite retention for the areas of proposed disturbance. Retention calculations will be based on the 100-year, 2-hour rainfall event. Retention basins will be located on the individual lots and will not impact offsite flows or be comingled with offsite flows.

Onsite retention is being provided within 4 smaller retention basins located within the proposed platted lots. Due to the difficulty of draining the site to a regional basin, this was the preferred option to accommodate the required onsite retention. Lots 1, 2 and 6 will drain southwesterly and be collected within a basin located on the south side of Lot 6. Lots 3, 4 and 5 will each provide onsite retention within a basin located at the low end of each lot respectively. Ultimately, each basin will be designed with a metered bleed-off into the existing wash network to ensure the basins will drain within a 36-hour time frame. The basins will be maximum 3 feet deep and have side slopes not exceeding 4:1.

7.0 HYDROLOGY/HYDRAULICS

Onsite washes requiring re-routing will be designed in accordance with the City of Scottsdale Design Standards & Policies Manual for 100-year peak discharges. Portions of the wash requiring re-routing will be designed with maximum 3:1 side slopes and where velocities exceed 6 ft/sec will contain angular rip-rap bank protection and rip-rap within the bottom of the washes where necessary to reduce flow velocities and prevent erosion. The intent of the re-routed washes is to keep the flow velocities similar to historical rates. Calculations provided within the "Figures" section of the report (FIGURE 7) conclude that the angular rip-rap will not be required because of the minimum increase and in some instances a reduction in channel velocities from historical velocities.

A HEC-RAS model has been developed to demonstrate that the existing and proposed conditions have no adverse impact on existing upstream and downstream conditions. The HEC-RAS Generated Report is provided digitally on a CD within "FIGURES" section 8 of this report. A summary table depicting existing and proposed Water Surface Elevations and existing and proposed Velocities has also been provided in "FIGURES" section 7 to demonstrate a comparison for the existing and proposed site drainage conditions based on the HEC-RAS models. Refer to the Existing Conditions Floodplain Map and the Proposed Conditions Floodplain Map in "FIGURES" section 8 for specific cross section data from the HEC-RAS output model.

A Preliminary Grading and Drainage Plan has been provided in "FIGURES" section 8 pocket to schematically depict how the lots and surrounding areas will drain to the proposed retention areas. As the single lot residential development occurs, it will be necessary for future lot owners to provide individual grading and drainage plans to address localized/specific on lot flows to ensure that lots properly drain to each of the onsite retention basins dedicated for onsite runoff flows. Additionally, on Lot 4 & 5 it will be necessary to provide a "wet" wash crossing to allow ingress/egress to the building pad locations. These aforementioned single lot grading and drainage plans will be required to be submitted to the City of Scottsdale for review and approval to ensure the overall design and drainage intent is consistent with the Preliminary Grading and Drainage Plan, Preliminary Plat, Final Plat and Drainage Report. Erosion protection "cut off" walls at a minimum of 3-feet in depth shall be required on both sides of driveway wash crossings. Additionally, a minimum of a 10-foot wide area of angular native rip-rap shall be placed upstream and downstream of the "cut off" walls to provide additional erosion protection. This information will be prepared by separate plan and permit.

Lot 1 onsite storm drain pipe shall be designed to handle the 100-year peak discharge. Headwalls will be constructed at both upstream and downstream ends of the pipe section. Native angular rip-rap will also be utilized to reduce velocities and erosion at the entrance and exit locations of pipe sections. The pipe culvert section shown on the plans that cross the private roadway from Lot 1 to Lot 6 will be a minimum size 18" Circular Concrete Pipe. The pipe slope will be approximately 1% with a length of 237 linear feet. The capacity of the 18" pipe will be approximately 4 cfs. This meets or exceeds the required peak discharge for the small upstream/onsite drainage area. Lot 4 onsite storm drain pipes shall be designed to pass runoff such that less than 12" of water depth over tops the proposed driveway crossing. The storm drain pipes will be 24" diameter concrete pipes.

8.0 CONCLUSSIONS AND RECOMMEDATIONS

The proposed development does have offsite flows impacting the proposed site characteristics. As a result, some wash re-routing will occur to create more usable building pads, provide natural NAOS desert landscape corridors and protect the native desert surroundings within this area. Portions of the wash not requiring re-routing will remain undisturbed and be utilized as NAOS for the proposed development.

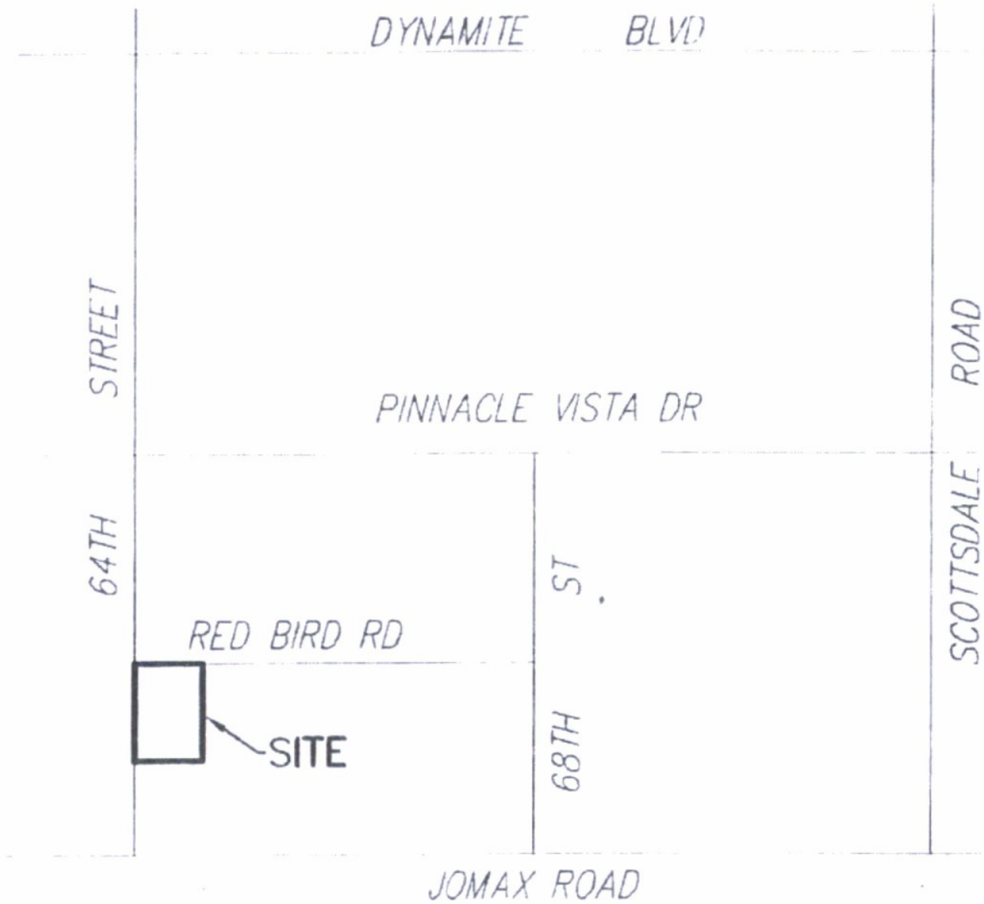
Wash entrance locations and exit locations will remain in their historical locations and conditions. The "wet crossing" at the southwest corner of the site will remain undisturbed as a result of this development. Although future offsite improvements are shown on the Preliminary Plat, they are not required to be constructed with this development plan.

Because of the difficulty of draining onsite runoff to a regional retention area, the proposed development will have 4 onsite retention basins that are designed to accommodate the onsite runoff that drains to each of the retention areas. These basins have been established at low points within the lots to ensure they receive onsite runoff. Ultimately, each basin will be designed with a metered bleed off pipe allowing the basin to drain within a 36-hour period.

Future lot owners will be required to provide individual grading and drainage plans to obtain building permits for their respective lots. Future development within the lots will be prohibited from modifying the washes (except for driveway crossings), therefore not impacting upstream or downstream capacities or velocities. All future Lowest Finished Floors shall be established at a minimum 1-foot above the highest adjacent grade or 1-foot above the high water elevation of the adjacent wash.

FIGURES

FIGURE 1

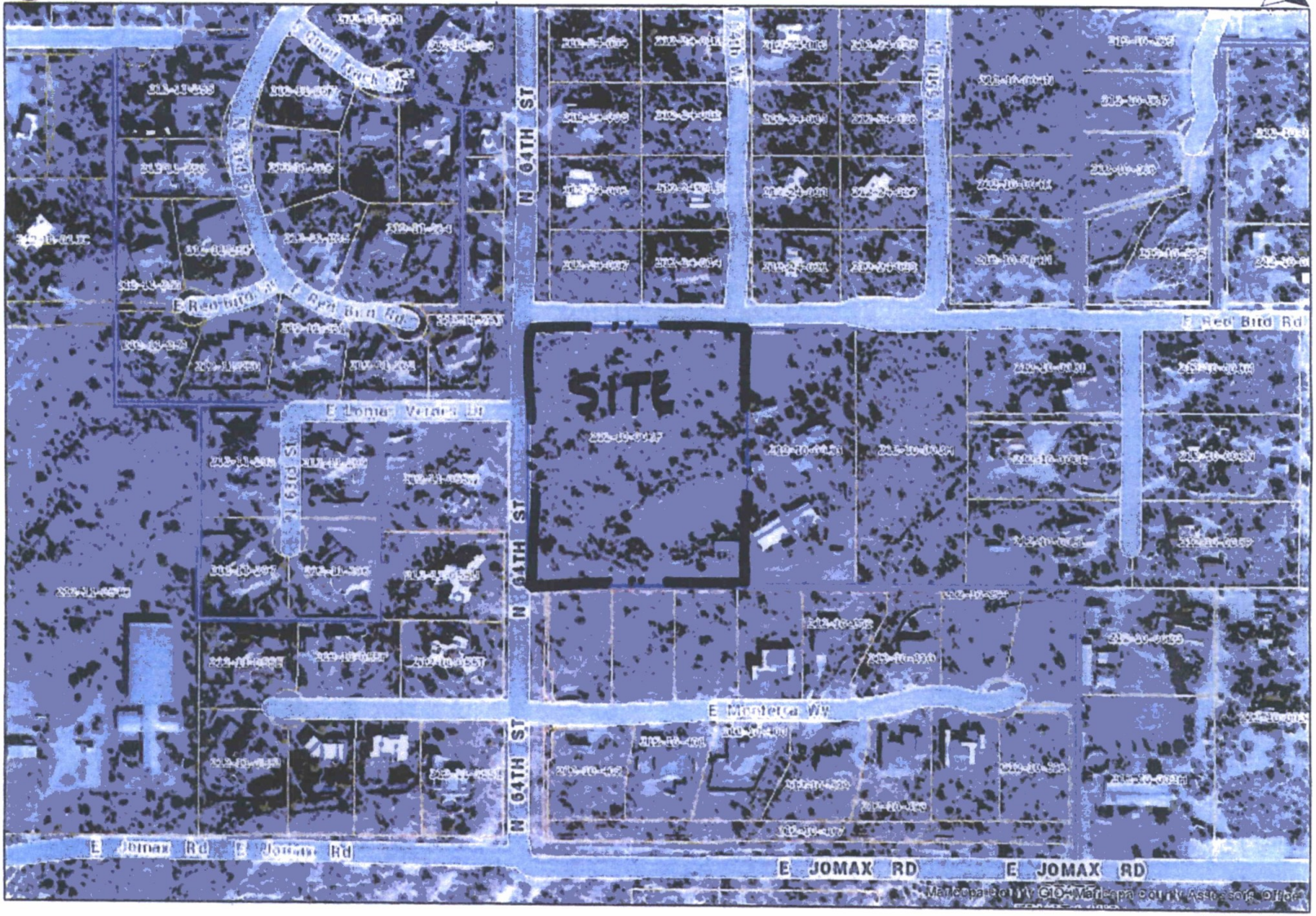


VICINITY MAP

FIGURE 1

FIGURE 2

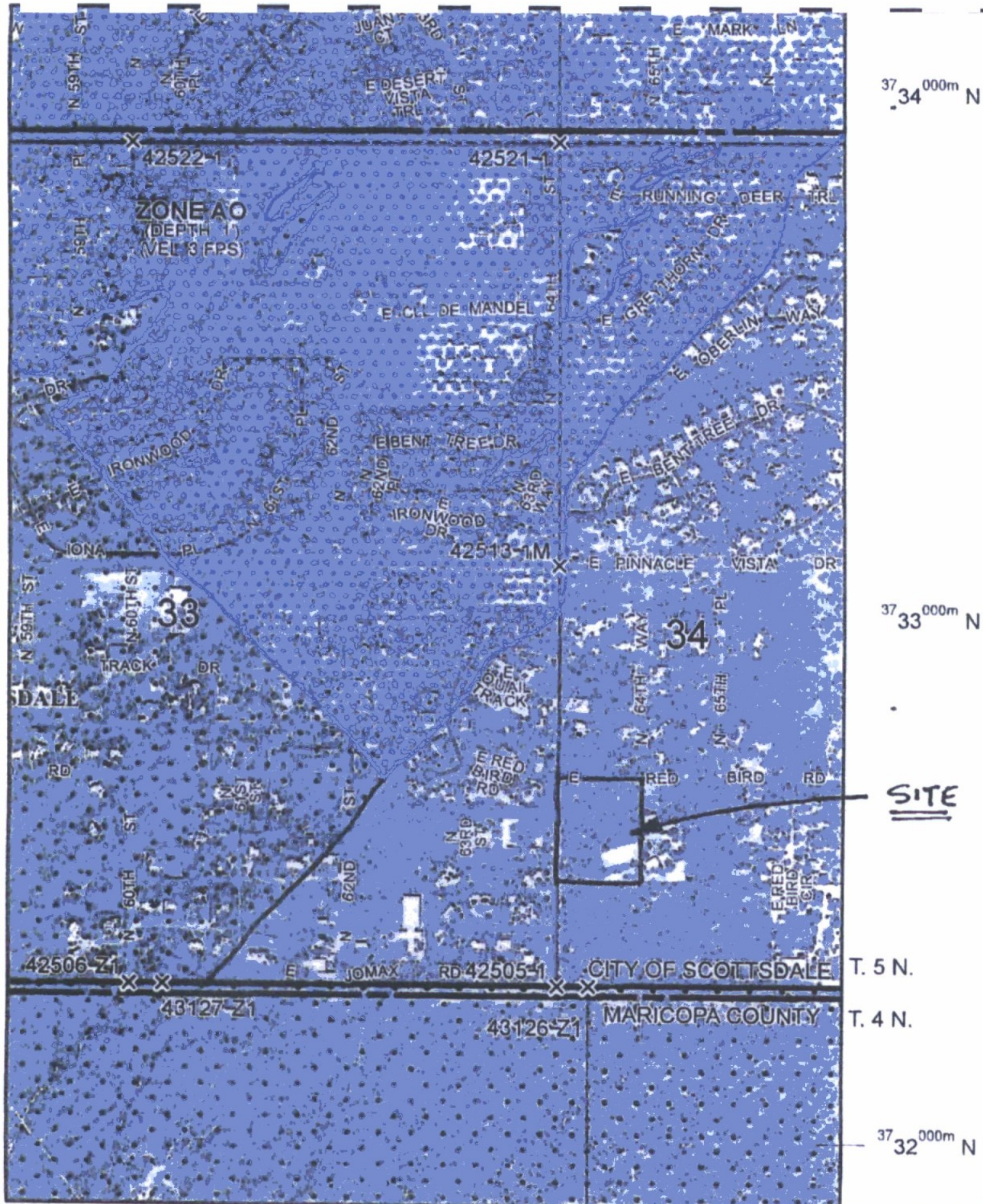
Map



1/25/2017 10:30:57 AM

FIGURE 2

FIGURE 3



MAP SCALE 1" = 1000'

500 0 1000 2000 FEET

METE

NFIP

PANEL 1305L

NATIONAL FLOOD INSURANCE PROGRAM

FIRM

FLOOD INSURANCE RATE MAP

MARICOPA COUNTY, ARIZONA

AND INCORPORATED AREAS

PANEL 1305 OF 4425

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
MARICOPA COUNTY	040037	1305	L
PHOENIX, CITY OF	040051	1305	L
SCOTTSDALE, CITY OF	045012	1305	L

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
04013C1305L

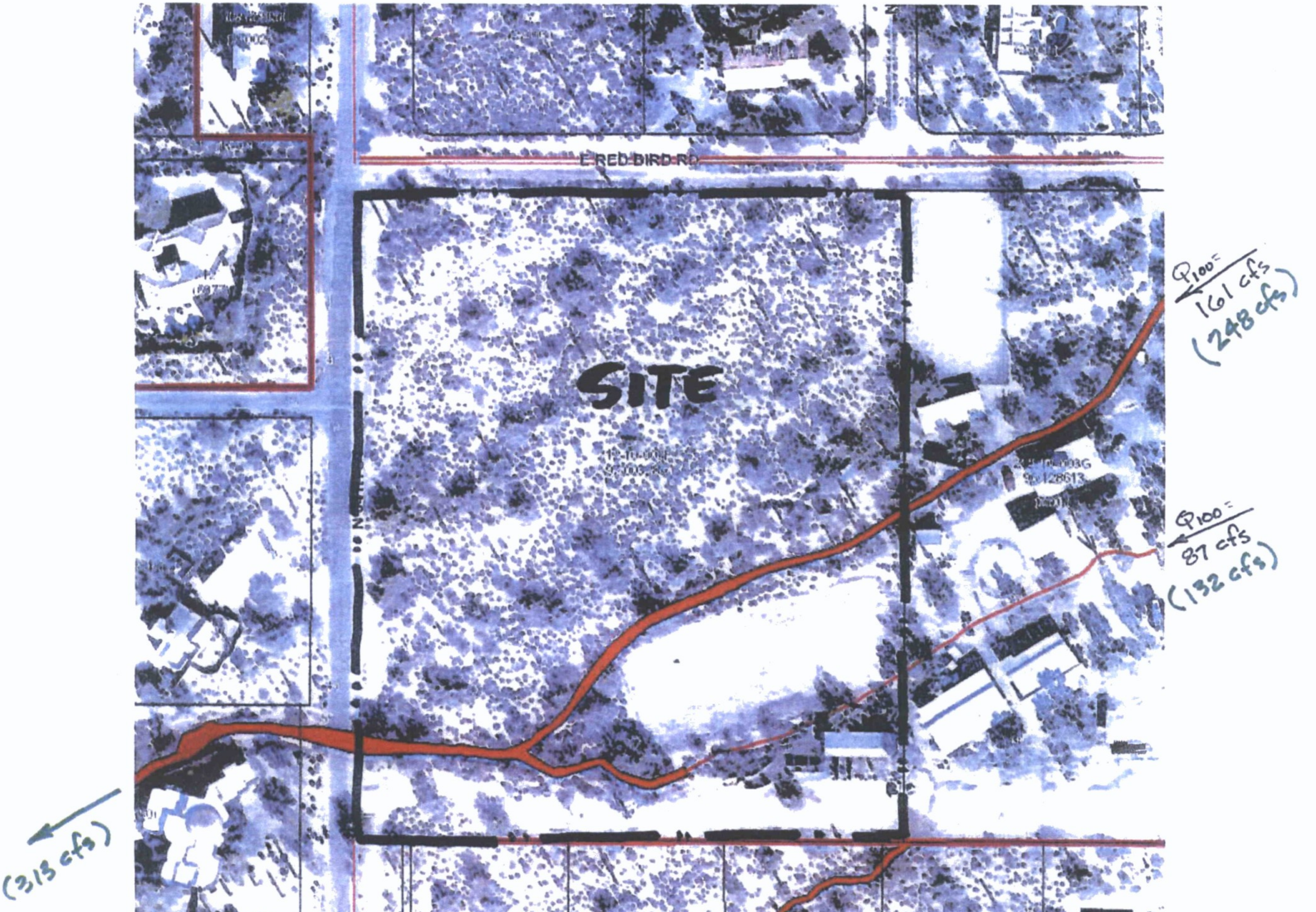
MAP REVISED
OCTOBER 16, 2013

Federal Emergency Management Agency

This is an official copy of a portion of the above referenced flood map. It was extracted using F-MIT On-Line. This map does not reflect changes or amendments which may have been made subsequent to the date on the title block. For the latest product information about National Flood Insurance Program flood maps check the FEMA Flood Map Store at www.msc.fema.gov

FIGURE 3

FIGURE 4



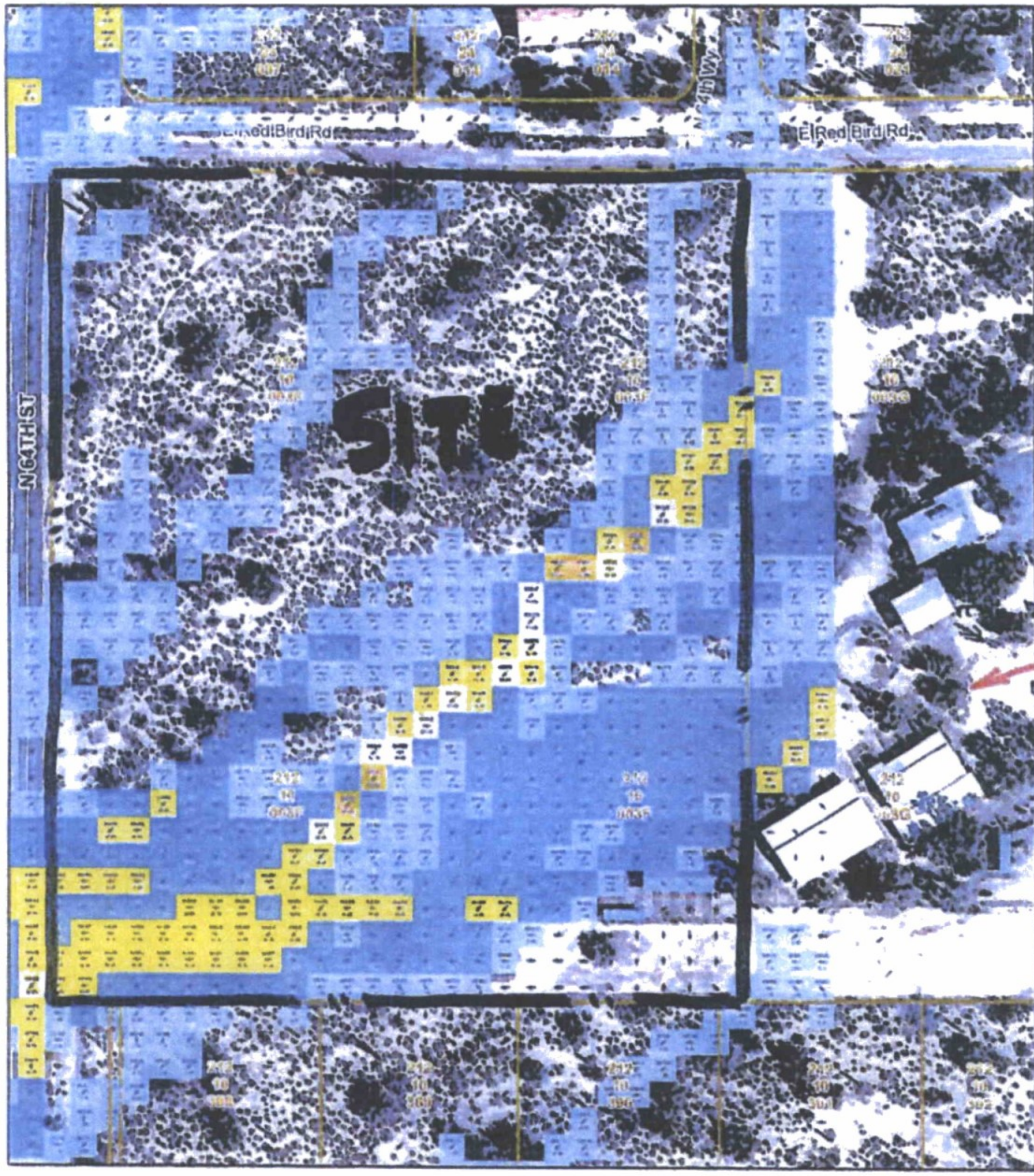
E RED BIRD RD
SITE
Q_{100} = 161 cfs
(248 cfs)
Q_{100} = 87 cfs
(132 cfs)
(313 cfs)

FIGURE 4

FIGURE 5

Flo2D Viewer

Model: flo2d_PinnaclePeakWestADMS/TR_100Y24H_WOW



Q_{100} = 161 cfs
(248 cfs)

Q_{100} = 87 cfs
(132 cfs)

(313 cfs)

1:1,500

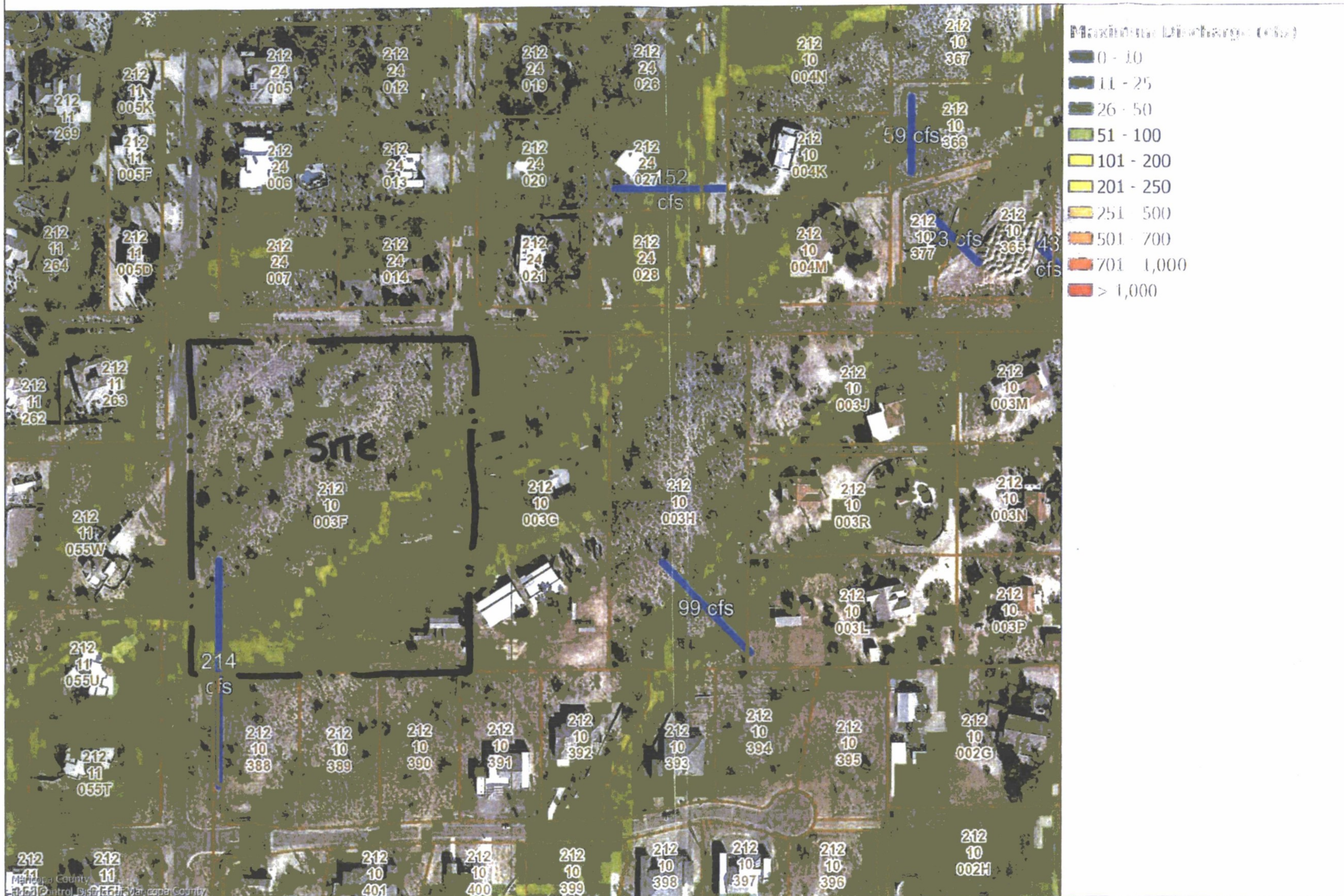
0 0.01 0.02 0.04 mi

DISCLAIMER The Flood Control District of Maricopa County (FCD) has made every reasonable effort to obtain and maintain this data as accurately as possible The FCD assumes no responsibility arising from the use of this information The data and maps are provided without warranty of any kind, either expressed or implied The FCD does not guarantee the accuracy, completeness, timeliness or correct sequencing of the data and information requested and hereby expressly disclaims any responsibility for the truth, lack of truth, validity, invalidity, accuracy, inaccuracy, errors or omissions or for the use or results obtained from the use of any said data and information You, the viewer or user, agree to indemnify the FCD, its officers, and employees from any liability that may arise from any such data or information in its actual or altered form Any download for commercial intent or resale of this information is prohibited except in accordance with a sublicensing agreement, and will be enforced in accordance with approved FCD policy and Arizona State Statutes 39-121-03 It is ultimately the viewer/users responsibility to verify accuracy prior to acceptance

Exported December 7, 2016 03:10 PM http://www.maricopa.gov

FIGURE 5

FLO-2D Model Results

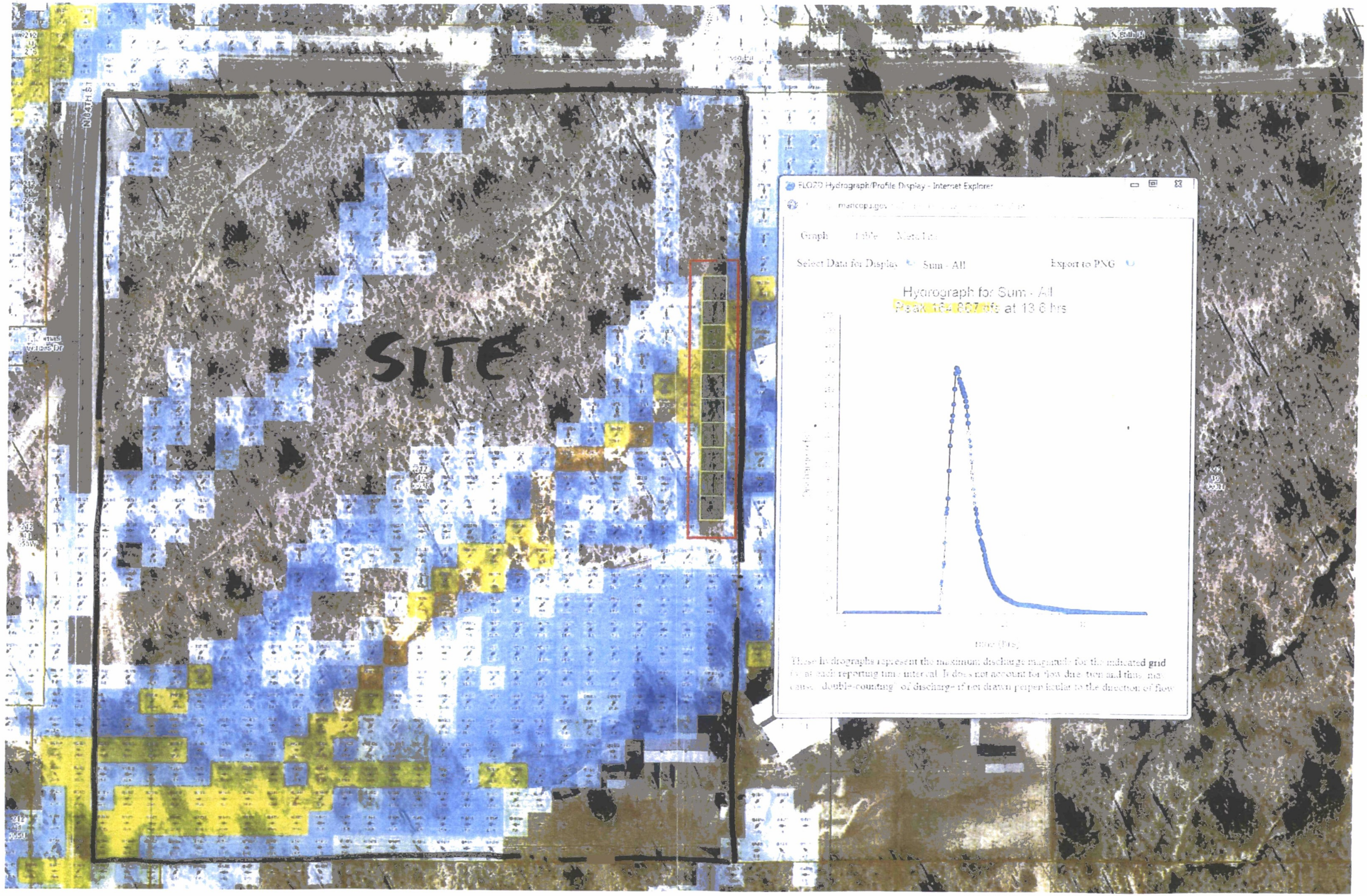


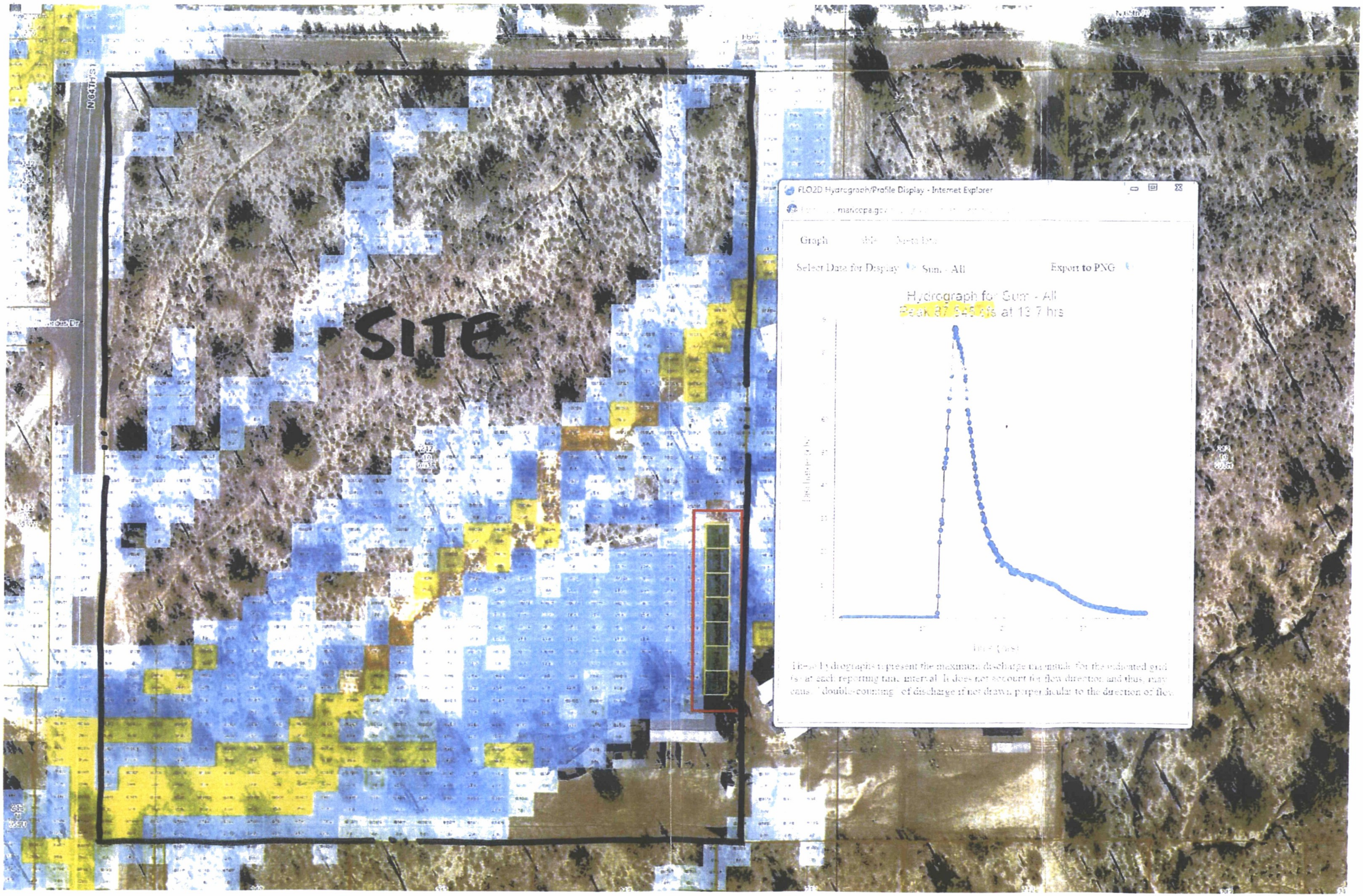
Flood Control District of Maricopa County

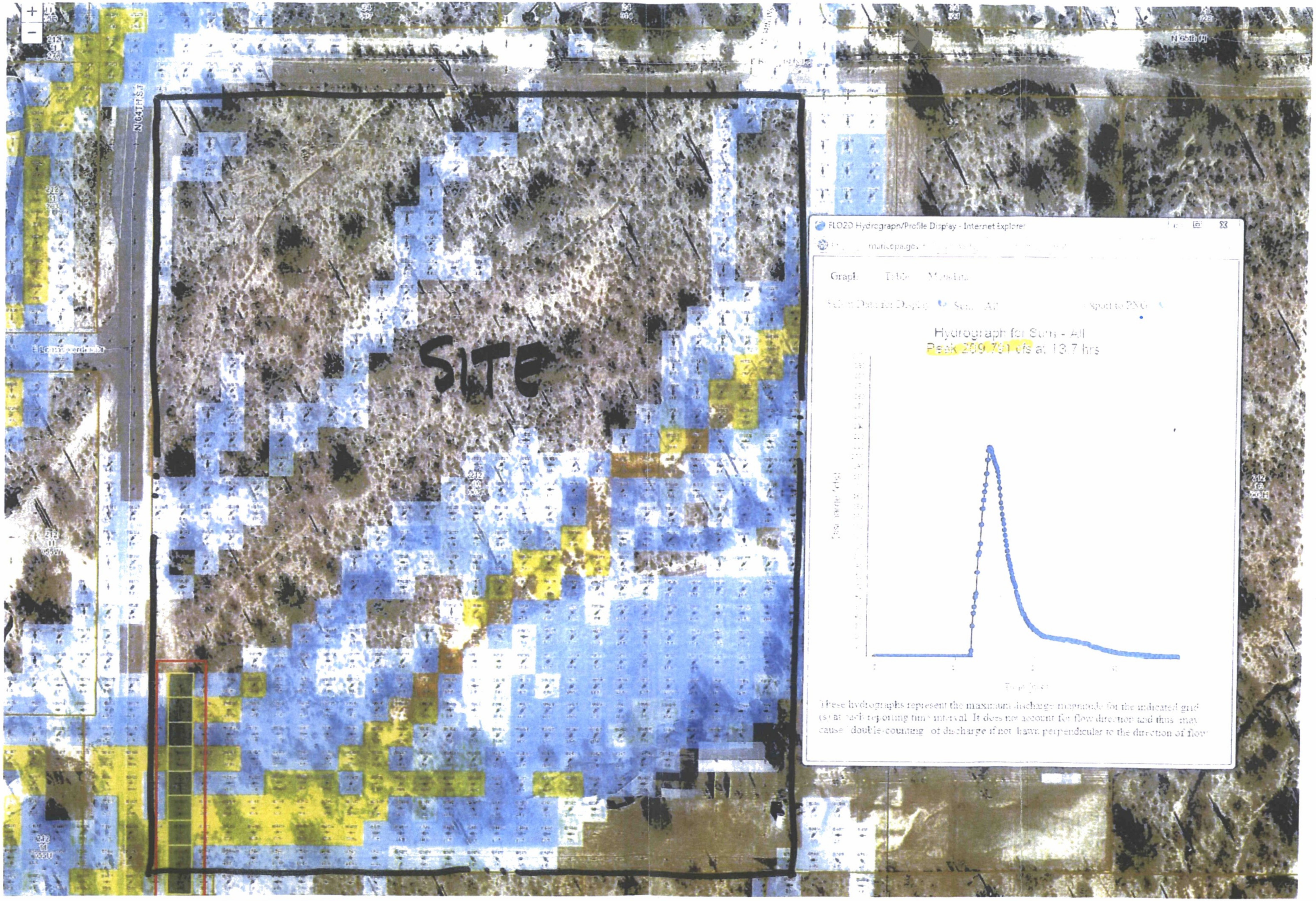
N

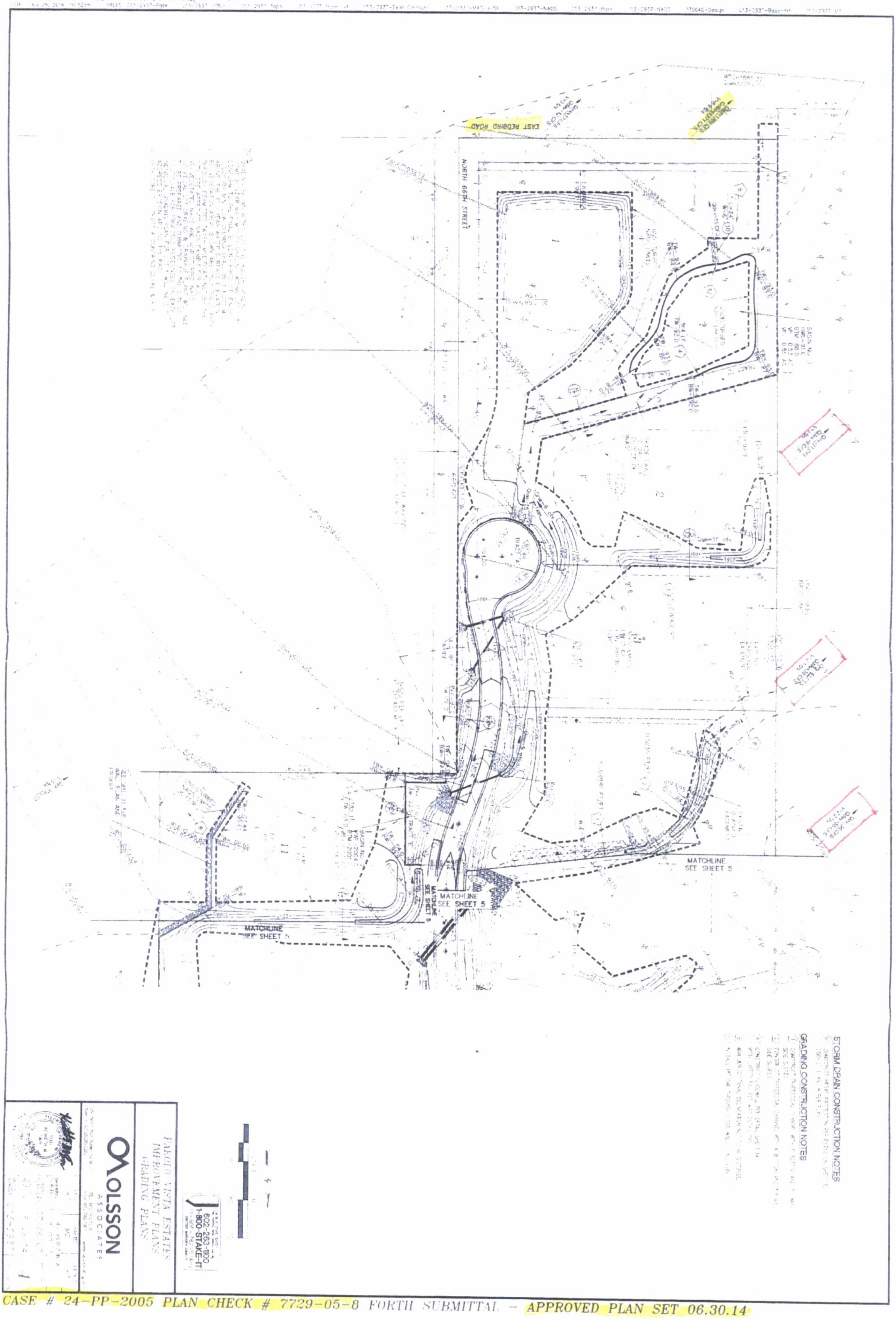
1 inch = 244 feet

0 130 260 520 ft


N 64th St
SITE
FLO2D Hydrograph/Profile Display - Internet Explorer
maricopa.gov
Graph
Select Data for Display
Sum - All
Export to PNG
Hydrograph for Sum - All
Peak 164.807 cfs at 13.8 hrs
time (hrs)
These hydrographs represent the maximum discharge magnitude for the indicated grid
at each reporting time interval. It does not account for flow direction and thus may
cause "double-counting" of discharge if not drawn perpendicular to the direction of flow


SITE
N 64TH ST
FLO2D Hydrograph/Profile Display - Internet Explorer
Graph
Select Data for Display
Sum - All
Export to PNG
Hydrograph for Sum - All
Peak 87.545 cfs at 13.7 hrs
These hydrographs represent the maximum discharge magnitude for the indicated grid
(s) at each reporting time interval. It does not account for flow direction and thus, may
cause "double-counting" of discharge if not drawn perpendicular to the direction of flow.


SITE
N 64TH ST
FLO2D Hydrograph/Profile Display - Internet Explorer
Graph
Hydrograph for Sum - All
These hydrographs represent the maximum discharge magnitude for the indicated grid (s) at each reporting time interval. It does not account for flow direction and thus may cause "double-counting" of discharge if not drawn perpendicular to the direction of flow.

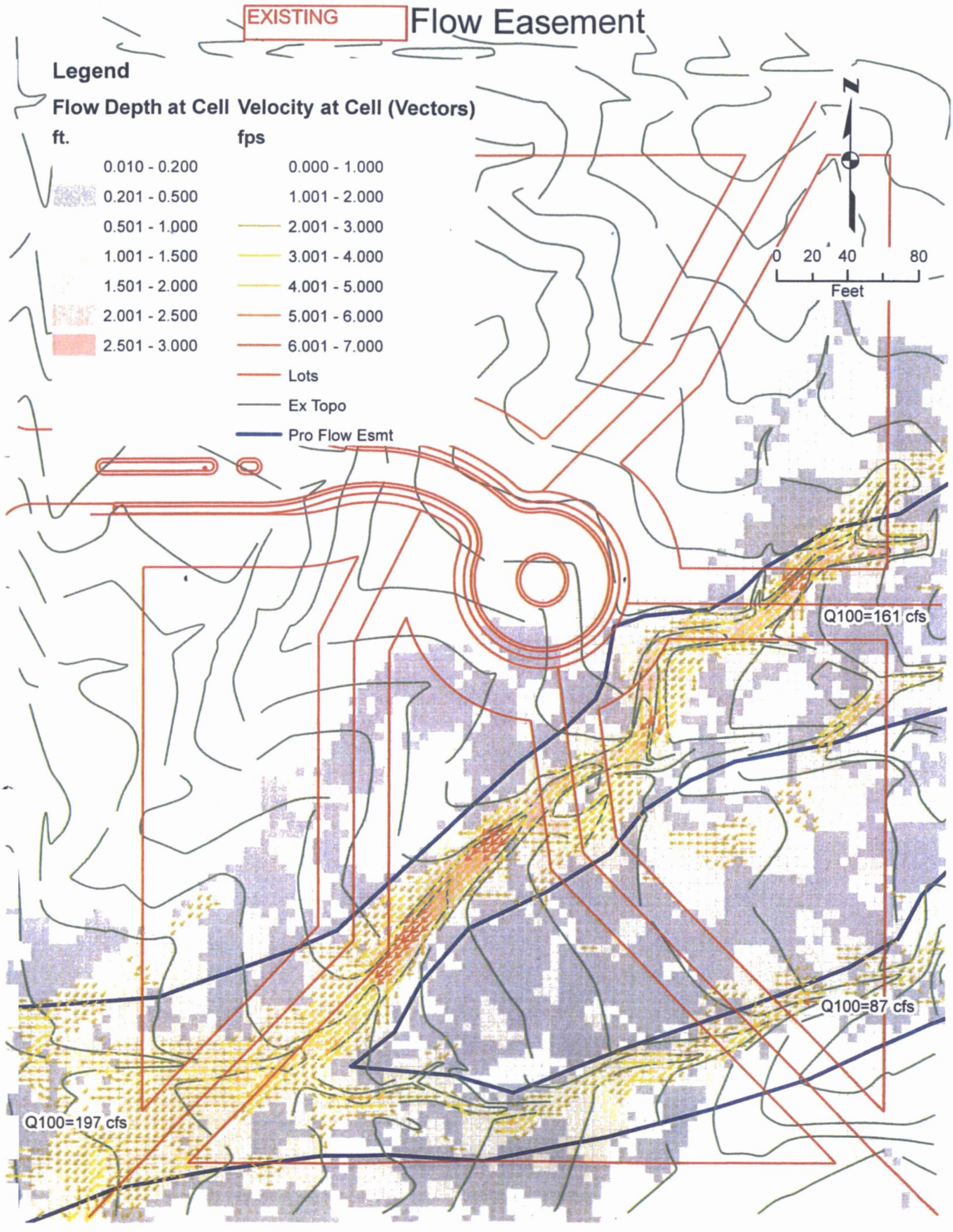


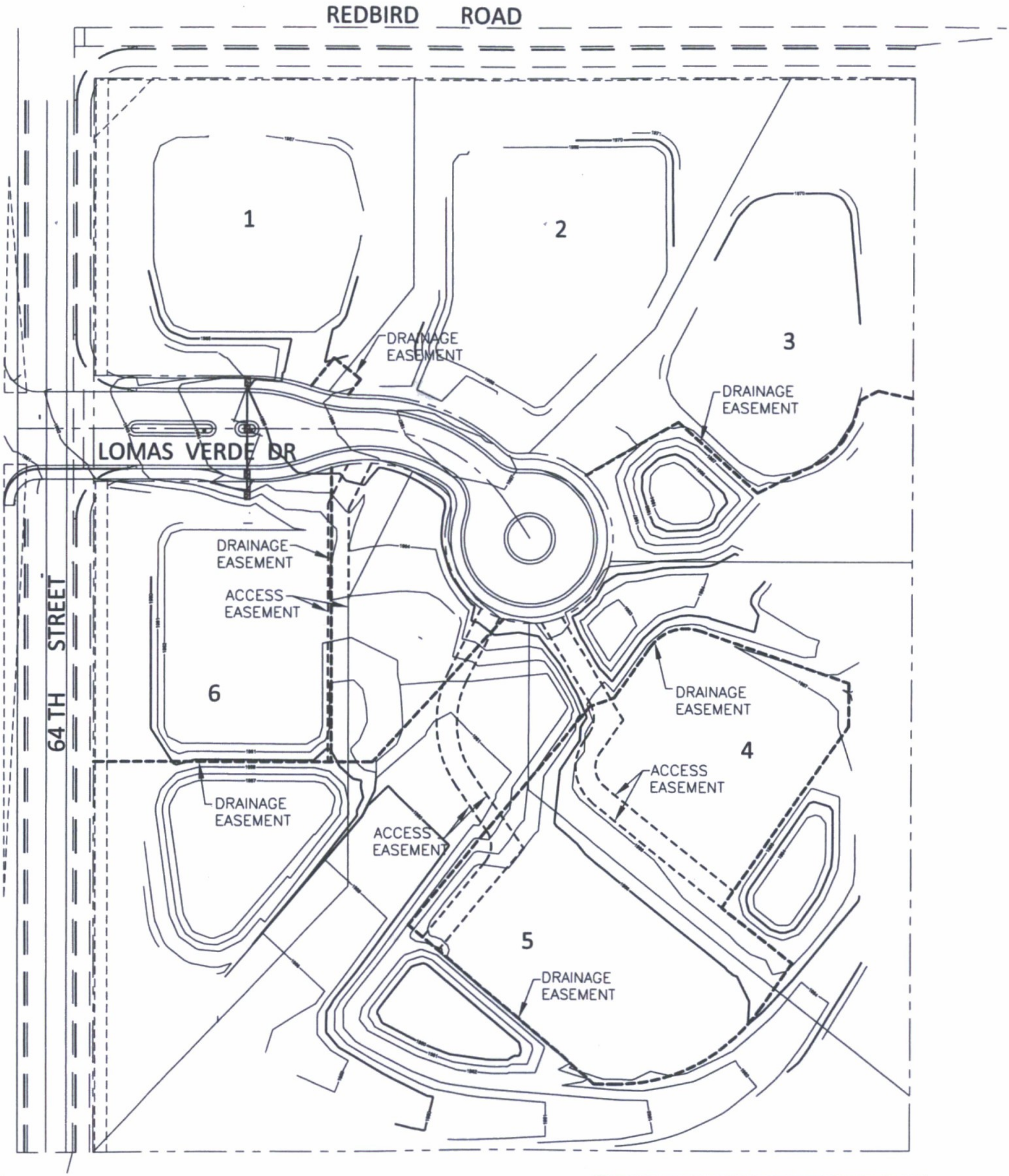
CASE # 24-PP-2005 PLAN CHECK # 7729-05-8 FORTH SUBMITTAL - APPROVED PLAN SET 06.30.14

PAROLO AT PINNICLE VISTA

HYDRAULIC WORK MAP

FIGURE 6


EXISTING Flow Easement
Legend
Flow Depth at Cell
ft.
0.010 - 0.200
0.201 - 0.500
0.501 - 1.000
1.001 - 1.500
1.501 - 2.000
2.001 - 2.500
2.501 - 3.000
Velocity at Cell (Vectors)
fps
0.000 - 1.000
1.001 - 2.000
2.001 - 3.000
3.001 - 4.000
4.001 - 5.000
5.001 - 6.000
6.001 - 7.000
Lots
Ex Topo
Pro Flow Esmt
N
0 20 40 80
Feet
Q100=161 cfs
Q100=87 cfs
Q100=197 cfs



DRAINAGE EASEMENT EXHIBIT

FIGURE 7

HEC-RAS Delta Table

Project: Lomas Verdes
Job No.: 16534.00

River	Reach	River Sta	Plan	W.S. Elev (ft)	Vel Chnl (ft/s)	Delta Elev	Delta Vel
SEE	South East Exten	507	Pro	1968.64	3.04	0.02	-0.17
SEE	South East Exten	507	Ex	1968.62	3.21		
SEE	South East Exten	457	Pro	1967.88	4.07	-0.02	-0.01
SEE	South East Exten	457	Ex	1967.9	4.08		
SEE	South East Exten	407	Pro	1966.88	2.84	0.06	-0.57
SEE	South East Exten	407	Ex	1966.82	3.41		
SEE	South East Exten	350	Pro	1965.63	4.15	-0.22	-0.5
SEE	South East Exten	350	Ex	1965.85	4.65		
SEE	South East Exten	300	Pro	1964.72	5.14	-0.29	0.67
SEE	South East Exten	300	Ex	1965.01	4.47		
SEE	South East Exten	250	Pro	1963.81	3.81	-0.39	-0.6
SEE	South East Exten	250	Ex	1964.2	4.41		
SEE	South East Exten	200	Pro	1963.1	4.54	0.02	1.02
SEE	South East Exten	200	Ex	1963.08	3.52		
SEE	South East Exten	150	Pro	1962.23	5.1	-0.09	0.66
SEE	South East Exten	150	Ex	1962.32	4.44		
SEE	South East Exten	100	Pro	1961.03	5	-0.09	-0.38
SEE	South East Exten	100	Ex	1961.12	5.38		
SEE	South East Exten	50	Pro	1960.29	4.66	0.02	0.1
SEE	South East Exten	50	Ex	1960.27	4.56		
One	Reach 1	150	Pro	1959.14	3.17	-0.24	-2.11
One	Reach 1	150	Ex	1959.38	5.28		
One	Reach 1	100	Pro	1958.59	6.17	0.06	0.56
One	Reach 1	100	Ex	1958.53	5.61		
One	Reach 1	50	Pro	1957.8	5.08	0.03	-0.78
One	Reach 1	50	Ex	1957.77	5.86		

One	Reach 1	0.39	Pro	1956.78	4.75	0.11	0.14
One	Reach 1	0.39	Ex	1956.67	4.61		
One	Reach 1	-50	Pro	1955.61	3.23	0	0
One	Reach 1	-50	Ex	1955.61	3.23		
One	Reach 1	-100	Pro	1955.08	3.73	0.01	-0.02
One	Reach 1	-100	Ex	1955.07	3.75		
One	Reach 1	-150	Pro	1954.37	4.08	0	0
One	Reach 1	-150	Ex	1954.37	4.08		
NEE	North East Exten	656	Pro	1971.53	4.65	0	0
NEE	North East Exten	656	Ex	1971.53	4.65		
NEE	North East Exten	606	Pro	1970.66	4.74	0	0
NEE	North East Exten	606	Ex	1970.66	4.74		
NEE	North East Exten	556	Pro	1969.27	2.24	0	-0.02
NEE	North East Exten	556	Ex	1969.27	2.26		
NEE	North East Exten	506	Pro	1969.08	2.27	0.03	-0.02
NEE	North East Exten	506	Ex	1969.05	2.29		
NEE	North East Exten	450	Pro	1967.76	6.16	-0.38	0.97
NEE	North East Exten	450	Ex	1968.14	5.19		
NEE	North East Exten	400	Pro	1966.5	3.86	-0.23	-1.31
NEE	North East Exten	400	Ex	1966.73	5.17		
NEE	North East Exten	350	Pro	1966.3	3.27	0.18	-2.62
NEE	North East Exten	350	Ex	1966.12	5.89		
NEE	North East Exten	300	Pro	1966.32	1.89	1.33	-3.67
NEE	North East Exten	300	Ex	1964.99	5.56		
NEE	North East Exten	287.*		1966.33	1.57		
NEE	North East Exten	261					
NEE	North East Exten	250		1964.32	4.47		
NEE	North East Exten	235.*		1962.96	5.37		
NEE	North East Exten	200	Pro	1962.24	5.06	-1.31	1.64
NEE	North East Exten	200	Ex	1963.55	3.42		
NEE	North East Exten	150	Pro	1961.31	4.78	-1.24	-1.76
NEE	North East Exten	150	Ex	1962.55	6.54		
NEE	North East Exten	100	Pro	1960.51	4.83	-0.92	0.7

NEE	North East Exten	100	Ex	1961.43	4.13		
NEE	North East Exten	50	Pro	1959.58	5.26	-0.96	0.17
NEE	North East Exten	50	Ex	1960.54	5.09		

Allowable Velocity and Lateral Migration Setback

Project:	**Lomas Verdes**
Job No.:	**16534.00**
Channel:	**3**

Allowable Velocity and Lateral Migration Setback Per ADWR
State Standard 5-96 Guideline 1. Assumes D75 of 4mm.

Allowable Velocity

$$V_a = V_b \times C_a \times C_b \times C_d$$

Where,

V_a	=	Maximum allowable 100-year velocity (ft/sec)
V_b	=	Basic maximum allowable flow velocity from Figure 1 (ft/sec)
C_a		
C_b	=	Correction factors from Figure 2 through 4
C_d		

Input

Sediment Laden

V_b	C_a	C_b	C_d
4	1	0.82	0.94

Sediment Free

V_b	C_a	C_b	C_d
2.5	1	0.82	0.94

Results

Sediment Laden

V_a
5

Sediment Free

V_a
3

Existing channel is wide spread braided sediment laden flow with velocities that range from 4.19-5.94.
Reach is slightly erosive and expected to generally be laterally stable.

Lateral Migration Setback

Straight reaches or reaches with minor curvature

$$SETBACK = 1.0(Q_{100})^{0.5}$$

Reaches with significant curvature or channel bends

$$SETBACK = 2.5(Q_{100})^{0.5}$$

Where,

SETBACK	=	Is the recommended setback (ft)
Q_{100}	=	100 year discharge (cfs)

Input

Minor Curvature

Q_{100}
132
248
315

Significant Curvature

Q_{100}
132
248
315

Results

Minor

Setback
11
16
18

Significant

Setback
29
39
44

Scour

Project: **Lomas Verdes**
Job No.: **16534.00**

Scour Per ADWR
State Standard 5-96 Guideline 2. Level I.
Channel Degradation Estimation for Alluvial Channels in Arizona

Scour

$$d_s = d_{gs} + d_{lts}$$

Where,

d_s	=	Total Scour Depth (ft.)
d_{gs}	=	General Degradation (ft.)
d_{lts}	=	Long Term Degradation (ft.)

$$d_{lts} = 0.02\,(Q_{100})^{0.6}$$

Straight reaches or reaches with minor curvature

$$d_{gs} = 0.157\,(Q_{100})^{0.4}$$

Reaches with significant curvature or channel bends

$$d_{gs} = 0.219\,(Q_{100})^{0.4}$$

Where,

Q_{100} = 100 year discharge (cfs)

Input

Minor Curvature

Q_{100}
132
248
315

Significant Curvature

Q_{100}
132
248
315

Results

Minor

Scour
1.5
2.0
2.2

Significant

Scour
1.9
2.5
2.8

Minimum shall be 3 feet

Date: 05/11/2017 Time: 07:42

```
*********************************************************
*        RIPRAP DESIGN SYSTEM (RDS)        *
*                BY                        *
*        WEST Consultants, Inc.            *
*                                          *
*                                          *
* Version 3.0              March, 2005     *
*                                          *
*                                          *
* COPYRIGHT (c) 2005                       *
* WEST CONSULTANTS, INC.                   *
* 16870 WEST BERNARDO DRIVE     PH: 858-487-9378 *
* SUITE 340                FAX:858-487-9448 *
* SAN DIEGO, CA 92127      WEB:WWW.WESTCONSULTANTS.COM *
*********************************************************
```

Project: Lomas Verdes CH3 150
Description: Section 150 of Channel Three

HEC-11 Method

Input Parameters:

Average Channel Velocity	4.27 ft/s
Average Flow Depth	0.89 ft
Unit Weight of Stone	165. lbs/cu ft
Cotangent of Side Slope	3.00
Material Angle of Repose	40.00 deg.
Riprap Placement	Channel Bank
Safety Factor	1.2

Output Results:

Computed D50 0.10 ft

** FHWA Gradation**

Gradation Class Facing
Layer Thickness 1.90 ft

Percent Smaller by Size	Rock Size, ft	Rock Weight, lbs
D100	1.30	200.
D50	0.95	75.
D10	0.40	5.

Date: 05/11/2017 Time: 07:46

****

* RIPRAP DESIGN SYSTEM (RDS) *

* BY *

* WEST Consultants, Inc. *

* *

* *

* Version 3.0 March, 2005 *

* *

* *

* WEST CONSULTANTS, INC. *

* 16870 WEST BERNARDO DRIVE PH: 858-487-9378 *

* SUITE 340 FAX:858-487-9448 *

* SAN DIEGO, CA 92127 WEB:WWW.WESTCONSULTANTS.COM *

****

Project: Lomas Verdes CH3 350

Description: Section 350 of Channel Three

HEC-11 Method

Input Parameters:

Average Channel Velocity	4.38 ft/s
Average Flow Depth	0.95 ft
Unit Weight of Stone	165. lbs/cu ft
Cotangent of Side Slope	3.00
Material Angle of Repose	40.00 deg.
Riprap Placement	Channel Bank
Safety Factor	1.2

Output Results:

Computed D50 0.11 ft

** FHWA Gradation**

Gradation Class Facing

Layer Thickness 1.90 ft

Percent Smaller by Size	Rock Size, ft	Rock Weight, lbs
D100	1.30	200.
D50	0.95	75.
D10	0.40	5.

Date: 05/11/2017 Time: 07:48

```
**********************************************************
*        RIPRAP DESIGN SYSTEM (RDS)        *
*              BY              *
*          WEST Consultants, Inc.          *
*                              *
*                              *
* Version 3.0              March, 2005 *
*                              *
*                              *
* COPYRIGHT (c) 2005                       *
* WEST CONSULTANTS, INC.                   *
* 16870 WEST BERNARDO DRIVE       PH: 858-487-9378 *
* SUITE 340              FAX:858-487-9448 *
* SAN DIEGO, CA 92127     WEB:WWW.WESTCONSULTANTS.COM *
**********************************************************
```

Project: Lomas Verdes CH 1 100
Description: Section 100 of Channel One

HEC-11 Method

Input Parameters:

Average Channel Velocity	5.95 ft/s
Average Flow Depth	1.67 ft
Unit Weight of Stone	165. lbs/cu ft
Cotangent of Side Slope	3.00
Material Angle of Repose	40.00 deg.
Riprap Placement	Channel Bank
Safety Factor	1.2

Output Results:

Computed D50 0.20 ft

** FHWA Gradation**

Gradation Class	Facing
Layer Thickness	1.90 ft

Percent Smaller by Size	Rock Size, ft	Rock Weight, lbs
D100	1.30	200.
D50	0.95	75.
D10	0.40	5.

Date: 05/11/2017 Time: 07:50

```
*********************************************************
*         RIPRAP DESIGN SYSTEM (RDS)           *
*                 BY                  *
*          WEST Consultants, Inc.         *
*                                     *
*                                     *
* Version 3.0                March, 2005 *
*                                     *
*                                     *
* COPYRIGHT (c) 2005                 *
* WEST CONSULTANTS, INC.                  *
* 16870 WEST BERNARDO DRIVE        PH: 858-487-9378 *
* SUITE 340                FAX:858-487-9448 *
* SAN DIEGO, CA 92127     WEB:WWW.WESTCONSULTANTS.COM *
*********************************************************
```

Project: Lomas Verdes CH2 100
Description: Section 100 of Channel Two

HEC-11 Method

Input Parameters:

Average Channel Velocity	4.97 ft/s
Average Flow Depth	1.96 ft
Unit Weight of Stone	165. lbs/cu ft
Cotangent of Side Slope	3.00
Material Angle of Repose	40.00 deg.
Riprap Placement	Channel Bank
Safety Factor	1.2

Output Results:

Computed D50	0.11 ft

** FHWA Gradation**

Gradation Class	Facing
Layer Thickness	1.90 ft

Percent Smaller by Size	Rock Size, ft	Rock Weight, lbs
D100	1.30	200.
D50	0.95	75.
D10	0.40	5.

Date: 05/11/2017 Time: 07:53

```
*********************************************************
*         RIPRAP DESIGN SYSTEM (RDS)              *
*                BY                    *
*          WEST Consultants, Inc.          *
*                                      *
*                                      *
* Version 3.0                March, 2005 *
*                                      *
*                                      *
* COPYRIGHT (c) 2005                    *
* WEST CONSULTANTS, INC.                 *
* 16870 WEST BERNARDO DRIVE        PH: 858-487-9378 *
* SUITE 340              FAX:858-487-9448 *
* SAN DIEGO, CA 92127     WEB:WWW.WESTCONSULTANTS.COM *
*********************************************************
```

Project: Lomas Verdes CH2 350

Description: Section 350 of Channel Two

HEC-11 Method

Input Parameters:

Average Channel Velocity	5.76 ft/s
Average Flow Depth	1.53 ft
Unit Weight of Stone	165. lbs/cu ft
Cotangent of Side Slope	3.00
Material Angle of Repose	40.00 deg.
Riprap Placement	Channel Bank
Safety Factor	1.2

Output Results:

Computed D50 0.19 ft

** FHWA Gradation**

Gradation Class Facing

Layer Thickness 1.90 ft

Percent Smaller by Size	Rock Size, ft	Rock Weight, lbs
D100	1.30	200.
D50	0.95	75.
D10	0.40	5.

Channel Freeboard

Project: **Lomas Verdes**
Job No.: **16534.00**

Channel Freeboard per Maracopa County Drainage Policies and Standards
Standard 6.8.7

$$FB = 0.25\left(Y + \frac{V^2}{2g}\right)$$

Where,

FB	=	Freeboard in Feet
Y	=	Flow Depth in Feet
V	=	Velocity in fps
g	=	Acceleration Due to Gravity in ft/s^2

River	Reach	River Sta	Min Ch El (ft)	W.S. Elev (ft)	Vel Chnl (ft/s)	Freeboard (ft)
SEE	South East Exten	507	1967.81	1968.64	3.04	0.24
SEE	South East Exten	457	1967	1967.88	4.07	0.28
SEE	South East Exten	407	1966	1966.88	2.84	0.25
SEE	South East Exten	350	1964.51	1965.63	4.15	0.35
SEE	South East Exten	300	1963.69	1964.72	5.14	0.36
SEE	South East Exten	250	1962.77	1963.81	3.81	0.32
SEE	South East Exten	200	1961.94	1963.1	4.54	0.37
SEE	South East Exten	150	1961.19	1962.23	5.1	0.36
SEE	South East Exten	100	1960.02	1961.03	5	0.35
SEE	South East Exten	50	1959.24	1960.29	4.66	0.35

One	Reach 1	150	1957.52	1959.14	3.17	0.44
One	Reach 1	100	1956.92	1958.59	6.17	0.57
One	Reach 1	50	1956.71	1957.8	5.08	0.37
One	Reach 1	0.39	1955.93	1956.78	4.75	0.30
One	Reach 1	-50	1954.01	1955.61	3.23	0.44
One	Reach 1	-100	1953.94	1955.08	3.73	0.34
One	Reach 1	-150	1953.09	1954.37	4.08	0.38
NEE	North East Exten	656	1970.57	1971.53	4.65	0.32
NEE	North East Exten	606	1969.75	1970.66	4.74	0.31
NEE	North East Exten	556	1968.66	1969.27	2.24	0.17
NEE	North East Exten	506	1966.99	1969.08	2.27	0.54
NEE	North East Exten	450	1965.94	1967.76	6.16	0.60
NEE	North East Exten	400	1964.69	1966.5	3.86	0.51
NEE	North East Exten	350	1963.63	1966.3	3.27	0.71
NEE	North East Exten	300	1962.86	1966.32	1.89	0.88
NEE	North East Exten	287.*	1962.65	1966.33	1.57	0.93
NEE	North East Exten	235.*	1961.82	1962.96	5.37	0.40
NEE	North East Exten	200	1961.26	1962.24	5.06	0.34
NEE	North East Exten	150	1960.36	1961.31	4.78	0.33
NEE	North East Exten	100	1959.49	1960.51	4.83	0.35
NEE	rth East Ex	50	1958.61	1959.58	5.26	0.35

Retention Estimate

Project Name: Lomas Verdes
Project #: 16534

Calculate 100-Year 2-Hour Retention Volume Requirement

Use method provided by Section 4-1.807 of the City of Scottsdale Design Standards & Policies Manual

$$V_r = \frac{P}{12} AC$$

BASIN 'A'
Lots 1, 2, 6

P	A	C
2.50	3.91	0.62

V_r (ac.ft)	V_r (ft^3)
0.505	22,000

Lot 3 – BASIN 'B'

P	A	C
2.50	1.02	0.62

V_r (ac.ft)	V_r (ft^3)
0.132	5,739

Lot 4 – BASIN 'C'

P	A	C
2.50	1.25	0.62

V_r (ac.ft)	V_r (ft^3)
0.161	7,033

Lot 5 – BASIN 'D'

P	A	C
2.50	1.82	0.62

V_r (ac.ft)	V_r (ft^3)
0.235	10,240

LOMAS VERDE ESTATES - DRAINAGE CALCULATIONS								
BASIN		VOL REQ		VOL PROV		SURFACE AREAS		
#	LOTS	AC FT	CU FT	AC FT	CU FT	BOT (SF)	TOP (SF)	DEPTH
A	1,2,6	0.505	22,000	0.513	22,368	5,440	9,472	3
B	3	0.132	5,739	0.150	6,521	1,033	3,314	3
C	4	0.161	7,033	0.176	7,661	1,235	3,872	3
D	5	0.235	10,240	0.262	11,397	2,190	5,408	3

2. Time of Concentration

Time of concentration "**Tc**" is the total time of travel from the most hydraulically remote part of the watershed to the concentration point of interest. The calculation of "Tc" must follow FCDMC Hydrology Manual procedures.

***Note: Do not add a standard set amount of time to the estimated "Tc" for lot runoff delay (such as 5 or 10 minutes).** Natural land slopes are too variable in Scottsdale to add a set amount of time for lot runoff.

3. Runoff Coefficients

Use Figure 4.1-4 or equivalent to obtain the runoff coefficients or "**C**" values. Composite "**C**" values for the appropriate zoning category or weighted average values calculated for the specific site are both acceptable approaches.

RUNOFF COEFFICIENTS - "C" VALUE			
Land Use	Storm Frequency		
Composite Area-wide Values	2-25 Year	50 Year	100 Year
Commercial & Industrial Areas	0.80	0.83	0.86
Residential Areas-Single Family (average lot size)			
R1-1-1901	0.33	0.50	0.53
R1-130	0.35	0.51	0.59
R1-70	0.37	0.52	0.60
R1-43	0.38	0.55	0.61
R1-35 (35,000 square feet/lot)	0.40	0.56	0.62
R1-18 (18,000 square feet/lot)	0.43	0.58	0.64
R1-10 (10,000 square feet/lot)	0.47	0.62	0.67
R1-7 (7,000 square feet/lot)	0.51	0.64	0.94
Townhouses (R-2, R-4)	0.63	0.74	0.94
Apartments & Condominiums (R-3, R-5)	0.76	0.83	0.94
Specific Surface Type Values			
Paved streets, parking lots (concrete or asphalt), roofs, drive-ways, etc.	0.90	0.93	0.95
Lawns, golf courses, & parks (grassed areas)	0.20	0.25	0.30
Undisturbed natural desert or desert landscaping (no impervious weed barrier)	0.37	0.42	045
Desert landscaping (with impervious weed barrier)	0.63	0.73	0.83
Mountain terrain – slopes greater than 10%	0.60	0.70	0.80
Agricultural areas (flood-irrigated fields)	0.16	0.18	0.20

FIGURE 4.1-4 RUNOFF COEFFICIENTS FOR USE WITH RATIONAL METHOD

a. Increasing the percent impervious on the L card to reflect the amount of impervious surfaces that will exist under fully developed conditions
b. Recalculate the time of concentration (**Tc**) based on the proposed drainage system, after full development. Normally there should be a reduction in **Tc** after development
c. The existing condition model must be sub-divided, as necessary, to create concentration points which will match the sub-watershed areas above each proposed storage facility under fully developed conditions
d. Each separate storage facility proposed must be modeled as it will physically exist under fully developed conditions with appropriate routing and combining operations through each basin and through the entire watershed. The modeling of storage capacity provided, as one hypothetical reservoir at the outlet with all the upstream storage arbitrarily combined at this one location, is not acceptable
e. As a minimum, the 2, 10 and 100-year frequency events shall be analyzed
f. Comparison of discharge values for existing and post development conditions must be made at concentration points just downstream from each proposed storage facility; other critical locations such as road crossings; and at points where flows exit the proposed development.

4-1.807 CALCULATION OF RUNOFF VOLUMES

The only accepted method for determining the required stormwater storage volume is the standard formula described below. HEC-1 modeling can be used for storage basin design and analysis, or if a pre-versus post volume difference is needed. City ordinance requires on-site storage of runoff from the 100-year, 2-hour frequency event.

A. Standard Formula for Runoff Volumes

$$Vr = (P/12)\,AC$$

Vr	=	*Required storage volume in acre-feet.*
P	=	*Precipitation amount = The depth of the 100-year 2-hour rainfall, from figure in Appendix 4-1D at the site.*
A	=	*Area in acres; the developed portion of the entire site in acres, to the centerline of adjacent streets, on which any man made change is planned, including, but not limited to: construction, excavation, filling, grading, paving, or mining.*
C	=	*Runoff coefficient; Rational Method values from Figure 4.1-4.*

B. HEC-1 Computer Modeling

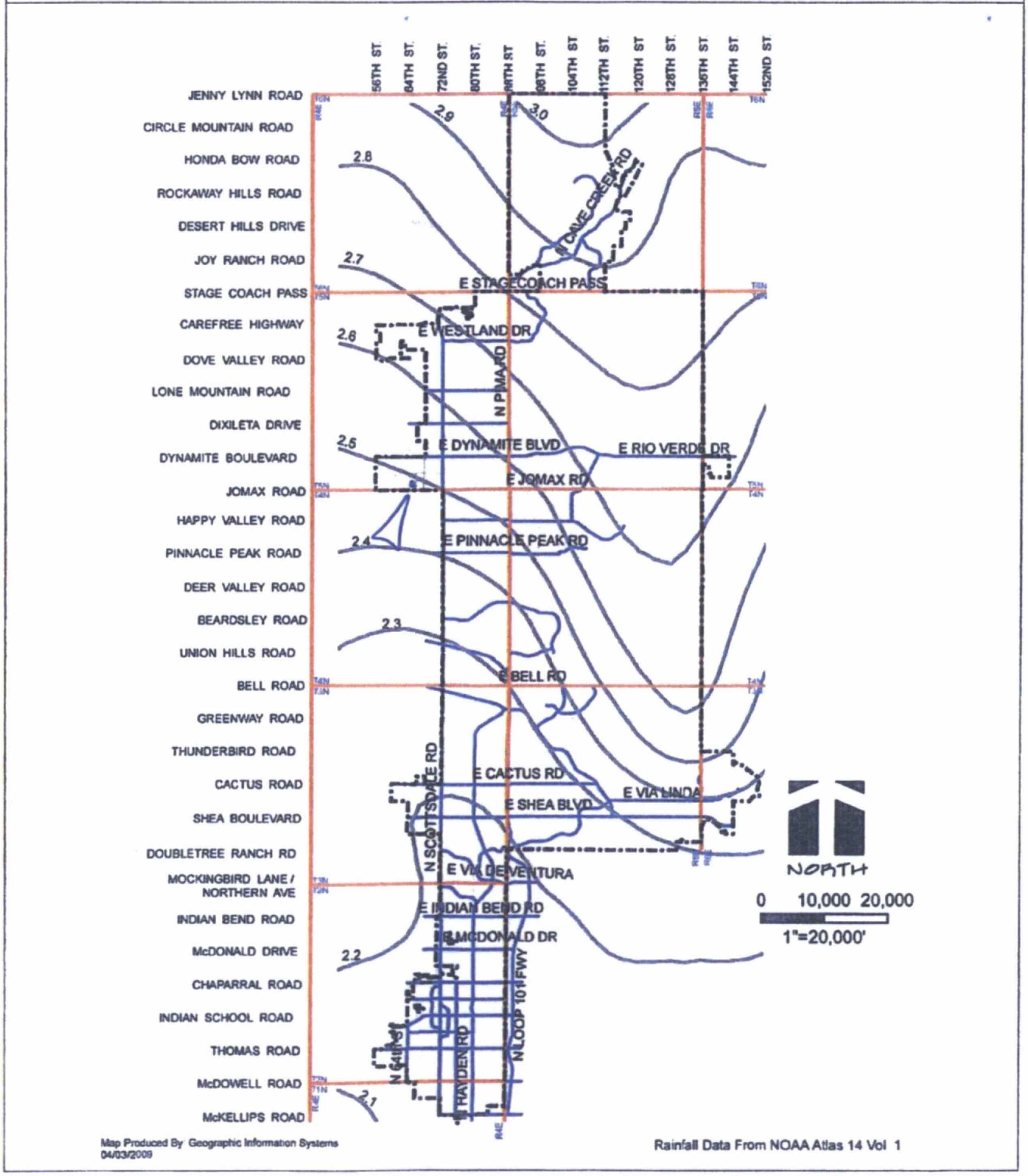
The HEC-1 model or similar computer program is not to be used to determine the ordinance required 100-year, 2-hour stormwater storage runoff volumes. The HEC-1 program may be used for the purpose of analyzing storage basin routing or for pre versus post analysis (a six-hour storm; procedures described in Section 4-1.806 paragraphs D and E must be used). Use modified Puls level pool routing option in HEC-1 for hydrograph routing through storage basins and lakes. For permanent lakes assume no available storage below the normal water surface elevation.

CAUTION: Do not use the built-in orifice equation in the HEC-1 model because errors can result. It is necessary to build a stage discharge table and input to the model.

4-1.808 METHODS FOR ESTIMATING WATER SURFACE ELEVATIONS AND INUNDATION LIMITS

The engineer may use any standard method for the determination of water surface elevations. Only the U.S. Army Corps of Engineers' HEC-2, Water Surface Profiles program and the HEC-RAS, River Analysis System are supported by the City. Prior approval by city staff is required for the use of other methods.

100 Year 2 Hour Precipitation in Inches



Culvert Analysis Report
Culvert-1

Culvert Summary

Computed Headwater Elev:	1.78 ft	Discharge	4.00 cfs
Inlet Control HW Elev.	1.73 ft	Tailwater Elevation	N/A ft
Outlet Control HW Elev.	1.78 ft	Control Type	Entrance Control
Headwater Depth/Height	0.75		

Grades

Upstream Invert	0.65 ft	Downstream Invert	0.00 ft
Length	65.00 ft	Constructed Slope	0.010000 ft/ft

Hydraulic Profile

Profile	S2	Depth, Downstream	0.64 ft
Slope Type	Steep	Normal Depth	0.64 ft
Flow Regime	Supercritical	Critical Depth	0.77 ft
Velocity Downstream	5.54 ft/s	Critical Slope	0.005413 ft/ft

Section

Section Shape	Circular	Mannings Coefficient	0.013
Section Material	Concrete	Span	1.50 ft
Section Size	18 inch	Rise	1.50 ft
Number Sections	1		

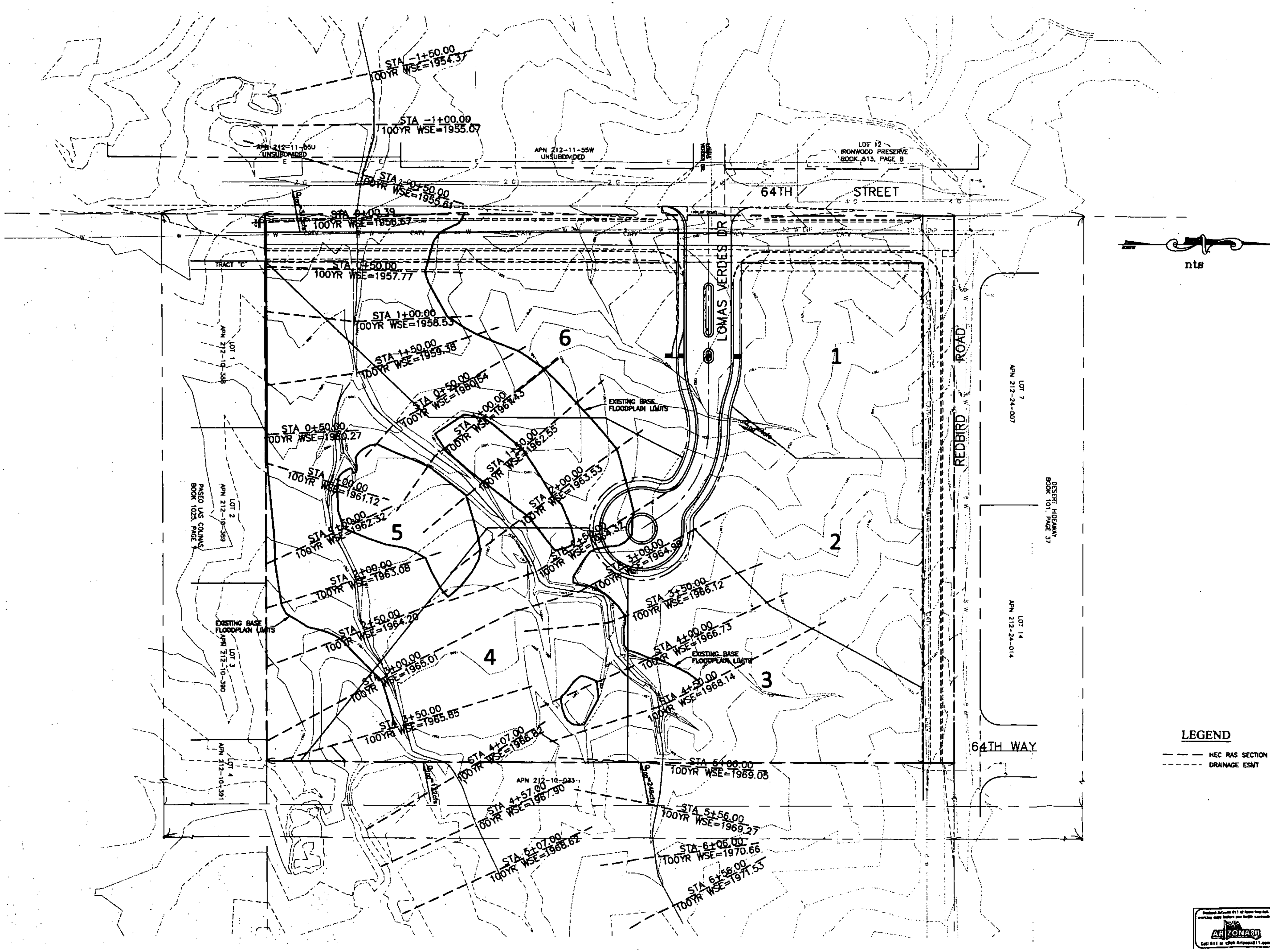
Outlet Control Properties

Outlet Control HW Elev.	1.78 ft	Upstream Velocity Head	0.30 ft
Ke	0.20	Entrance Loss	0.06 ft

Inlet Control Properties

Inlet Control HW Elev.	1.73 ft	Flow Control	Unsubmerged
Inlet Type	Groove end projecting	Area Full	1.8 ft²
K	0.00450	HDS 5 Chart	1
M	2.00000	HDS 5 Scale	3
C	0.03170	Equation Form	1
Y	0.69000		

FIGURE 8



LOMAS VERDES ESTATES
6501 E. REDBIRD ROAD, SCOTTSDALE, AZ

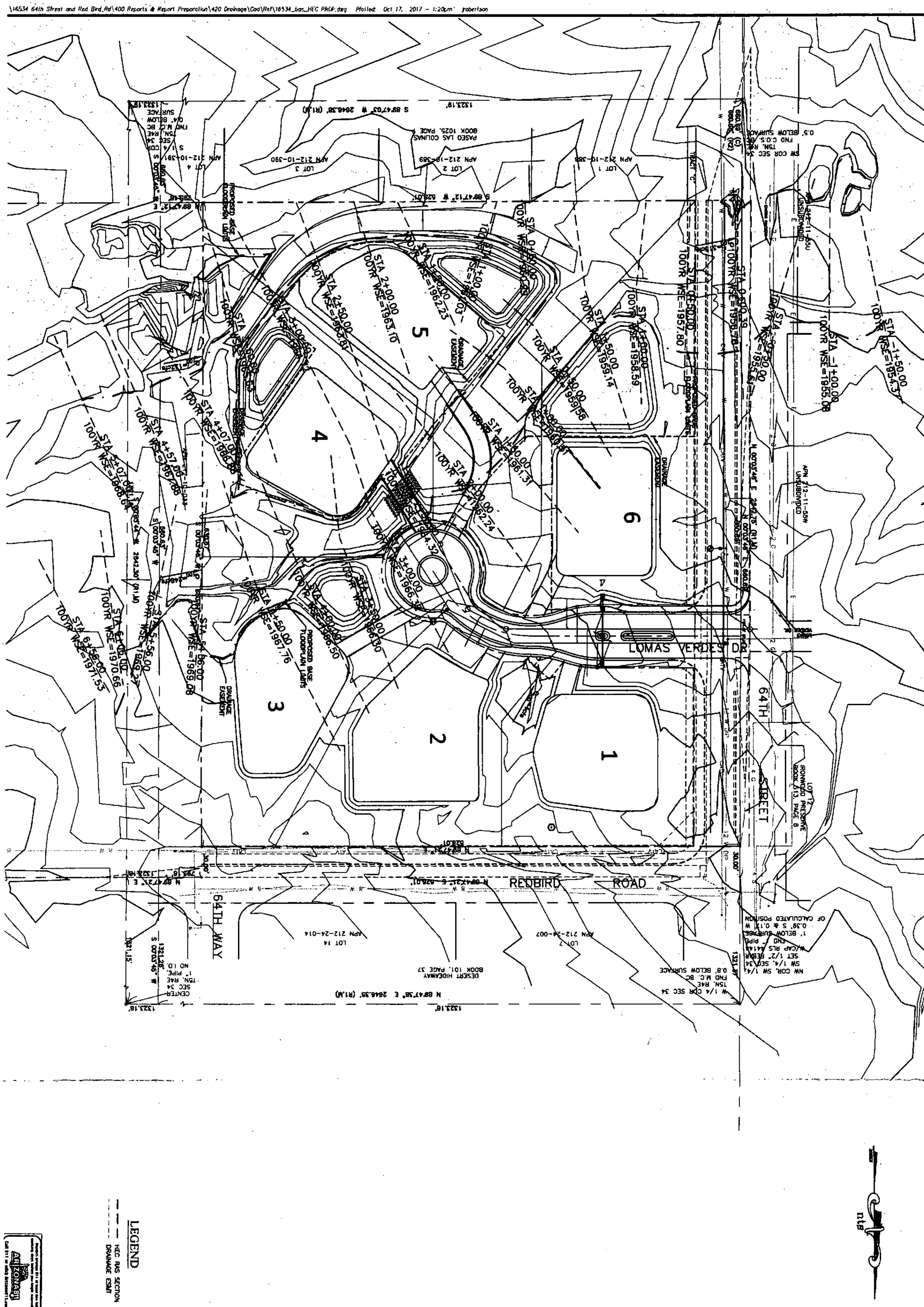
LOCATED IN SECTION 34, T 5 N, R 4 E, G&SRM, MARICOPA COUNTY, ARIZONA

EXISTING CONDITION FLOOD PLAIN MAP

Engineering and Environmental Consultants, Inc.
7740 N. 16th Street, Suite 135 | Phoenix, Arizona 85020
Tel 602.248.7702 | Fax 602.248.7851

DESIGN BY:	JMC
DRAWN BY:	JAR
CHK'D BY:	JMC
DATE:	OCTOBER 2017
EEC NO	16534
DRAWING NO	1 OF 1

\16534 64th Street and Red Bird Rd\400 Reports & Report Preparation\420 Drainage\Cad\Ref\16534_bas_HEC PROP.dwg Plotted: Oct 17, 2017 - 1:20pm jroberson



LEGEND

HEC RAS SECTION

DRAINAGE ESMT

ARIZONA 811

DESIGN BY:	JAG
DRAWN BY:	JAR
CHK'D BY:	JAG
DATE:	OCTOBER 2017
SCALE:	
EEC NO	16534
DRAWING NO	1 OF 1

JAMES M. GRIFFIN 31482 ARIZONA, USA

eec civil engineering • land development surveying • environmental services staking • flood control and drainage transportation

Engineering and Environmental Consultants, Inc.
7740 N. 16th Street, Suite 135 | Phoenix, Arizona 85020
Tel 602.248.7702 | Fax 602.248.7851

SHEET TITLE

PROJECT TITLE
PROPOSED CONDITION FLOOD PLAIN MAP

LOMAS VERDES ESTATES
6501 E. REDBIRD ROAD, SCOTTSDALE, AZ

LOCATED IN SECTION 34,
T 5 N, R 4 E, G&SRM, MARICOPA COUNTY, ARIZONA

REV.	DATE	DESCRIPTION	CK. BY

5-PP-16 903-PA-2016 OS: 49-43

FIGURE 9

Pinnacle Peak West Area Drainage Master Study

PCN 122.01.20 FCD 2011C024 WA#3

Executive Summary of the

Hydrology and Hydraulics Technical Support Data Notebook

Prepared For:

Flood Control District of Maricopa County

FINAL

Prepared By:

JE FULLER
HYDROLOGY & GEOMORPHOLOGY, INC.

December, 2014

Pinnacle Peak West ADMS Executive Summary

Introduction

The Pinnacle Peak West (PPW) Area Drainage Master Study (ADMS) will identify and evaluate flood hazards in the study area by implementing a work plan which includes data collection; review of previous planning and engineering studies; information gathering and sharing from/to project partners, stakeholders, and the public; hydrologic and hydraulic modeling; geomorphologic assessments; field surveys; landscape architecture; and environmental overview.

This executive summary only summarizes the methodology and findings of the hydrologic and hydraulic modeling of the watershed; refer to the Hydrology and Hydraulics Technical Support Data Notebook (TSDN) for more detailed information regarding the modeling and the items discussed in this summary.

The primary goal of the modeling component of the PPW ADMS is to update and characterize the flood hazard using current detailed topography, updated precipitation data, and two-dimensional modeling methodologies. Based on this updated understanding of the flooding hazard, this project may include formulation of flood hazard mitigation strategies to address the identified flooding hazards. The modeling results can also be used as input to the planning and design of drainage infrastructure and flood mitigation measures that are appropriate for the physical environment for both existing and future development.

Hydrologic analyses were performed for the 10-, 25-, and 100-year events. The results of the 10- and 25-year analyses are to be used for risk assessment purposes. The methods and results of the hazard and risk assessment are presented in the Task 12 and 13 – Hazard and Risk Memo provided under separate cover.

The results of the hydrologic and hydraulic analyses will be used to:

- More accurately characterize the location and extent of the existing flood hazards in the study area;
- Determine the adequacy of current and proposed drainage infrastructure;
- Plan and design future drainage infrastructure;
- Determine if there are practicable mitigation solutions that can reduce all or part of the flood hazard risk; and
- Compare to the effective FEMA floodplains and determine if additional floodplains should be delineated or if the existing floodplains should be redelineated.

Authority of Study

The Flood Control District of Maricopa County (District) has retained JE Fuller Hydrology and Geomorphology, Inc. (JEF) for completion of the PPW ADMS project. The District's contact and contract information is provided in Table 1 and the JEF contact information is provided below in Table 2.

Flood Control District
of Maricopa County

INTEROFFICE MEMORANDUM

Date: April 2, 2015

To: William D. Wiley, P.E., Chief Engineer and General Manager

From: Theresa Pinto

Subject: Pinnacle Peak West Area Drainage Master Study – Hydrology and Hydraulic Models and Results

The hydrology and hydraulic modeling for the Pinnacle Peak West Area Drainage Master Study (PPW ADMS) is complete and available to be used for this study area. The model results are based on the best available data at the time the model was developed, and standard modeling practices, assumptions, and engineering judgment. The models and results were thoroughly reviewed and approved by staff within the District's Engineering Division and Planning and Project Management Divisions.

The hydrology and hydraulic models were developed to identify flood hazards and risks in the PPW ADMS area. If the model results are used for other purposes, it is the user's responsibility to check the results for accuracy and applicability to their purpose. Furthermore, the results do not supersede or negate FEMA effective floodplains or any local, state, or federal floodplain or drainage regulatory requirements.

The results, models, and associated reports are available in the District's library. The report is titled "Pinnacle Peak West Area Drainage Master Study Hydrology and Hydraulics Technical Support Data Notebook December, 2014". The results and reports will also be available online in Spring/Summer 2015.

By signing below, you accept and approve the use of the PPW ADMS model and results as described herein.

Theresa Pinto Date: 4-2-15 Theresa Pinto, AICP, PMP Project Manager	Date: 4/6/15 William D. Wiley, P.E., Chief Engineer and General Manager
Catherine W. Regester Date: 4/3/15 Catherine Regester, P.E. Hydrology/Hydraulics Branch Manager	Kelli Sertich Date: 4/6/15 Kelli Sertich, AICP Floodplain Management & Services Division Manager
C. Scott Vogel Date: 4/6/15 Scott Vogel, P.E. Engineering Division Manager	Don Rerick Date: 4/3/15 Don Rerick, P.E. Planning and Project Management Division Manager

2801 West Durango Street Phoenix, Arizona 85009 Phone: 602-506-1501 Fax: 602-506-4601

Table 1. Flood Control District of Maricopa County Contact and Contract Information.

Authorizing Agency	*Flood Control District of Maricopa County (District)*
Contact Information	**Theresa Pinto**, AICP, CFM, PMP; Project Manager 2801 W Durango St., Phoenix, AZ 85009 602-506-8127 tmp@mail.maricopa.gov
Contract	Contract FCD 2011C024
Study Duration	Start Date: March 19, 2012; End Date: September 30, 2015

Table 2. Consulting Firm Information.

Primary Consulting Firm	*JE Fuller Hydrology & Geomorphology, Inc. (JEF)*
Contact Information	**Patricia K. Quinn**, PE, RLS, AVS; Project Manager 8400 S. Kyrene Rd, Ste. 201,Tempe, AZ 85284 480-222-5708 pat@jefuller.com

Location of Study

The PPW ADMS project study area is 97 square miles in size and is located in the northeastern portion of Maricopa County and encompasses land within the jurisdiction of the City of Phoenix, City of Scottsdale, Town of Cave Creek, Town of Carefree, and unincorporated Maricopa County. The primary stakeholders affected by the project are the City of Phoenix, City of Scottsdale, Maricopa County, and Arizona State Land Department (ASLD). The project is bound by approximately the Carefree Highway and Cave Creek Road to the north, the Pinnacle Peak South (PPS) ADMS study area and drainage divide to the east, the Central Arizona Project (CAP) Reach 11 Dikes to the south, and Cave Creek Road and the eastern Cave Creek floodplain limits to the west. The study area location and limits are shown in Figure 1.

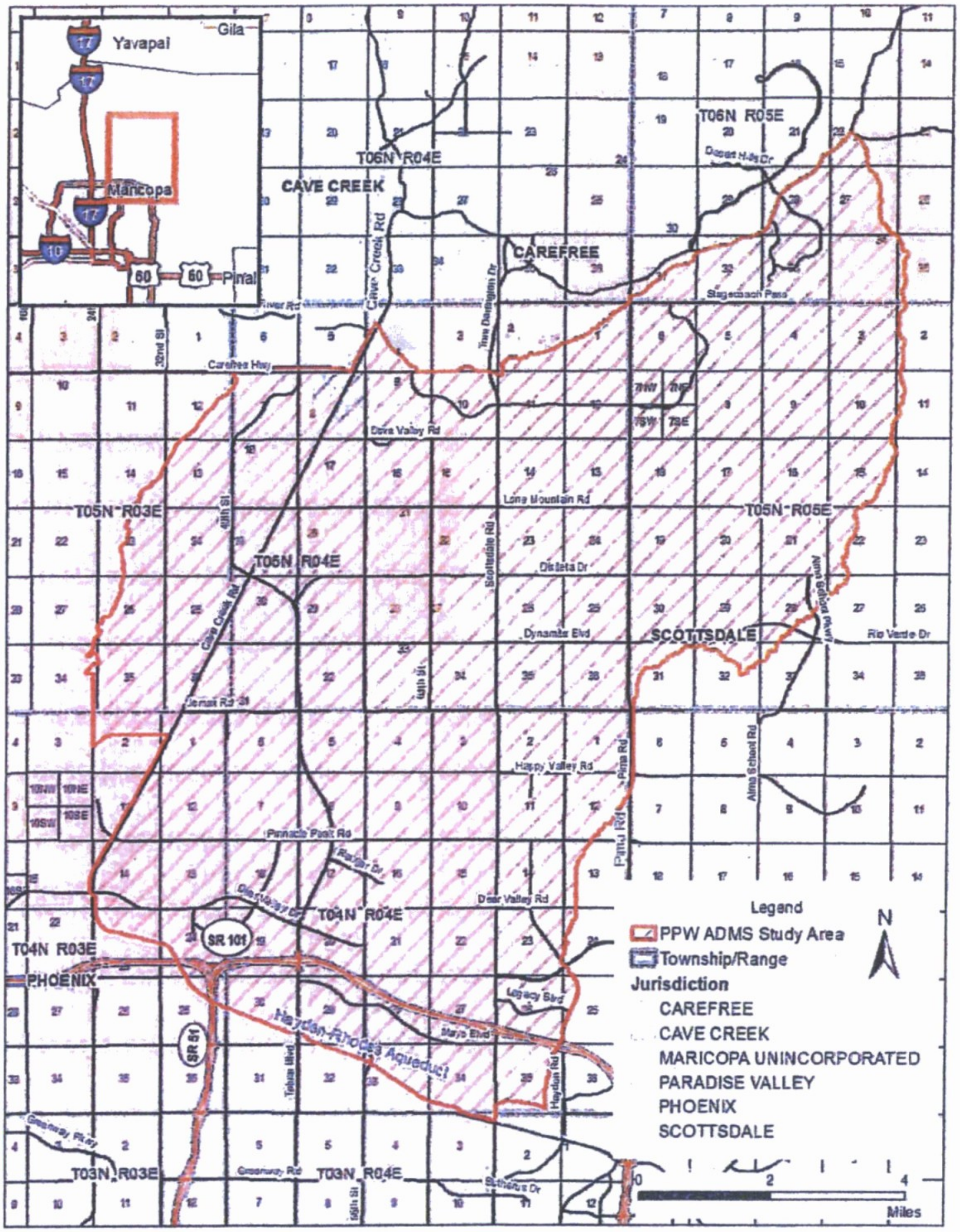


Figure 1. PPW ADMS Vicinity Map

Pinnacle Peak West ADMS Executive Summary

Methodology

Hydrologic and hydraulic (H&H) modeling for the PPW ADMS Project has been completed with the use of FLO-2D Professional Version FLO-2D (FLO-2D PRO), Build No. 13.07.05 and an executable dated 9-10-2013. The grid cell size used for all modeling is 20 feet by 20 feet. This 2-D modeling approach is highly suited for simulating the shallow, distributary flow prevalent within the watershed as flow travels from northeast to southwest through shallow braided channels in the undeveloped areas and through streets and around building structures in the developed areas. The models simulate rainfall/runoff for the 24-hour event with SCS Type II distribution using NOAA Atlas 14 rainfall data and Green and Ampt infiltration methodology. The FLO-2D model also incorporates building footprints using area reduction factors, hydraulically significant culverts, property walls, and channels within the model area. Significant storm drains within the model area are modeled as hydraulic structures. The models are developed using the existing land use conditions at the time of the TSDN documentation and were simulated under three scenarios related to property walls:

- *Without Property Walls* – There were no property walls were modeled.
- *With Property Walls and No Failure* – Property walls are modeled but walls were not failed regardless of ponding depth.
- *With Property Walls and With Failure* – Property walls are modeled but walls were failed when there was two feet of flow depth against them.

The PPW study area receives off-site flow from two sources, Unnamed Central Tributary to Cave Creek from the north (Carefree Drainage Master Plan) and the Pinnacle Peak South (PPS) ADMS from the east; see Figure 2 for locations. The PPW study area was subdivided into multiple model domains sub-areas due to the large watershed size and the grid cell size of 20 feet. Flow is passed from upstream sub-area model to the downstream sub-area(s) on a cell-to-cell basis along the overlapping sub-area boundaries. The nomenclature for the sub-area naming is based on prominent geographic features (e.g. Rawhide Wash) or master-planned communities (e.g. Desert Ridge) that lie within the vicinity of the sub-area domain. The prominent feature name and approximate model area for each sub-area model is listed in Table 3. See Figure 2 for the sub-area domain boundaries. Area R-11 is considered to be a unique condition as it overlaps with the Area DR and Area LR model domains, see the TSDN for a detailed discussion of Area R-11.

Table 3. PPW ADMS FLO-2D Model Sub-area Nomenclature Legend

Sub-Area ID	Prominent Feature Name	Area (mi^2)
Area LT	Legend Trail	12.0
Area UR	Upper Rawhide Wash	13.3
Area WR	Whisper Rock	15.1
Area TR	Tatum Ranch	15.7
Area LR	Lower Rawhide Wash	15.7
Area CB	Cave Buttes	9.9
Area DR	Desert Ridge	15.7
Area R-11*	Reach 11 Dikes	2.6**

*R-11 Model was developed to model the ponding of the Reach-11 Dikes upstream of the CAP canal.
**The area of the R-11 Model is included in the overlapping areas of LR and DR. The 2.6 square-mile area is not in addition to the total area.

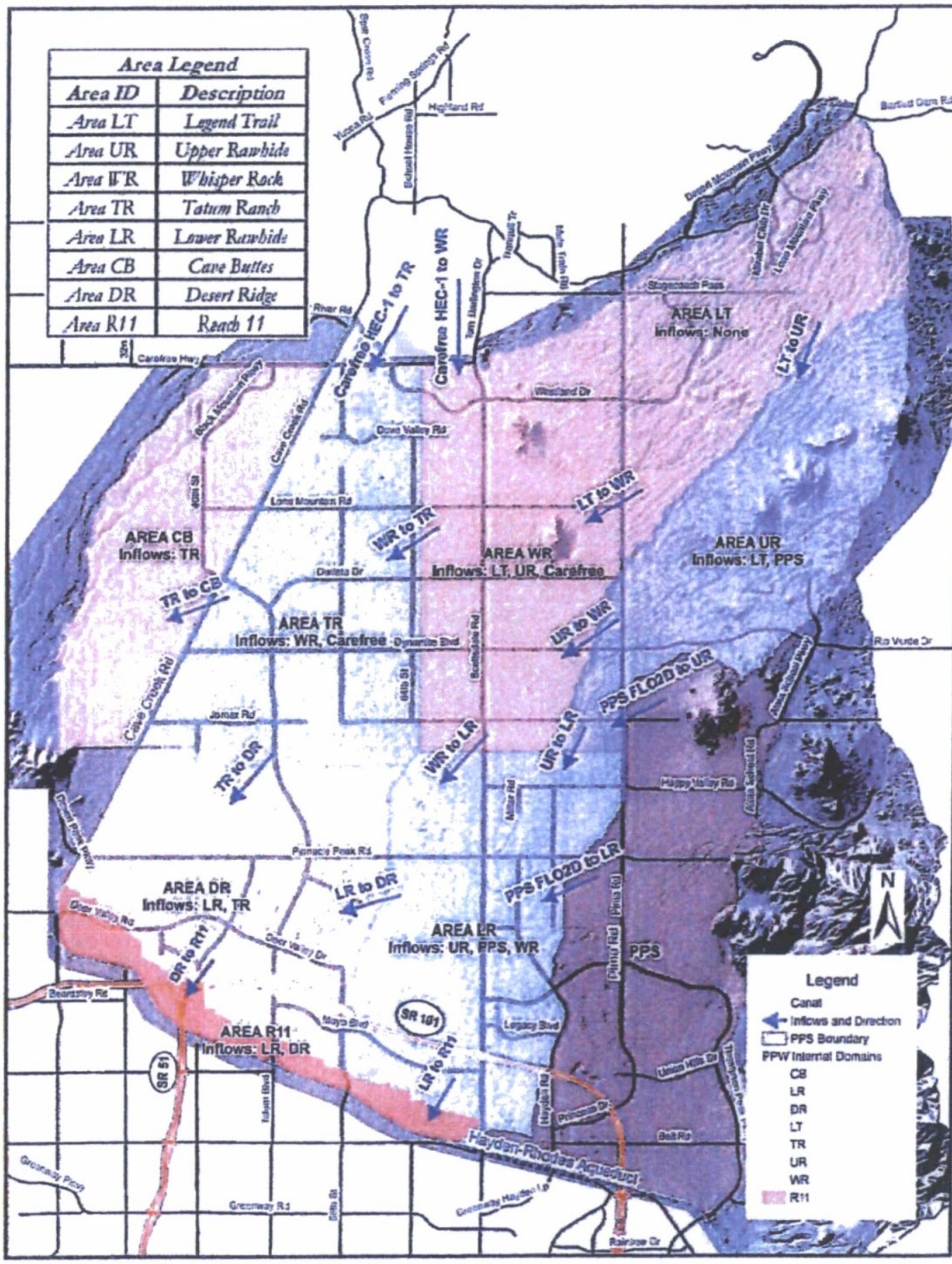


Figure 2. PPW ADMS FLO-2D Sub-Areas

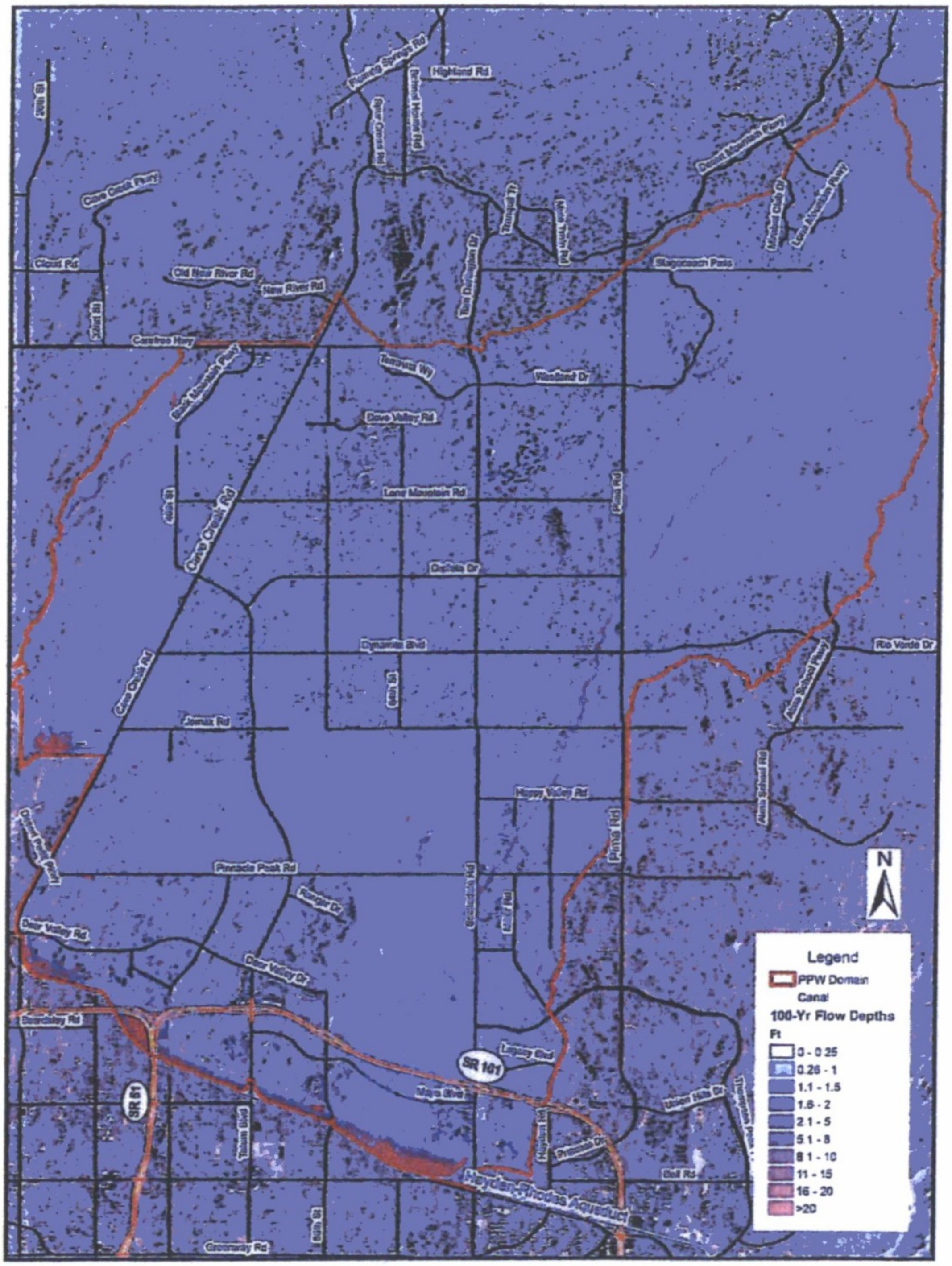


Figure 3. 100-Year Flow Depth Results

PANGOLIN STRUCTURAL

6868 N 7th Avenue
Suite 203
Phoenix, AZ 85013
p.602.888.0336
www.pangolinstr.com

STRUCTURAL CALULATIONS

LOMAS VERDES ESTATES
GATE ENTRY SITE WALLS

SCOTTSDALE, ARIZONA

17-050

MAY 12, 2017

Registered Professional Engineer (Structural)
CERTIFICATE NO.
45712
CRYSTAL
ENDERS
BLANTON
DATE SIGNED 5/12/2017
ARIZONA U.S.A.

Exp. 12/31/18

5-PP-2017
05/15/17

PANGOLIN STRUCTURAL

Sheet No. ______

Project No. 17-050

Project Name Lomas Verdes

Date May 2017

Subject GENERAL INFORMATION

Computed By CB

PROJECT DESCRIPTION

The project scope consists of 8 foot tall site walls and a 20' wide wood/steel gate. The following calculations determine wind loading and design of the masonry walls and foundations. The gate and steel sign are by others.

Pangolin Structural
6868 N 7th Ave, Ste 203
Phoenix, AZ 85013
602-888-0336

JOB TITLE Lomas Verdes

JOB NO. 17-050 SHEET NO.
CALCULATED BY cb DATE
CHECKED BY jl DATE

Wind Loads : ASCE 7- 10

Ultimate Wind Speed	110 mph
Nominal Wind Speed	85.2 mph
Risk Category	I
Exposure Category	C
Enclosure Classif.	Enclosed Building
Internal pressure	+/-0.18
Directionality (Kd)	0.85
Kh case 1	1.156
Kh case 2	1.156
Type of roof	Monoslope

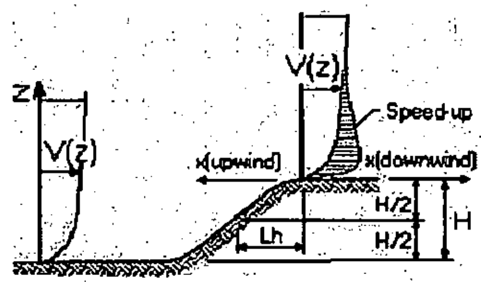
Topographic Factor (Kzt)

Topography	Flat
Hill Height (H)	80.0 ft
Half Hill Length (Lh)	100.0 ft
Actual H/Lh =	0.80
Use H/Lh =	0.50
Modified Lh =	160.0 ft
From top of crest: x =	50.0 ft
Bldg up/down wind?	downwind

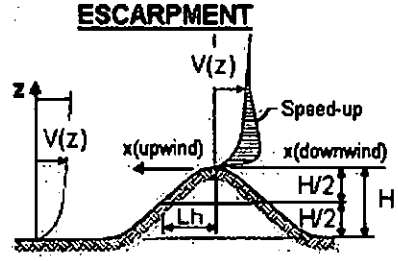
H/Lh= 0.50	K_1 = 0.000
x/Lh = 0.31	K_2 = 0.792
z/Lh = 0.41	K_3 = 1.000

At Mean Roof Ht:

$Kzt = (1+K_1K_2K_3)^{\wedge}2 = 1.00$



ESCARPMENT



2D RIDGE or 3D AXISYMMETRICAL HILL

Gust Effect Factor

h =	65.0 ft
B =	150.0 ft
/z (0.6h) =	39.0 ft

Flexible structure if natural frequency < 1 Hz (T > 1 second).
However, if building h/B < 4 then probably rigid structure (rule of thumb).
h/B = 0.43 Therefore, probably rigid structure

G = 0.85 Using rigid structure default

Rigid Structure

ē =	0.20
ℓ =	500 ft
z_{min} =	15 ft
c =	0.20
g_Q, g_v =	3.4
L_z =	517.0 ft
Q =	0.86
I_z =	0.19
G =	0.85 use G = 0.85

Flexible or Dynamically Sensitive Structure

Natural Frequency (η_1) =	0.0 Hz				
Damping ratio (β) =	0				
/b =	0.65				
/α =	0.15				
Vz =	107.6				
N_1 =	0.00				
R_n =	0.000				
R_h =	28.282	η =	0.000	h =	65.0 ft
R_B =	28.282	η =	0.000		
R_L =	28.282	η =	0.000		
g_R =	0.000				
R =	0.000				
G =	0.000				

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Enclosure Classification

Test for Enclosed Building: A building that does not qualify as open or partially enclosed.

Test for Open Building: All walls are at least 80% open.
Ao ≥ 0.8Ag

Test for Partially Enclosed Building:

	Input			Test	
Ao	100000.0	sf	Ao ≥ 1.1Aoi	YES	
Ag	0.0	sf	Ao > 4' or 0.01Ag	YES	
Aoi	0.0	sf	Aoi / Agi ≤ 0.20	NO	Building is NOT Partially Enclosed
Agi	0.0	sf			

ERROR: Ag must be greater than Ao

Conditions to qualify as Partially Enclosed Building. Must satisfy all of the following:
- Ao ≥ 1.1Aoi
- Ao > smaller of 4' or 0.01 Ag
- Aoi / Agi ≤ 0.20

Where:
Ao = the total area of openings in a wall that receives positive external pressure.
Ag = the gross area of that wall in which Ao is identified.
Aoi = the sum of the areas of openings in the building envelope (walls and roof) not including Ao.
Agi = the sum of the gross surface areas of the building envelope (walls and roof) not including Ag.

Reduction Factor for large volume partially enclosed buildings (Ri) :

If the partially enclosed building contains a single room that is unpartitioned , the internal pressure coefficient may be multiplied by the reduction factor Ri.

Total area of all wall & roof openings (Aog):		0 sf
Unpartitioned internal volume (Vi) :		0 cf
	Ri =	1.00

Altitude adjustment to constant 0.00256 (caution - see code) :

Altitude = 0 feet
Constant = 0.00256

Average Air Density = 0.0765 lbm/ft^3

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Wind Loads - Other Structures: ASCE 7- 10 Ultimate Wind Pressures

Wind Factor =	1.00		
Gust Effect Factor (G) =	0.85	Ultimate Wind Speed =	110 mph
Kzt =	1.00	Exposure =	C

A. Solid Freestanding Walls & Solid Signs (& open signs with less than 30% open)

		s/h =	1.00		Case A & B	
Dist to sign top (h)	8.0 ft	B/s =	6.88		C_f =	1.33
Height (s)	8.0 ft	Lr/s =	0.00		F = qz G Cf As =	**25.3 As**
Width (B)	55.0 ft	Kz =	0.849		As =	432.0 sf
Wall Return (Lr) =	0.0 ft	qz =	22.4 psf		F =	10926 lbs
Directionality (Kd)	0.85					
Percent of open area		Open reduction			CaseC	
to gross area	0.0%	factor =	1.00	Horiz dist from windward edge	Cf	F=qzGCfAs (psf)
		Case C reduction factors		0 to s	2.71	51.5 **As**
		Factor if s/h>0.8 =	0.80	s to 2s	1.79	34.0 **As**
		Wall return factor		2s to 3s	1.31	24.9 **As**
		for Cf at 0 to s =	1.00	3s to 10s	0.84	16.0 **As**

B. Open Signs & Lattice Frameworks (openings 30% or more of gross area)

Height to centroid of Af (z)	15.0 ft			Kz =	0.849
				Base pressure (qz) =	22.4 psf
Width (zero if round)	0.0 ft				
Diameter (zero if rect)	2.0 ft	D(qz)^.5 =	9.46	F = q_z G C_f A_f =	**20.9 Af**
Percent of open area		ε =	0.65	Solid Area: A_f =	10.0 sf
to gross area	35.0%	C_f =	1.1	F =	209 lbs
Directionality (Kd)	0.85				

PANGOLIN STRUCTURAL

Sheet No.
Project No. 17-050

Project Name Lomas Verdes
Date May 2017
Subject TYPICAL SITE WALL AND CENTER GATE PIER
Computed By CB

Typical Site Wall Design

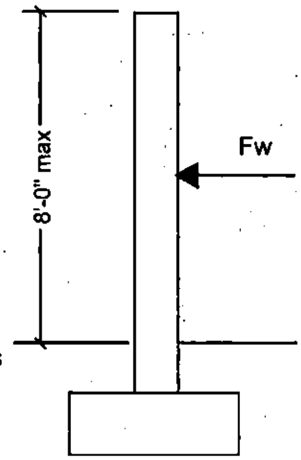
8'-0" tall
Max wind load = 51.5 psf/1.6 = 32 psf (allowable)

Point load to wall for design:
Fw = 32 psf * 8 ft = 256 pounds/ft of wall

SEE TEDDS OUTPUT

8" masonry wall with #5 at 16"o.c.
5'-0" wide footing x 16" thick with #6 at 12" o.c. top and bottom transverse

NOTE: Use same footing size for the steel signage since the wall height is similar



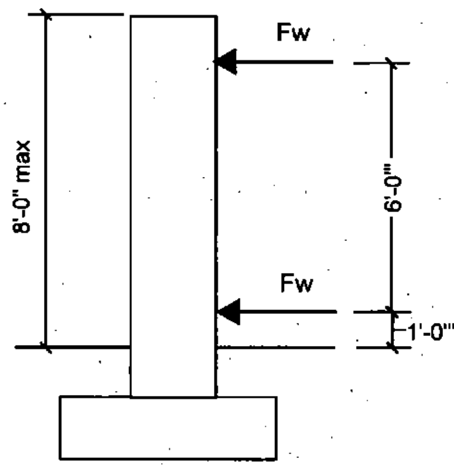
CENTER Masonry Pier Design

Consider 16" masonry pier (varies, 16" is least dimension)
8'-0" tall
Max wind load = 34 psf/1.6 = 21.3 psf (allowable)

Point load to wall for design:
Fw = 21.3 psf * 4 ft * 25 ft = 2.13k

For TEDDS design 2.13k / 5'-0" wide pier = 430 pounds/ft

SEE TEDDS OUTPUT



PANGOLIN STRUCTURAL
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Project	Lomas Verdes Estates	Sheet No.	1
		Project No.	17-050
		Date	5/12/2017
Subject	Typical site wall at entry	Computed By	CB

RETAINING WALL ANALYSIS

In accordance with International Building Code 2015

Tedds calculation version 2.8.01

NOTE: Utilized TEDDS retaining wall analysis to so that a cantilevered wall design could be performed but there is no retaining here - disregard soil pressures other than bearing capacity = 1500 psf

Retaining wall details

Stem type	Cantilever
Stem height	h_{stem} = **9.5** ft
Stem thickness	t_{stem} = **8** in
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **150** pcf
Toe length	l_{toe} = **2.17** ft
Heel length	l_{heel} = **2.17** ft
Base thickness	t_{base} = **16** in
Base density	γ_{base} = **150** pcf
Height of retained soil	h_{ret} = **0.083** ft
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **1.33** ft
Depth of excavation	d_{exc} = **0.667** ft

Retained soil properties

Soil type	Medium dense well graded sand
Moist density	γ_{mr} = **125** pcf
Saturated density	γ_{sr} = **137** pcf
Prescribed active lateral soil pressure	p_{Ar} = **30** psf/ft

Base soil properties

Soil type	Medium dense well graded sand
Soil density	γ_{b} = **115** pcf
Prescribed passive lateral soil pressure	p_{0b} = **60** psf/ft
Allowable bearing pressure	$P_{bearing}$ = **1000** psf

Loading details

Live surcharge load	$Surcharge_L$ = **200** psf
Horizontal line load at 6 ft	P_{L1} = **256** plf

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Project	Lomas Verdes Estates	Sheet No.	2
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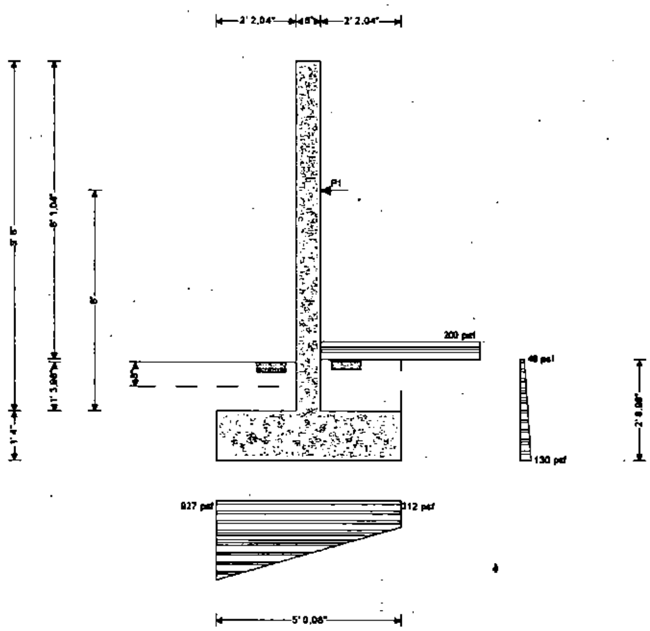
		Date	5/12/2017
Subject	Typical site wall at entry	Computed By	CB



General arrangement

Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem} + l_{heel} = \mathbf{5.007}$ ft
Moist soil height	$h_{moist} = h_{soil} = \mathbf{1.413}$ ft
Length of surcharge load	$l_{sur} = l_{heel} = \mathbf{2.17}$ ft
- Distance to vertical component	$x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{3.922}$ ft
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{2.747}$ ft
- Distance to horizontal component	$x_{sur\_h} = h_{eff} / 2 = \mathbf{1.373}$ ft
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem} = \mathbf{6.333}$ ft^2
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{2.503}$ ft
Area of wall base	$A_{base} = l_{base} \times t_{base} = \mathbf{6.676}$ ft^2
- Distance to vertical component	$x_{base} = l_{base} / 2 = \mathbf{2.503}$ ft
Area of moist soil	$A_{moist} = h_{moist} \times l_{heel} = \mathbf{3.067}$ ft^2
- Distance to vertical component	$x_{moist\_v} = l_{base} - (h_{moist} \times l_{heel}^2 / 2) / A_{moist} = \mathbf{3.922}$ ft
- Distance to horizontal component	$x_{moist\_h} = h_{eff} / 3 = \mathbf{0.916}$ ft
Area of base soil	$A_{pass} = d_{cover} \times l_{toe} = \mathbf{2.886}$ ft^2
- Distance to vertical component	$x_{pass\_v} = l_{base} - (d_{cover} \times l_{toe} \times (l_{base} - l_{toe} / 2)) / A_{pass} = \mathbf{1.085}$ ft
- Distance to horizontal component	$x_{pass\_h} = (d_{cover} + h_{base}) / 3 = \mathbf{0.888}$ ft
Area of excavated base soil	$A_{exc} = h_{pass} \times l_{toe} = \mathbf{1.439}$ ft^2
- Distance to vertical component	$x_{exc\_v} = l_{base} - (h_{pass} \times l_{toe} \times (l_{base} - l_{toe} / 2)) / A_{exc} = \mathbf{1.085}$ ft
- Distance to horizontal component	$x_{exc\_h} = (h_{pass} + h_{base}) / 3 = \mathbf{0.666}$ ft

Soil coefficients

Coefficient of friction to back of wall — K_{fr} = **0.325**

Coefficient of friction to front of wall — K_{fb} = **0.325**

Coefficient of friction beneath base — K_{fbb} = **0.325**

From IBC 2015 cl.1807.2.3 Safety factor

Load combination 1 — 1.0 × Dead + 1.0 × Live + 1.0 × Lateral earth

Sliding check

Vertical forces on wall

Wall stem — $F_{stem} = A_{stem} \times \gamma_{stem}$ = **950** plf

Wall base — $F_{base} = A_{base} \times \gamma_{base}$ = **1001** plf

Moist retained soil — $F_{moist\_v} = A_{moist} \times \gamma_{mr}$ = **383** plf

Base soil — $F_{exc\_v} = A_{exc} \times \gamma_b$ = **166** plf

Total — $F_{total\_v} = F_{stem} + F_{base} + F_{moist\_v} + F_{exc\_v}$ = **2500** plf

Horizontal forces on wall

Surcharge load — $F_{sur\_h} = p_{Ar} / \gamma_{mr} \times Surcharge_L \times h_{eff}$ = **132** plf

Line loads — $F_{P\_h} = P_{L1}$ = **256** plf

Moist retained soil — $F_{moist\_h} = p_{Ar} \times h_{eff}^2 / 2$ = **113** plf

Total — $F_{total\_h} = F_{moist\_h} + F_{sur\_h} + F_{P\_h}$ = **501** plf

Check stability against sliding

Base soil resistance — $F_{exc\_h} = p_{0b} \times (h_{pass} + h_{base})^2 / 2$ = **120** plf

Base friction — $F_{friction} = F_{total\_v} \times K_{fbb}$ = **813** plf

Resistance to sliding — $F_{rest} = F_{exc\_h} + F_{friction}$ = **932** plf

Factor of safety — $FoS_{sl} = F_{rest} / F_{total\_h}$ = **1.861** > 1.5

PASS - Factor of safety against sliding is adequate

Overturning check

Vertical forces on wall

Wall stem — $F_{stem} = A_{stem} \times \gamma_{stem}$ = **950** plf

Wall base — $F_{base} = A_{base} \times \gamma_{base}$ = **1001** plf

Moist retained soil — $F_{moist\_v} = A_{moist} \times \gamma_{mr}$ = **383** plf

Base soil — $F_{exc\_v} = A_{exc} \times \gamma_b$ = **166** plf

Total — $F_{total\_v} = F_{stem} + F_{base} + F_{moist\_v} + F_{exc\_v}$ = **2500** plf

Horizontal forces on wall

Surcharge load — $F_{sur\_h} = p_{Ar} / \gamma_{mr} \times Surcharge_L \times h_{eff}$ = **132** plf

Line loads — $F_{P\_h} = P_{L1}$ = **256** plf

Moist retained soil — $F_{moist\_h} = p_{Ar} \times h_{eff}^2 / 2$ = **113** plf

Base soil — $F_{exc\_h} = -p_{0b} \times (h_{pass} + h_{base})^2 / 2$ = **-120** plf

Total — $F_{total\_h} = F_{moist\_h} + F_{exc\_h} + F_{sur\_h} + F_{P\_h}$ = **381** plf

Overturning moments on wall

Surcharge load — $M_{sur\_OT} = F_{sur\_h} \times x_{sur\_h}$ = **181** lb_ft/ft

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Line loads — $M_{P\_OT} = abs(P_{L1}) \times (p_1 + t_{base}) = \mathbf{1877}$ lb_ft/ft

Moist retained soil — $M_{moist\_OT} = F_{moist\_h} \times x_{moist\_h} = \mathbf{104}$ lb_ft/ft

Total — $M_{total\_OT} = M_{moist\_OT} + M_{sur\_OT} + M_{P\_OT} = \mathbf{2162}$ lb_ft/ft

Restoring moments on wall

Wall stem — $M_{stem\_R} = F_{stem} \times x_{stem} = \mathbf{2378}$ lb_ft/ft

Wall base — $M_{base\_R} = F_{base} \times x_{base} = \mathbf{2507}$ lb_ft/ft

Moist retained soil — $M_{moist\_R} = F_{moist\_v} \times x_{moist\_v} = \mathbf{1503}$ lb_ft/ft

Base soil — $M_{exc\_R} = F_{exc\_v} \times x_{exc\_v} - F_{exc\_h} \times x_{exc\_h} = \mathbf{259}$ lb_ft/ft

Total — $M_{total\_R} = M_{stem\_R} + M_{base\_R} + M_{moist\_R} + M_{exc\_R} = \mathbf{6647}$ lb_ft/ft

Check stability against overturning

Factor of safety — $FoS_{ot} = M_{total\_R} / M_{total\_OT} = \mathbf{3.075} > 1.5$

PASS - Factor of safety against overturning is adequate

Bearing pressure check

Vertical forces on wall

Wall stem — $F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{950}$ plf

Wall base — $F_{base} = A_{base} \times \gamma_{base} = \mathbf{1001}$ plf

Surcharge load — $F_{sur\_v} = Surcharge_L \times l_{heel} = \mathbf{434}$ plf

Moist retained soil — $F_{moist\_v} = A_{moist} \times \gamma_{mr} = \mathbf{383}$ plf

Base soil — $F_{pass\_v} = A_{pass} \times \gamma_b = \mathbf{332}$ plf

Total — $F_{total\_v} = F_{stem} + F_{base} + F_{moist\_v} + F_{pass\_v} + F_{sur\_v} = \mathbf{3101}$ plf

Horizontal forces on wall

Surcharge load — $F_{sur\_h} = p_{Ar} / \gamma_{mr} \times Surcharge_L \times h_{eff} = \mathbf{132}$ plf

Line loads — $F_{P\_h} = P_{L1} = \mathbf{256}$ plf

Moist retained soil — $F_{moist\_h} = p_{Ar} \times h_{eff}^2 / 2 = \mathbf{113}$ plf

Base soil — $F_{pass\_h} = -p_{0b} \times (d_{cover} + h_{base})^2 / 2 = \mathbf{-213}$ plf

Total — $F_{total\_h} = max(F_{moist\_h} + F_{pass\_h} + F_{sur\_h} + F_{P\_h} - F_{total\_v} \times K_{fbb}, 0 \text{ plf}) = \mathbf{0}$ plf

Moments on wall

Wall stem — $M_{stem} = F_{stem} \times x_{stem} = \mathbf{2378}$ lb_ft/ft

Wall base — $M_{base} = F_{base} \times x_{base} = \mathbf{2507}$ lb_ft/ft

Surcharge load — $M_{sur} = F_{sur\_v} \times x_{sur\_v} - F_{sur\_h} \times x_{sur\_h} = \mathbf{1521}$ lb_ft/ft

Line loads — $M_P = -(P_{L1} \times (p_1 + t_{base})) = \mathbf{-1877}$ lb_ft/ft

Moist retained soil — $M_{moist} = F_{moist\_v} \times x_{moist\_v} - F_{moist\_h} \times x_{moist\_h} = \mathbf{1400}$ lb_ft/ft

Base soil — $M_{pass} = F_{pass\_v} \times x_{pass\_v} - F_{pass\_h} \times x_{pass\_h} = \mathbf{549}$ lb_ft/ft

Total — $M_{total} = M_{stem} + M_{base} + M_{moist} + M_{pass} + M_{sur} + M_P = \mathbf{6477}$ lb_ft/ft

Check bearing pressure

Distance to reaction — $\bar{x} = M_{total} / F_{total\_v} = \mathbf{2.089}$ ft

Eccentricity of reaction — $e = \bar{x} - l_{base} / 2 = \mathbf{-0.414}$ ft

Loaded length of base — $l_{load} = l_{base} = \mathbf{5.007}$ ft

Bearing pressure at toe — $q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) = \mathbf{927}$ psf

Bearing pressure at heel — $q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base})$ = **312** psf

Factor of safety — $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel})$ = **1.079**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

RETAINING WALL DESIGN

In accordance with ACI 318-11 and MSJC-11 using the strength design method

Tedds calculation version 2.8.01

Concrete details

Compressive strength of concrete	f'_c = **2500** psi
Concrete type	Normal weight

Reinforcement details

Yield strength of reinforcement	f_y = **60000** psi
Modulus of elasticity or reinforcement	E_s = **29000000** psi

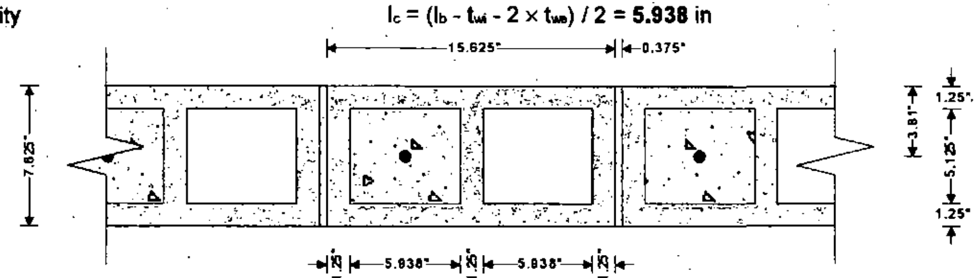
Cover to reinforcement

Top face of base	c_{bt} = **2** in
Bottom face of base	c_{bb} = **3** in

Masonry details

8" CMU in running bond, fully bedded with PCL class M mortar, grouted at 16" centers

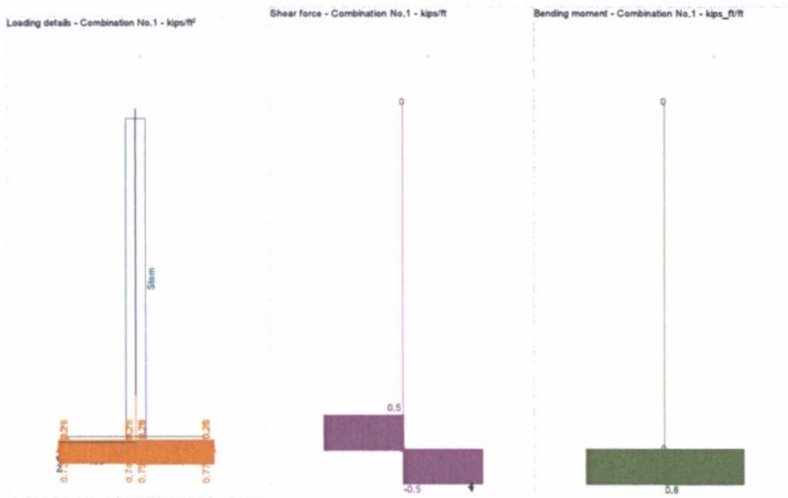
Compressive strength of unit	f_{cu} = **2800** psi
Net compressive strength - Table 2	f'_m = **2000** psi
Net modulus of elasticity - cl.1.8.2.2.1	$E_m = 900 \times f'_m$ = **1800000** psi
Modulus of rupture - Table 3.1.8.2	f_r = **113** psi
Thickness of unit	t_b = **7.625** in
Length of unit	l_b = **15.625** in
Height of unit	h_b = **7.625** in
Thickness of joint	t_j = **0.375** in
Face shell thickness	t_{wf} = **1.25** in
End shell thickness	t_{we} = **1.25** in
Internal web thickness	t_{wi} = **1.25** in
Depth of cavity	$t_c = t_b - 2 \times t_{wf}$ = **5.125** in
Length of cavity	$l_c = (l_b - t_{wi} - 2 \times t_{we}) / 2$ = **5.938** in

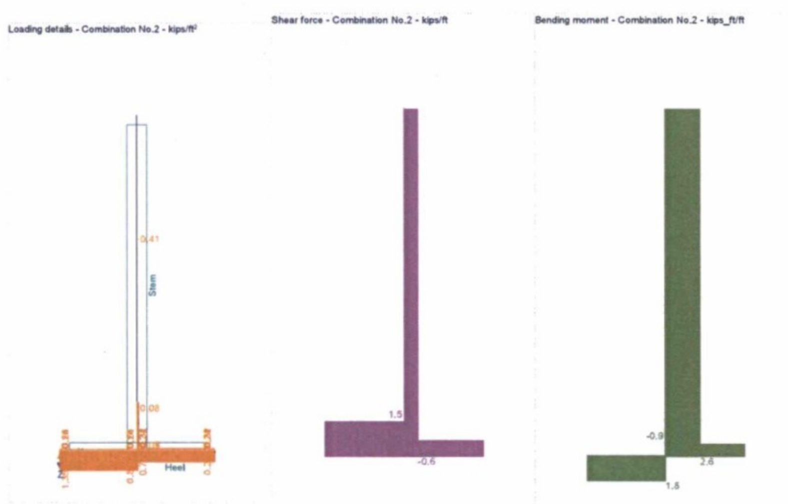


From IBC 2015 cl.1605.2.1 Basic load combinations

Load combination no.1 — 1.4 × Dead

Load combination no.2	$1.2 \times$ Dead + $1.6 \times$ Live + $1.6 \times$ Lateral earth
Load combination no.3	$1.2 \times$ Dead + $1.0 \times$ Earthquake + $1.0 \times$ Live + $1.6 \times$ Lateral earth
Load combination no.4	$0.9 \times$ Dead + $1.0 \times$ Earthquake + $1.6 \times$ Lateral earth





Check stem design at base of stem

Depth of section: $t = \mathbf{8}$ in

Masonry section properties

Gross cross-sectional area	$A = t_b - l_c \times t_c / (l_b + t_j) = \mathbf{68.7}$ in²/ft
Gross moment of inertia	$I = t_b^3 / 12 - l_c \times t_c^3 / (12 \times (l_b + t_j)) = \mathbf{393.4}$ in⁴/ft
Gross section modulus	$S = 2 \times I / t_b = \mathbf{103.2}$ in³/ft
Gross radius of gyration	$r = \sqrt{(I / A)} = \mathbf{2.4}$ in

Reinforced masonry - Section 3.3

Design bending moment combination 2 — M = **30683 lb_in/ft**

Axial load — $P = 1.2 \times \gamma_{stem} \times h_{stem} \times A$ = **816 lb/ft**

Effective height — $h = 2 \times h_{stem}$ = **19 ft**

Slenderness ratio — h / r = **95.267**

Nominal axial strength - exp.3-18 — $P_n = 0.8 \times (0.8 \times (A - A_{sr.prov}) \times f'_m) \times [1 - (h / (140 \times r))^2]$ = **47044 lb/ft**

Strength reduction factor - cl.3.1.4 — ϕ = **0.9**

Design axial strength — $\phi P_n = \phi \times P_n$ = **42339 lb/ft**

$P / \phi P_n$ = **0.019**

PASS - Nominal axial strength exceeds axial load

Reinforcement provided — No.5 bars @ 16" c/c

Area of reinforcement provided — $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr})$ = **0.23 in²/ft**

Depth of reinforcement — d = **3.81 in**

Maximum usable compressive strain of masonry - cl.3.3.2

ε_{mu} = **0.0025**

Tensile strain in reinforcement at balance point — $\varepsilon_s = f_y / E_s$ = **0.002069**

Tension reinforcement strain factor — α_s = **1.5**

Maximum area of reinforcement — $A_{sr.max} = 0.64 \times f'_m \times d \times [\varepsilon_{mu} / (\varepsilon_{mu} + \alpha_s \times \varepsilon_s)] / f_y$ = **0.435 in²/ft**

PASS - Area of stem reinforcement provided is less than maximum allowable

Distance from fiber of maximum compressive strain to neutral axis

$c = d \times \varepsilon_{mu} / (\varepsilon_{mu} + \varepsilon_s)$ = **2.085 in**

Tensile force at balance point — $T_b = A_{sr.prov} \times f_y$ = **13806 lb/ft**

β_1 = **0.8**

Compressive force at balance point — $C_b = 0.8 \times f'_m \times \beta_1 \times (l_b + t_j - l_c) / (l_b + t_j) \times c$ = **20138 lb/ft**

Design axial force at balance point — $P_b = \phi \times (C_b - T_b)$ = **5699 lb/ft**

Design moment at balance point — $M_b = \phi \times (T_b \times (d - t_b / 2) + C_b \times (t_b / 2 - \beta_1 \times c / 2))$ = **53955 lb_in/ft**

Strength interaction diagram

c / d	c (in)	C (lb/ft)	T (lb/ft)	f_s (psi)	M (lb_in/ft)	P (lb/ft)
0.01	0.038	368	13806	60000	1227	-12094
0.1	0.381	3680	13806	60000	12093	-9113
0.2	0.762	7361	13806	60000	23207	-5800
0.3	1.143	11041	13806	60000	33311	-2488
0.4	1.524	14722	13806	60000	42406	824
0.5	1.905	18402	13806	60000	50492	4137
0.547	2.085	20138	13806	60000	53955	5699
0.6	2.286	22083	11121	48333	57573	9865

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Project	Lomas Verdes Estates	Sheet No.	8
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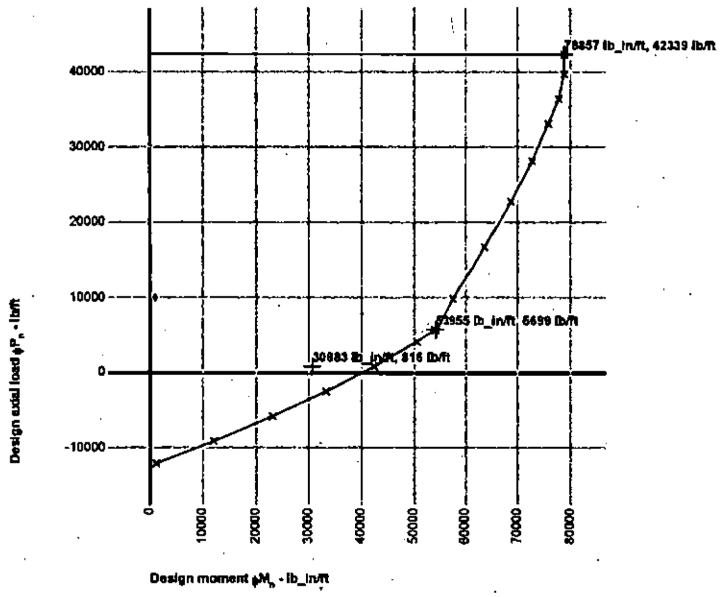
Subject	Typical site wall at entry	Computed By	CB

0.7	2.667	25763	7149	31071	63648	16752
0.8	3.048	29444	4171	18125	68711	22746
0.9	3.429	33124	1854	8056	72763	28144
1	3.81	36805	0	0	75805	33124
1.1	4.191	40485	0	0	77832	36437
1.2	4.572	44166	0	0	78850	39749
1.3	4.953	47846	0	0	78857	42339



From strength interaction diagram...

Maximum moment M_{max} = **78858** lb_in/ft

Limiting moment under applied axial load M_{limit} = **42382** lb_in/ft

M / M_{limit} = **0.724**

PASS - Design flexural strength exceeds factored bending moment

Design shear force V = **566** lb/ft

Nominal shear strength - cl.3.3.4.1.2 $V_n = \min((4 - 1.75 \times \min(M / (V \times t_b), 1)) \times A \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times P, 4 \times A \times \sqrt{(f'_m \times 1 \text{ psi})})$ = **7114** lb/ft

Strength reduction factor - cl.3.1.4 ϕ_v = **0.8**

Design shear strength $\phi V_n = \phi_v \times V_n$ = **5692** lb/ft

$V / \phi V_n$ = **0.099**

PASS - Design shear strength exceeds applied shear force

Check base design at toe

Depth of section — $h = \mathbf{16}$ in

Rectangular section in flexure - Chapter 10

Design bending moment combination 2 — $M = \mathbf{1791}$ lb_ft/ft

Depth of tension reinforcement — $d = h - c_{bb} - \phi_{bb} / 2 = \mathbf{12.625}$ in

Compression reinforcement provided — No.6 bars @ 12" c/c

Area of compression reinforcement provided — $A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt}) = \mathbf{0.442}$ in^2/ft

Tension reinforcement provided — No.6 bars @ 12" c/c

Area of tension reinforcement provided — $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) = \mathbf{0.442}$ in^2/ft

Maximum reinforcement spacing - cl.10.5.4 — $s_{max} = \min(18 \text{ in}, 3 \times h) = \mathbf{18}$ in

PASS - Reinforcement is adequately spaced

Depth of compression block — $a = A_{bb.prov} \times f_y / (0.85 \times f'_c) = \mathbf{1.039}$ in

Neutral axis factor - cl.10.2.7.3 — $\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4 \text{ ksi}) / 1 \text{ ksi}, 0.65), 0.85) = \mathbf{0.85}$

Depth to neutral axis — $c = a / \beta_1 = \mathbf{1.223}$ in

Strain in reinforcement — $\varepsilon_t = 0.003 \times (d - c) / c = \mathbf{0.02797}$

Section is in the tension controlled zone

Strength reduction factor — $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) = \mathbf{0.9}$

Nominal flexural strength — $M_n = A_{bb.prov} \times f_y \times (d - a / 2) = \mathbf{26740}$ lb_ft/ft

Design flexural strength — $\phi M_n = \phi_f \times M_n = \mathbf{24066}$ lb_ft/ft

$M / \phi M_n = \mathbf{0.074}$

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis — $A_{bb.des} = \mathbf{0.032}$ in^2/ft

Minimum area of reinforcement - cl.7.12.2.1 — $A_{bb.min} = 0.0018 \times h = \mathbf{0.346}$ in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Chapter 11

Design shear force — $V = \mathbf{1471}$ lb/ft

Concrete modification factor - cl.8.6.1 — $\lambda = \mathbf{1}$

Nominal concrete shear strength - eqn.11-3 — $V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times d = \mathbf{15150}$ lb/ft

Strength reduction factor — $\phi_s = \mathbf{0.75}$

Design concrete shear strength - cl.11.4.6.1 — $\phi V_c = \phi_s \times V_c = \mathbf{11363}$ lb/ft

$V / \phi V_c = \mathbf{0.129}$

PASS - No shear reinforcement is required

Check base design at heel

Depth of section — $h = \mathbf{16}$ in

Rectangular section in flexure - Chapter 10

Design bending moment combination 2 — $M = \mathbf{943}$ lb_ft/ft

Depth of tension reinforcement — $d = h - c_{bt} - \phi_{bt} / 2 = \mathbf{13.625}$ in

Compression reinforcement provided — No.6 bars @ 12" c/c

Area of compression reinforcement provided $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **0.442** in²/ft

Tension reinforcement provided No.6 bars @ 12" c/c

Area of tension reinforcement provided $A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt})$ = **0.442** in²/ft

Maximum reinforcement spacing - cl.10.5.4 s_{max} = min(18 in, 3 × h) = **18** in

PASS - Reinforcement is adequately spaced

Depth of compression block $a = A_{bt.prov} \times f_y / (0.85 \times f'_c)$ = **1.039** in

Neutral axis factor - cl.10.2.7.3 β_1 = min(max(0.85 - 0.05 × (f'c - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**

Depth to neutral axis $c = a / \beta_1$ = **1.223** in

Strain in reinforcement $\varepsilon_t = 0.003 \times (d - c) / c$ = **0.030424**

Section is in the tension controlled zone

Strength reduction factor ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.9**

Nominal flexural strength $M_n = A_{bt.prov} \times f_y \times (d - a / 2)$ = **28949** lb_ft/ft

Design flexural strength $\phi M_n = \phi_f \times M_n$ = **26054** lb_ft/ft

$M / \phi M_n$ = **0.036**

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis $A_{bt.des}$ = **0.015** in²/ft

Minimum area of reinforcement - cl.7.12.2.1 $A_{bt.min} = 0.0018 \times h$ = **0.346** in²/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Chapter 11

Design shear force V = **690** lb/ft

Concrete modification factor - cl.8.6.1 λ = **1**

Nominal concrete shear strength - eqn.11-3 $V_c = 2 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times d$ = **16350** lb/ft

Strength reduction factor ϕ_s = **0.75**

Design concrete shear strength - cl.11.4.6.1 $\phi V_c = \phi_s \times V_c$ = **12263** lb/ft

$V / \phi V_c$ = **0.056**

PASS - No shear reinforcement is required

Transverse reinforcement parallel to base

Minimum area of reinforcement - cl.7.12.2.1 $A_{bx.req} = 0.0018 \times t_{base}$ = **0.346** in²/ft

Transverse reinforcement provided No.4 bars @ 8" c/c each face

Area of transverse reinforcement provided $A_{bx.prov} = 2 \times \pi \times \phi_{bx}^2 / (4 \times s_{bx})$ = **0.589** in²/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required

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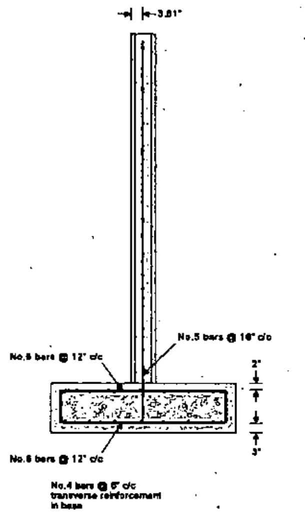
		Sheet No.	11
		Project No.	17-050
Project	Lomas Verdes Estates	Date	5/12/2017
Subject	Typical site wall at entry	Computed By	CB



Reinforcement details

RETAINING WALL ANALYSIS

In accordance with International Building Code 2015

Tedds calculation version 2.8.01

Retaining wall details

Stem type	Cantilever
Stem height	h_{stem} = **9.5** ft
Stem thickness	t_{stem} = **40** in
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **150** pcf
Toe length	l_{toe} = **2.33** ft
Heel length	l_{heel} = **2.33** ft
Base thickness	t_{base} = **24** in
Base density	γ_{base} = **150** pcf
Height of retained soil	h_{ret} = **0.083** ft
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **1.33** ft
Depth of excavation	d_{exc} = **0.667** ft

Retained soil properties

Soil type	Medium dense well graded sand
Moist density	γ_{mr} = **125** pcf
Saturated density	γ_{sr} = **137** pcf
Prescribed active lateral soil pressure	p_{Ar} = **30** psf/ft

Base soil properties

Soil type	Medium dense well graded sand
Soil density	γ_b = **115** pcf
Prescribed passive lateral soil pressure	p_{0b} = **60** psf/ft
Allowable bearing pressure	$P_{bearing}$ = **1900** psf

Loading details

Horizontal line load at 8.5 ft	P_{L1} = **430** plf
Horizontal line load at 2.5 ft	P_{L2} = **430** plf
Vertical line load at 4 ft	P_{D3} = **600** plf

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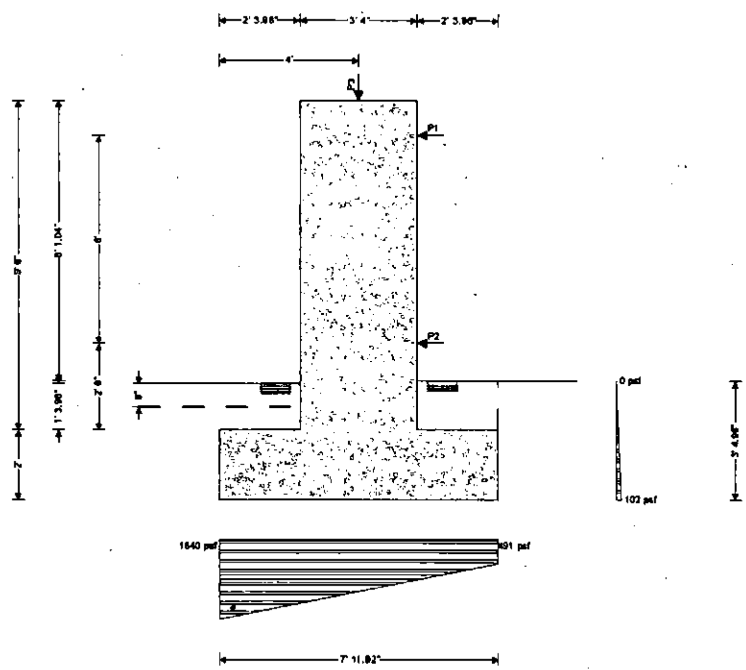
		Sheet No.	2
		Project No.	17-050
Project	Lomas Verdes Estates	Date	5/12/2017
Subject	Center Masonry Pier	Computed By	CB



General arrangement

Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem} + l_{heel} = \mathbf{7.993\ ft}$
Moist soil height	$h_{moist} = h_{soil} = \mathbf{1.413\ ft}$
Retained surface length	$l_{sur} = l_{heel} = \mathbf{2.33\ ft}$
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{3.413\ ft}$
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem} = \mathbf{31.667\ ft^2}$
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{3.997\ ft}$
Area of wall base	$A_{base} = l_{base} \times t_{base} = \mathbf{15.987\ ft^2}$
- Distance to vertical component	$x_{base} = l_{base} / 2 = \mathbf{3.997\ ft}$
Area of moist soil	$A_{moist} = h_{moist} \times l_{heel} = \mathbf{3.293\ ft^2}$
- Distance to vertical component	$x_{moist\_v} = l_{base} - (h_{moist} \times l_{heel}^2 / 2) / A_{moist} = \mathbf{6.828\ ft}$
- Distance to horizontal component	$x_{moist\_h} = h_{eff} / 3 = \mathbf{1.138\ ft}$
Area of base soil	$A_{pass} = d_{cover} \times l_{toe} = \mathbf{3.099\ ft^2}$
- Distance to vertical component	$x_{pass\_v} = l_{base} - (d_{cover} \times l_{toe} \times (l_{base} - l_{toe} / 2)) / A_{pass} = \mathbf{1.165\ ft}$
- Distance to horizontal component	$x_{pass\_h} = (d_{cover} + h_{base}) / 3 = \mathbf{1.11\ ft}$
Area of excavated base soil	$A_{exc} = h_{pass} \times l_{toe} = \mathbf{1.546\ ft^2}$
- Distance to vertical component	$x_{exc\_v} = l_{base} - (h_{pass} \times l_{toe} \times (l_{base} - l_{toe} / 2)) / A_{exc} = \mathbf{1.165\ ft}$
- Distance to horizontal component	$x_{exc\_h} = (h_{pass} + h_{base}) / 3 = \mathbf{0.888\ ft}$

Soil coefficients

Coefficient of friction to back of wall	K_{fr} = **0.325**
Coefficient of friction to front of wall	K_{fb} = **0.325**
Coefficient of friction beneath base	K_{fbb} = **0.325**

From IBC 2015 cl.1807.2.3 Safety factor

Load combination 1 — 1.0 × Dead + 1.0 × Live + 1.0 × Lateral earth

Sliding check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem}$ = **4750** plf
Wall base	$F_{base} = A_{base} \times \gamma_{base}$ = **2398** plf
Line loads	$F_{P\_v} = P_{D3}$ = **600** plf
Moist retained soil	$F_{moist\_v} = A_{moist} \times \gamma_{mr}$ = **412** plf
Base soil	$F_{exc\_v} = A_{exc} \times \gamma_b$ = **178** plf
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{moist\_v} + F_{exc\_v} + F_{P\_v}$ = **8337** plf

Horizontal forces on wall

Line loads	$F_{P\_h} = P_{L1} + P_{L2}$ = **860** plf
Moist retained soil	$F_{moist\_h} = p_{Ar} \times h_{eff}^2 / 2$ = **175** plf
Total	$F_{total\_h} = F_{moist\_h} + F_{P\_h}$ = **1035** plf

Check stability against sliding

Base soil resistance	$F_{exc\_h} = p_{0b} \times (h_{pass} + h_{base})^2 / 2$ = **213** plf
Base friction	$F_{friction} = F_{total\_v} \times K_{fbb}$ = **2710** plf
Resistance to sliding	$F_{rest} = F_{exc\_h} + F_{friction}$ = **2922** plf
Factor of safety	$FoS_{sl} = F_{rest} / F_{total\_h}$ = **2.824** > 1.5

PASS - Factor of safety against sliding is adequate

Overturning check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem}$ = **4750** plf
Wall base	$F_{base} = A_{base} \times \gamma_{base}$ = **2398** plf
Line loads	$F_{P\_v} = P_{D3}$ = **600** plf
Moist retained soil	$F_{moist\_v} = A_{moist} \times \gamma_{mr}$ = **412** plf
Base soil	$F_{exc\_v} = A_{exc} \times \gamma_b$ = **178** plf
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{moist\_v} + F_{exc\_v} + F_{P\_v}$ = **8337** plf

Horizontal forces on wall

Line loads	$F_{P\_h} = P_{L1} + P_{L2}$ = **860** plf
Moist retained soil	$F_{moist\_h} = p_{Ar} \times h_{eff}^2 / 2$ = **175** plf
Base soil	$F_{exc\_h} = -p_{0b} \times (h_{pass} + h_{base})^2 / 2$ = **-213** plf
Total	$F_{total\_h} = F_{moist\_h} + F_{exc\_h} + F_{P\_h}$ = **822** plf

Overturning moments on wall

Line loads	$M_{P\_OT} = abs(P_{L1}) \times (p_1 + t_{base}) + abs(P_{L2}) \times (p_2 + t_{base})$ = **6450** lb_ft/ft

Moist retained soil	$M_{moist\_OT} = F_{moist\_h} \times x_{moist\_h} =$ **199** lb_ft/ft
Total	$M_{total\_OT} = M_{moist\_OT} + M_{P\_OT} =$ **6649** lb_ft/ft

Restoring moments on wall

Wall stem	$M_{stem\_R} = F_{stem} \times x_{stem} =$ **18984** lb_ft/ft
Wall base	$M_{base\_R} = F_{base} \times x_{base} =$ **9584** lb_ft/ft
Line loads	$M_{P\_R} = abs(P_{D3}) \times p_3 =$ **2400** lb_ft/ft
Moist retained soil	$M_{moist\_R} = F_{moist\_v} \times x_{moist\_v} =$ **2811** lb_ft/ft
Base soil	$M_{exc\_R} = F_{exc\_v} \times x_{exc\_v} - F_{exc\_h} \times x_{exc\_h} =$ **396** lb_ft/ft
Total	$M_{total\_R} = M_{stem\_R} + M_{base\_R} + M_{moist\_R} + M_{exc\_R} + M_{P\_R} =$ **34175** lb_ft/ft

Check stability against overturning

Factor of safety	$FoS_{ot} = M_{total\_R} / M_{total\_OT} =$ **5.14** > 1.5

PASS - Factor of safety against overturning is adequate

Bearing pressure check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem} =$ **4750** plf
Wall base	$F_{base} = A_{base} \times \gamma_{base} =$ **2398** plf
Line loads	$F_{P\_v} = P_{D3} =$ **600** plf
Moist retained soil	$F_{moist\_v} = A_{moist} \times \gamma_{mr} =$ **412** plf
Base soil	$F_{pass\_v} = A_{pass} \times \gamma_b =$ **356** plf
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{moist\_v} + F_{pass\_v} + F_{P\_v} =$ **8516** plf

Horizontal forces on wall

Line loads	$F_{P\_h} = P_{L1} + P_{L2} =$ **860** plf
Moist retained soil	$F_{moist\_h} = p_{Ar} \times h_{eff}^2 / 2 =$ **175** plf
Base soil	$F_{pass\_h} = -p_{0b} \times (d_{cover} + h_{base})^2 / 2 =$ **-333** plf
Total	$F_{total\_h} = \max(F_{moist\_h} + F_{pass\_h} + F_{P\_h} - F_{total\_v} \times K_{fbb}, 0\text{ plf}) =$ **0** plf

Moments on wall

Wall stem	$M_{stem} = F_{stem} \times x_{stem} =$ **18984** lb_ft/ft
Wall base	$M_{base} = F_{base} \times x_{base} =$ **9584** lb_ft/ft
Line loads	$M_P = P_{D3} \times p_3 - (P_{L1} \times (p_1 + t_{base}) + (P_{L2}) \times (p_2 + t_{base})) =$ **-4050** lb_ft/ft
Moist retained soil	$M_{moist} = F_{moist\_v} \times x_{moist\_v} - F_{moist\_h} \times x_{moist\_h} =$ **2612** lb_ft/ft
Base soil	$M_{pass} = F_{pass\_v} \times x_{pass\_v} - F_{pass\_h} \times x_{pass\_h} =$ **784** lb_ft/ft
Total	$M_{total} = M_{stem} + M_{base} + M_{moist} + M_{pass} + M_P =$ **27915** lb_ft/ft

Check bearing pressure

Distance to reaction	$\bar{x} = M_{total} / F_{total\_v} =$ **3.278** ft
Eccentricity of reaction	$e = \bar{x} - l_{base} / 2 =$ **-0.719** ft
Loaded length of base	$l_{load} = l_{base} =$ **7.993** ft
Bearing pressure at toe	$q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) =$ **1640** psf
Bearing pressure at heel	$q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base}) =$ **491** psf
Factor of safety	$FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) =$ **1.158**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

RETAINING WALL DESIGN

In accordance with ACI 318-11 and MSJC-11 using the strength design method

Tedds calculation version 2.8.01

Concrete details

Compressive strength of concrete f'_c = **2500** psi

Concrete type Normal weight

Reinforcement details

Yield strength of reinforcement f_y = **60000** psi

Modulus of elasticity or reinforcement E_s = **29000000** psi

Cover to reinforcement

Top face of base c_{bt} = **2** in

Bottom face of base c_{bb} = **3** in

Masonry details

12" CMU in running bond, fully bedded with PCL class M mortar, fully grouted

Compressive strength of unit f_{cu} = **2800** psi

Net compressive strength - Table 2 f'_m = **2000** psi

Net modulus of elasticity - cl.1.8.2.2.1 $E_m = 900 \times f'_m$ = **1800000** psi

Modulus of rupture - Table 3.1.8.2 f_r = **163** psi

Thickness of unit t_b = **11.625** in

Length of unit l_b = **15.625** in

Height of unit h_b = **7.625** in

Thickness of joint t_j = **0.375** in

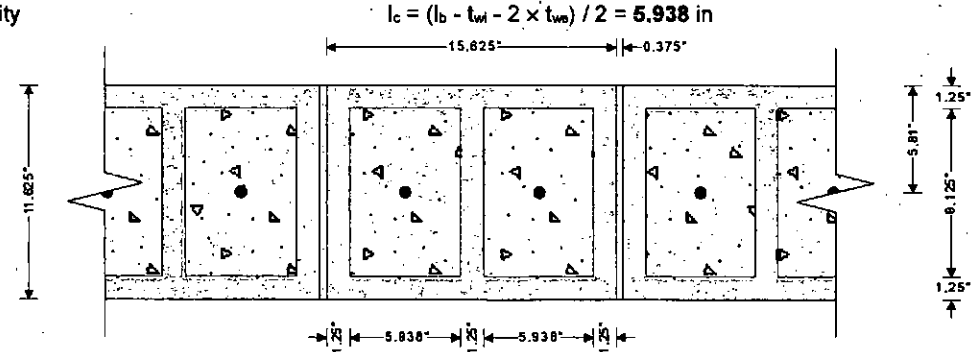
Face shell thickness t_{wf} = **1.25** in

End shell thickness t_{we} = **1.25** in

Internal web thickness t_{wi} = **1.25** in

Depth of cavity $t_c = t_b - 2 \times t_{wf}$ = **9.125** in

Length of cavity $l_c = (l_b - t_{wi} - 2 \times t_{we}) / 2$ = **5.938** in



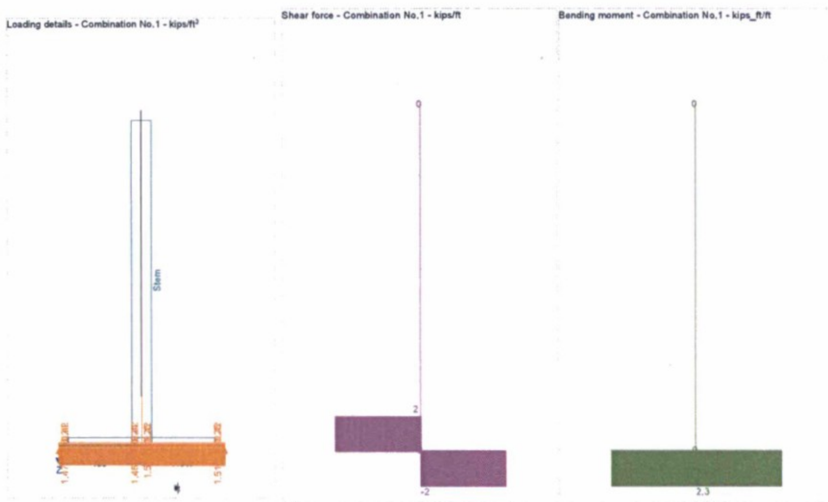
From IBC 2015 cl.1605.2.1 Basic load combinations

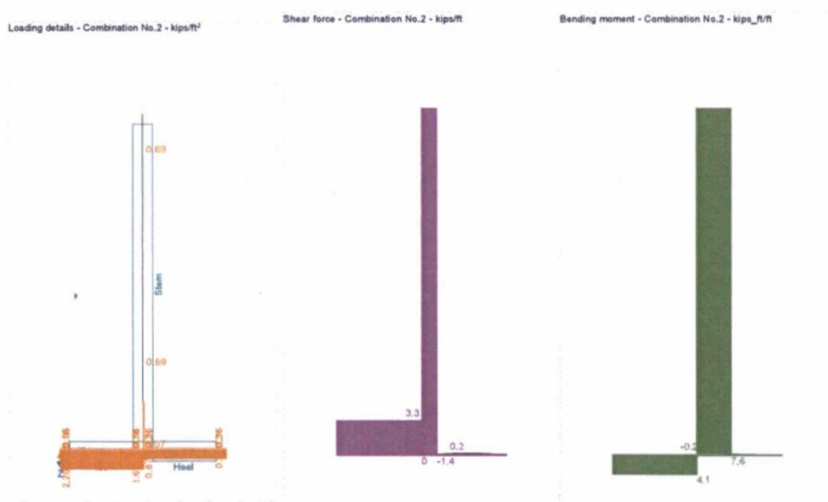
Load combination no.1 1.4 × Dead

Load combination no.2 — $1.2 \times$ Dead + $1.6 \times$ Live + $1.6 \times$ Lateral earth

Load combination no.3 — $1.2 \times$ Dead + $1.0 \times$ Earthquake + $1.0 \times$ Live + $1.6 \times$ Lateral earth

Load combination no.4 — $0.9 \times$ Dead + $1.0 \times$ Earthquake + $1.6 \times$ Lateral earth





Check stem design at base of stem

Depth of section — t = **40** in

Masonry section properties

Gross cross-sectional area — $A = t_b$ = **139.5** in²/ft

Gross moment of inertia — $I = t_b^3 / 12$ = **1571** in⁴/ft

Gross section modulus — $S = 2 \times I / t_b$ = **270.3** in³/ft

Gross radius of gyration — $r = \sqrt{(I / A)}$ = **3.4** in

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Reinforced masonry - Section 3.3

Design bending moment combination 2 — M = **91087** lb_in/ft

Axial load — $P = 1.2 \times (\gamma_{stem} \times h_{stem} \times A + P_{D3})$ = **2377** lb/ft

Effective height — $h = 2 \times h_{stem}$ = **19 ft**

Slenderness ratio — h / r = **67.941**

Nominal axial strength - exp.3-18 — $P_n = 0.8 \times (0.8 \times (A - A_{sr.prov}) \times f'_m) \times [1 - (h / (140 \times r))^2]$ = **136057** lb/ft

Strength reduction factor - cl.3.1.4 — ϕ = **0.9**

Design axial strength — $\phi P_n = \phi \times P_n$ = **122451** lb/ft

$P / \phi P_n$ = **0.019**

PASS - Nominal axial strength exceeds axial load

Reinforcement provided — No.5 bars @ 8" c/c

Area of reinforcement provided — $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr})$ = **0.46** in^2/ft

Depth of reinforcement — d = **5.81** in

Maximum usable compressive strain of masonry - cl.3.3.2

ε_{mu} = **0.0025**

Tensile strain in reinforcement at balance point — $\varepsilon_s = f_y / E_s$ = **0.002069**

Tension reinforcement strain factor — α_s = **1.5**

Maximum area of reinforcement — $A_{sr.max} = 0.64 \times f'_m \times d \times [\varepsilon_{mu} / (\varepsilon_{mu} + \alpha_s \times \varepsilon_s)] / f_y$ = **0.664** in^2/ft

PASS - Area of stem reinforcement provided is less than maximum allowable

Distance from fiber of maximum compressive strain to neutral axis

$c = d \times \varepsilon_{mu} / (\varepsilon_{mu} + \varepsilon_s)$ = **3.179** in

Tensile force at balance point — $T_b = A_{sr.prov} \times f_y$ = **27612** lb/ft

β_1 = **0.8**

Compressive force at balance point — $C_b = 0.8 \times f'_m \times \beta_1 \times c$ = **48830** lb/ft

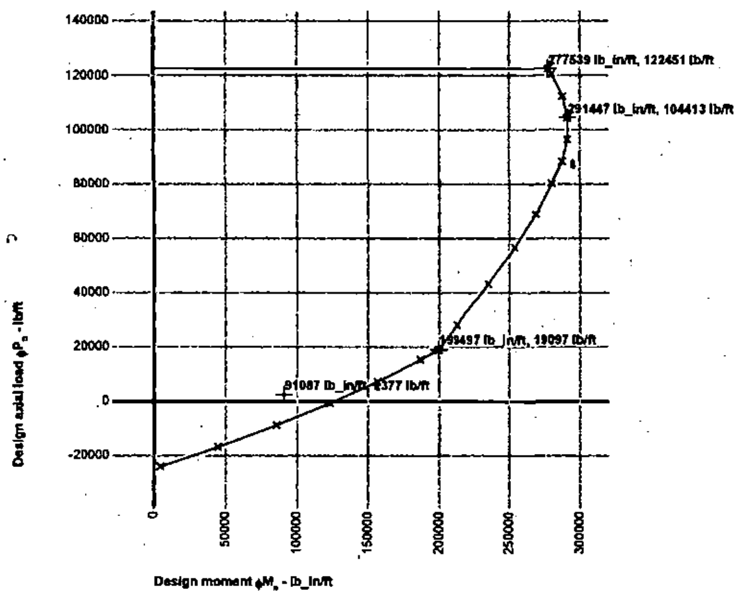
Design axial force at balance point — $P_b = \phi \times (C_b - T_b)$ = **19097** lb/ft

Design moment at balance point — $M_b = \phi \times (T_b \times (d - t_b / 2) + C_b \times (t_b / 2 - \beta_1 \times c / 2))$ = **199497** lb_in/ft

Strength interaction diagram

c / d	c (in)	C (lb/ft)	T (lb/ft)	f_s (psi)	M (lb_in/ft)	P (lb/ft)
0.01	0.058	892	27612	60000	4588	-24047
0.1	0.581	8924	27612	60000	44756	-16819
0.2	1.162	17848	27612	60000	85841	-8787
0.3	1.743	26772	27612	60000	123192	-755
0.4	2.324	35697	27612	60000	156811	7276
0.5	2.905	44621	27612	60000	186696	15308
0.547	3.179	48830	27612	60000	199497	19097
0.6	3.486	53545	22243	48333	212860	28172

0.7	4.067	62469	14299	31071	235297	43353
0.8	4.648	71393	8341	18125	253996	56747
0.9	5.229	80317	3707	8056	268960	68949
1	5.81	89242	0	0	280187	80317
1.1	6.391	98166	0	0	287674	88349
1.2	6.972	107090	0	0	291427	96381
1.3	7.553	116014	0	0	291447	104413
1.4	8.134	124938	0	0	287734	112444
1.5	8.715	133862	0	0	280288	120476
1.6	9.296	142787	0	0	277539	122451



From strength interaction diagram...

Maximum moment M_{max} = **291447** lb_in/ft

Limiting moment under applied axial load M_{limit} = **136301** lb_in/ft

M / M_{limit} = **0.668**

PASS - Design flexural strength exceeds factored bending moment

Design shear force V = **1424** lb/ft

Nominal shear strength - cl.3.3.4.1.2 $V_n = \min((4 - 1.75 \times \min(M / (V \times t_b), 1)) \times A \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times P, 4 \times A \times \sqrt{(f'_m \times 1 \text{ psi})})$ = **14631** lb/ft

Strength reduction factor - cl.3.1.4 ϕ_v = **0.8**

Design shear strength	$\phi V_n = \phi_v \times V_n =$ **11705** lb/ft
	$V / \phi V_n =$ **0.122**

PASS - Design shear strength exceeds applied shear force

Check base design at toe

Depth of section	h = **24** in

Rectangular section in flexure - Chapter 10

Design bending moment combination 2	M = **4141** lb_ft/ft
Depth of tension reinforcement	$d = h - c_{bb} - \phi_{bb} / 2 =$ **20.625** in
Compression reinforcement provided	No.6 bars @ 10" c/c
Area of compression reinforcement provided	$A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt}) =$ **0.53** in^2/ft
Tension reinforcement provided	No.6 bars @ 10" c/c
Area of tension reinforcement provided	$A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) =$ **0.53** in^2/ft
Maximum reinforcement spacing - cl.10.5.4	$s_{max} = \min(18\text{ in}, 3 \times h) =$ **18** in

PASS - Reinforcement is adequately spaced

Depth of compression block	$a = A_{bb.prov} \times f_y / (0.85 \times f'_c) =$ **1.247** in
Neutral axis factor - cl.10.2.7.3	$\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4\text{ ksi}) / 1\text{ ksi}, 0.65), 0.85) =$ **0.85**
Depth to neutral axis	$c = a / \beta_1 =$ **1.468** in
Strain in reinforcement	$\varepsilon_t = 0.003 \times (d - c) / c =$ **0.039163**

Section is in the tension controlled zone

Strength reduction factor	$\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) =$ **0.9**
Nominal flexural strength	$M_n = A_{bb.prov} \times f_y \times (d - a / 2) =$ **53018** lb_ft/ft
Design flexural strength	$\phi M_n = \phi_f \times M_n =$ **47716** lb_ft/ft
	$M / \phi M_n =$ **0.087**

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis	$A_{bb.des} =$ **0.045** in^2/ft
Minimum area of reinforcement - cl.7.12.2.1	$A_{bb.min} = 0.0018 \times h =$ **0.518** in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Chapter 11

Design shear force	V = **3332** lb/ft
Concrete modification factor - cl.8.6.1	$\lambda =$ **1**
Nominal concrete shear strength - eqn.11-3	$V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times d =$ **24750** lb/ft
Strength reduction factor	$\phi_s =$ **0.75**
Design concrete shear strength - cl.11.4.6.1	$\phi V_c = \phi_s \times V_c =$ **18563** lb/ft
	$V / \phi V_c =$ **0.180**

PASS - No shear reinforcement is required

Check base design at heel

Depth of section	h = **24** in

Rectangular section in flexure - Chapter 10

Design bending moment combination 2 — $M = \mathbf{229}$ lb_ft/ft

Depth of tension reinforcement — $d = h - c_{bt} - \phi_{bt} / 2 = \mathbf{21.625}$ in

Compression reinforcement provided — No.6 bars @ 10" c/c

Area of compression reinforcement provided — $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) = \mathbf{0.53}$ in²/ft

Tension reinforcement provided — No.6 bars @ 10" c/c

Area of tension reinforcement provided — $A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt}) = \mathbf{0.53}$ in²/ft

Maximum reinforcement spacing - cl.10.5.4 — $s_{max} = \min(18 \text{ in}, 3 \times h) = \mathbf{18}$ in

PASS - Reinforcement is adequately spaced

Depth of compression block — $a = A_{bt.prov} \times f_y / (0.85 \times f'_c) = \mathbf{1.247}$ in

Neutral axis factor - cl.10.2.7.3 — $\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4 \text{ ksi}) / 1 \text{ ksi}, 0.65), 0.85) = \mathbf{0.85}$

Depth to neutral axis — $c = a / \beta_1 = \mathbf{1.468}$ in

Strain in reinforcement — $\varepsilon_t = 0.003 \times (d - c) / c = \mathbf{0.041207}$

Section is in the tension controlled zone

Strength reduction factor — $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) = \mathbf{0.9}$

Nominal flexural strength — $M_n = A_{bt.prov} \times f_y \times (d - a / 2) = \mathbf{55669}$ lb_ft/ft

Design flexural strength — $\phi M_n = \phi_f \times M_n = \mathbf{50102}$ lb_ft/ft

$M / \phi M_n = \mathbf{0.005}$

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis — $A_{bt.des} = \mathbf{0.002}$ in²/ft

Minimum area of reinforcement - cl.7.12.2.1 — $A_{bt.min} = 0.0018 \times h = \mathbf{0.518}$ in²/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Chapter 11

Design shear force — $V = \mathbf{1955}$ lb/ft

Concrete modification factor - cl.8.6.1 — $\lambda = \mathbf{1}$

Nominal concrete shear strength - eqn.11-3 — $V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times d = \mathbf{25950}$ lb/ft

Strength reduction factor — $\phi_s = \mathbf{0.75}$

Design concrete shear strength - cl.11.4.6.1 — $\phi V_c = \phi_s \times V_c = \mathbf{19463}$ lb/ft

$V / \phi V_c = \mathbf{0.100}$

PASS - No shear reinforcement is required

Transverse reinforcement parallel to base

Minimum area of reinforcement - cl.7.12.2.1 — $A_{bx.req} = 0.0018 \times t_{base} = \mathbf{0.518}$ in²/ft

Transverse reinforcement provided — No.4 bars @ 8" c/c each face

Area of transverse reinforcement provided — $A_{bx.prov} = 2 \times \pi \times \phi_{bx}^2 / (4 \times s_{bx}) = \mathbf{0.589}$ in²/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required

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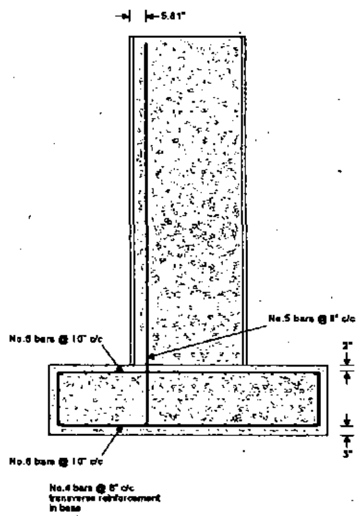


Reinforcement details

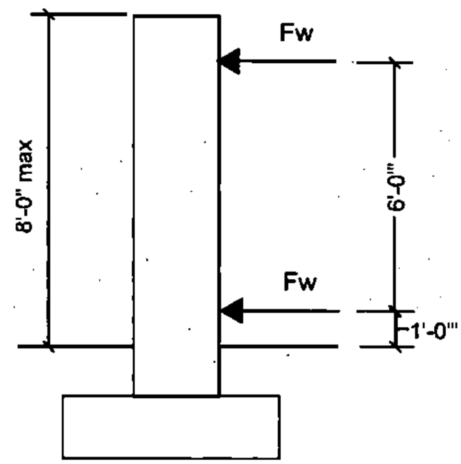
EDGE Masonry Pier Design

Consider 16" masonry pier (varies, 16" is least dimension)
8'-0" tall
Max wind load = 34 psf/1.6 = 21.3 psf (allowable)

Point load to wall for design:
Fw = 21.3 psf * 4 ft * 12 ft = 1.0k

For TEDDS design 1.0k / 3'-0" wide (avg) pier = 333 pounds/ft

SEE TEDDS OUTPUT



RETAINING WALL ANALYSIS

In accordance with International Building Code 2015

Tedds calculation version 2.8.01

Retaining wall details

Stem type	Cantilever
Stem height	h_{stem} = **9.5** ft
Stem thickness	t_{stem} = **24** in
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **150** pcf
Toe length	l_{toe} = **3** ft
Heel length	l_{heel} = **3** ft
Base thickness	t_{base} = **24** in
Base density	γ_{base} = **150** pcf
Height of retained soil	h_{ret} = **0.083 ft**
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **1.33 ft**
Depth of excavation	d_{exc} = **0.667 ft**

Retained soil properties

Soil type	Medium dense well graded sand
Moist density	γ_{mr} = **125** pcf
Saturated density	γ_{sr} = **137** pcf
Prescribed active lateral soil pressure	p_{Ar} = **30 psf/ft**

Base soil properties

Soil type	Medium dense well graded sand
Soil density	γ_b = **115** pcf
Prescribed passive lateral soil pressure	p_{Ob} = **60 psf/ft**
Allowable bearing pressure	$P_{bearing}$ = **1500 psf**

Loading details

Horizontal line load at 8.5 ft	P_{L1} = **333** plf
Horizontal line load at 2.5 ft	P_{L2} = **333** plf
Vertical line load at 4 ft	P_{D3} = **1200** plf

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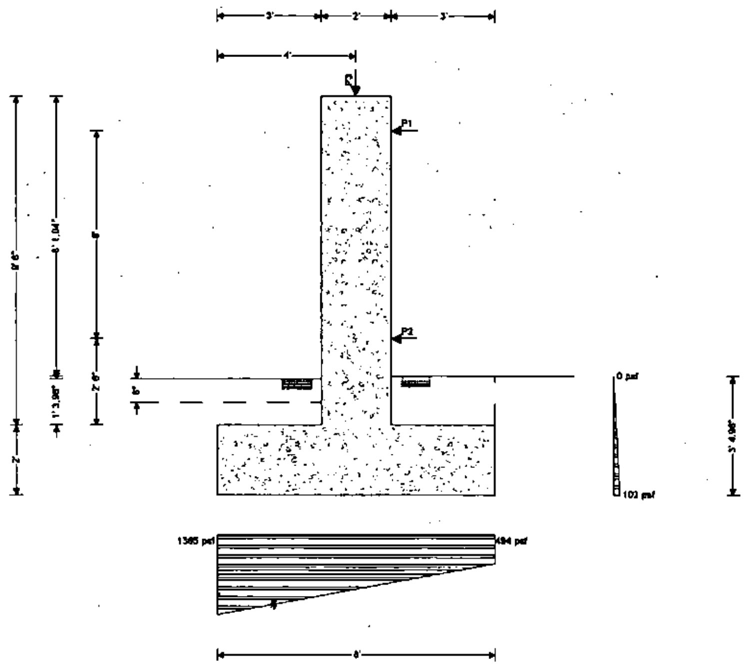


General arrangement

Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem} + l_{heel} = \mathbf{8}$ ft
Moist soil height	$h_{moist} = h_{soil} = \mathbf{1.413}$ ft
Retained surface length	$l_{sur} = l_{heel} = \mathbf{3}$ ft
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{3.413}$ ft
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem} = \mathbf{19}$ ft^2
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{4}$ ft
Area of wall base	$A_{base} = l_{base} \times t_{base} = \mathbf{16}$ ft^2
- Distance to vertical component	$x_{base} = l_{base} / 2 = \mathbf{4}$ ft
Area of moist soil	$A_{moist} = h_{moist} \times l_{heel} = \mathbf{4.24}$ ft^2
- Distance to vertical component	$x_{moist\_v} = l_{base} - (h_{moist} \times l_{heel}^2 / 2) / A_{moist} = \mathbf{6.5}$ ft
- Distance to horizontal component	$x_{moist\_h} = h_{eff} / 3 = \mathbf{1.138}$ ft
Area of base soil	$A_{pass} = d_{cover} \times l_{toe} = \mathbf{3.99}$ ft^2
- Distance to vertical component	$x_{pass\_v} = l_{base} - (d_{cover} \times l_{toe} \times (l_{base} - l_{toe} / 2)) / A_{pass} = \mathbf{1.5}$ ft
- Distance to horizontal component	$x_{pass\_h} = (d_{cover} + h_{base}) / 3 = \mathbf{1.11}$ ft
Area of excavated base soil	$A_{exc} = h_{pass} \times l_{toe} = \mathbf{1.99}$ ft^2
- Distance to vertical component	$x_{exc\_v} = l_{base} - (h_{pass} \times l_{toe} \times (l_{base} - l_{toe} / 2)) / A_{exc} = \mathbf{1.5}$ ft
- Distance to horizontal component	$x_{exc\_h} = (h_{pass} + h_{base}) / 3 = \mathbf{0.888}$ ft

Soil coefficients

Coefficient of friction to back of wall — K_{fr} = **0.325**

Coefficient of friction to front of wall — K_{fb} = **0.325**

Coefficient of friction beneath base — K_{fbb} = **0.325**

From IBC 2015 cl.1807.2.3 Safety factor

Load combination 1 — 1.0 × Dead + 1.0 × Live + 1.0 × Lateral earth

Sliding check

Vertical forces on wall

Wall stem — $F_{stem} = A_{stem} \times \gamma_{stem}$ = **2850** plf

Wall base — $F_{base} = A_{base} \times \gamma_{base}$ = **2400** plf

Line loads — $F_{P\_v} = P_{D3}$ = **1200** plf

Moist retained soil — $F_{moist\_v} = A_{moist} \times \gamma_{mr}$ = **530** plf

Base soil — $F_{exc\_v} = A_{exc} \times \gamma_b$ = **229** plf

Total — $F_{total\_v} = F_{stem} + F_{base} + F_{moist\_v} + F_{exc\_v} + F_{P\_v}$ = **7209** plf

Horizontal forces on wall

Line loads — $F_{P\_h} = P_{L1} + P_{L2}$ = **666** plf

Moist retained soil — $F_{moist\_h} = p_{Ar} \times h_{eff}^2 / 2$ = **175** plf

Total — $F_{total\_h} = F_{moist\_h} + F_{P\_h}$ = **841** plf

Check stability against sliding

Base soil resistance — $F_{exc\_h} = p_{0b} \times (h_{pass} + h_{base})^2 / 2$ = **213** plf

Base friction — $F_{friction} = F_{total\_v} \times K_{fbb}$ = **2343** plf

Resistance to sliding — $F_{rest} = F_{exc\_h} + F_{friction}$ = **2556** plf

Factor of safety — $FoS_{sl} = F_{rest} / F_{total\_h}$ = **3.04** > 1.5

PASS - Factor of safety against sliding is adequate

Overturning check

Vertical forces on wall

Wall stem — $F_{stem} = A_{stem} \times \gamma_{stem}$ = **2850** plf

Wall base — $F_{base} = A_{base} \times \gamma_{base}$ = **2400** plf

Line loads — $F_{P\_v} = P_{D3}$ = **1200** plf

Moist retained soil — $F_{moist\_v} = A_{moist} \times \gamma_{mr}$ = **530** plf

Base soil — $F_{exc\_v} = A_{exc} \times \gamma_b$ = **229** plf

Total — $F_{total\_v} = F_{stem} + F_{base} + F_{moist\_v} + F_{exc\_v} + F_{P\_v}$ = **7209** plf

Horizontal forces on wall

Line loads — $F_{P\_h} = P_{L1} + P_{L2}$ = **666** plf

Moist retained soil — $F_{moist\_h} = p_{Ar} \times h_{eff}^2 / 2$ = **175** plf

Base soil — $F_{exc\_h} = -p_{0b} \times (h_{pass} + h_{base})^2 / 2$ = **-213** plf

Total — $F_{total\_h} = F_{moist\_h} + F_{exc\_h} + F_{P\_h}$ = **628** plf

Overturning moments on wall

Line loads — $M_{P\_OT} = abs(P_{L1}) \times (p_1 + t_{base}) + abs(P_{L2}) \times (p_2 + t_{base})$ = **4995** lb_ft/ft

Moist retained soil	$M_{moist\_OT} = F_{moist\_h} \times x_{moist\_h} =$ **199** lb_ft/ft
Total	$M_{total\_OT} = M_{moist\_OT} + M_{P\_OT} =$ **5194** lb_ft/ft
Restoring moments on wall	
Wall stem	$M_{stem\_R} = F_{stem} \times x_{stem} =$ **11400** lb_ft/ft
Wall base	$M_{base\_R} = F_{base} \times x_{base} =$ **9600** lb_ft/ft
Line loads	$M_{P\_R} = abs(P_{D3}) \times p_3 =$ **4800** lb_ft/ft
Moist retained soil	$M_{moist\_R} = F_{moist\_v} \times x_{moist\_v} =$ **3445** lb_ft/ft
Base soil	$M_{exc\_R} = F_{exc\_v} \times x_{exc\_v} - F_{exc\_h} \times x_{exc\_h} =$ **532** lb_ft/ft
Total	$M_{total\_R} = M_{stem\_R} + M_{base\_R} + M_{moist\_R} + M_{exc\_R} + M_{P\_R} =$ **29777** lb_ft/ft
Check stability against overturning	
Factor of safety	$FoS_{ot} = M_{total\_R} / M_{total\_OT} =$ **5.733** > 1.5
	PASS - Factor of safety against overturning is adequate
Bearing pressure check	
Vertical forces on wall	
Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem} =$ **2850** plf
Wall base	$F_{base} = A_{base} \times \gamma_{base} =$ **2400** plf
Line loads	$F_{P\_v} = P_{D3} =$ **1200** plf
Moist retained soil	$F_{moist\_v} = A_{moist} \times \gamma_{mr} =$ **530** plf
Base soil	$F_{pass\_v} = A_{pass} \times \gamma_b =$ **459** plf
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{moist\_v} + F_{pass\_v} + F_{P\_v} =$ **7439** plf
Horizontal forces on wall	
Line loads	$F_{P\_h} = P_{L1} + P_{L2} =$ **666** plf
Moist retained soil	$F_{moist\_h} = p_{Ar} \times h_{eff}^2 / 2 =$ **175** plf
Base soil	$F_{pass\_h} = -p_{ob} \times (d_{cover} + h_{base})^2 / 2 =$ **-333** plf
Total	$F_{total\_h} = max(F_{moist\_h} + F_{pass\_h} + F_{P\_h} - F_{total\_v} \times K_{fbb}, 0\ plf) =$ **0** plf
Moments on wall	
Wall stem	$M_{stem} = F_{stem} \times x_{stem} =$ **11400** lb_ft/ft
Wall base	$M_{base} = F_{base} \times x_{base} =$ **9600** lb_ft/ft
Line loads	$M_P = P_{D3} \times p_3 - (P_{L1} \times (p_1 + t_{base}) + (P_{L2}) \times (p_2 + t_{base})) =$ **-195** lb_ft/ft
Moist retained soil	$M_{moist} = F_{moist\_v} \times x_{moist\_v} - F_{moist\_h} \times x_{moist\_h} =$ **3246** lb_ft/ft
Base soil	$M_{pass} = F_{pass\_v} \times x_{pass\_v} - F_{pass\_h} \times x_{pass\_h} =$ **1058** lb_ft/ft
Total	$M_{total} = M_{stem} + M_{base} + M_{moist} + M_{pass} + M_P =$ **25109** lb_ft/ft
Check bearing pressure	
Distance to reaction	$\bar{x} = M_{total} / F_{total\_v} =$ **3.375** ft
Eccentricity of reaction	$e = \bar{x} - l_{base} / 2 =$ **-0.625** ft
Loaded length of base	$l_{load} = l_{base} =$ **8** ft
Bearing pressure at toe	$q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) =$ **1365** psf
Bearing pressure at heel	$q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base}) =$ **494** psf
Factor of safety	$FoS_{bp} = P_{bearing} / max(q_{toe}, q_{heel}) =$ **1.099**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

RETAINING WALL DESIGN

In accordance with ACI 318-11 and MSJC-11 using the strength design method

Tedds calculation version 2.8.01

Concrete details

Compressive strength of concrete f'_c = **2500** psi

Concrete type Normal weight

Reinforcement details

Yield strength of reinforcement f_y = **60000** psi

Modulus of elasticity or reinforcement E_s = **29000000** psi

Cover to reinforcement

Top face of base c_{bt} = **2** in

Bottom face of base c_{bb} = **3** in

Masonry details

12" CMU in running bond, fully bedded with PCL class M mortar, fully grouted

Compressive strength of unit f_{cu} = **2800** psi

Net compressive strength - Table 2 f'_m = **2000** psi

Net modulus of elasticity - cl.1.8.2.2.1 $E_m = 900 \times f'_m$ = **1800000** psi

Modulus of rupture - Table 3.1.8.2 f_r = **163** psi

Thickness of unit t_b = **11.625** in

Length of unit l_b = **15.625** in

Height of unit h_b = **7.625** in

Thickness of joint t_j = **0.375** in

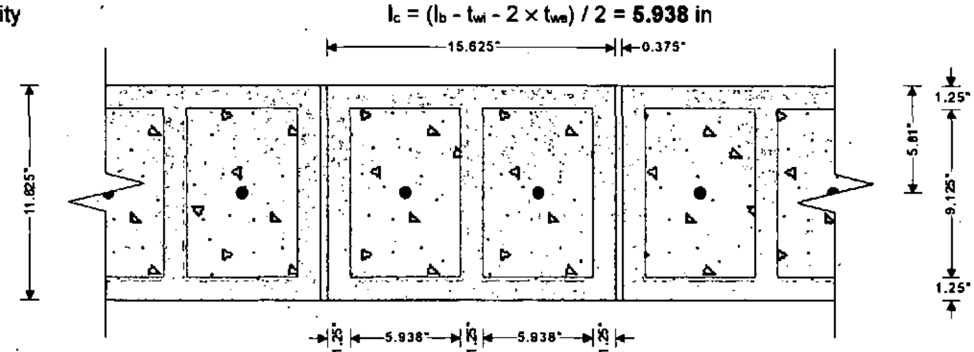
Face shell thickness t_{wf} = **1.25** in

End shell thickness t_{we} = **1.25** in

Internal web thickness t_{wi} = **1.25** in

Depth of cavity $t_c = t_b - 2 \times t_{wf}$ = **9.125** in

Length of cavity $l_c = (l_b - t_{wi} - 2 \times t_{we}) / 2$ = **5.938** in



From IBC 2015 cl.1605.2.1 Basic load combinations

Load combination no.1 1.4 × Dead

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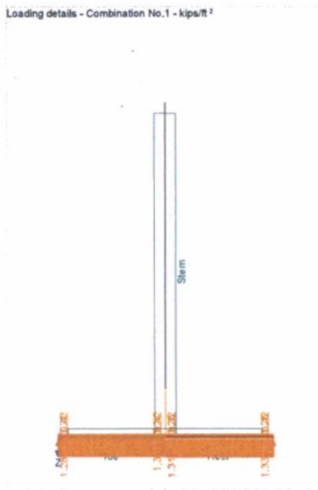
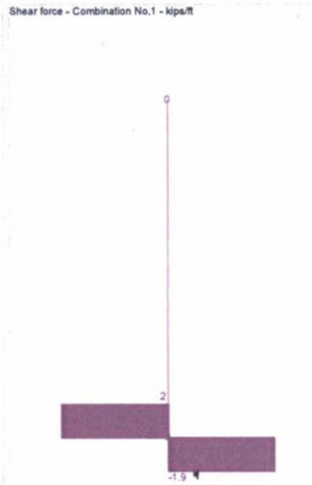
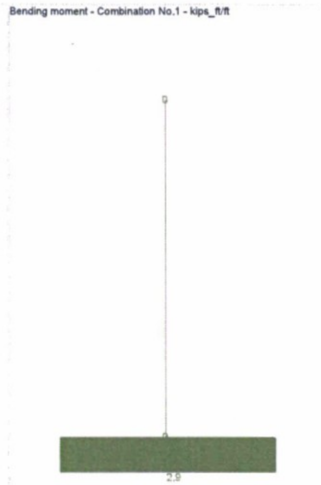
Load combination no.2 — $1.2 \times$ Dead + $1.6 \times$ Live + $1.6 \times$ Lateral earth

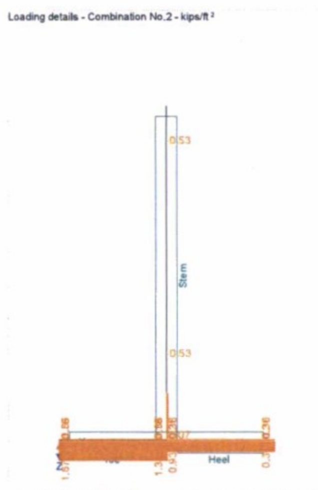
Load combination no.3 — $1.2 \times$ Dead + $1.0 \times$ Earthquake + $1.0 \times$ Live + $1.6 \times$ Lateral earth

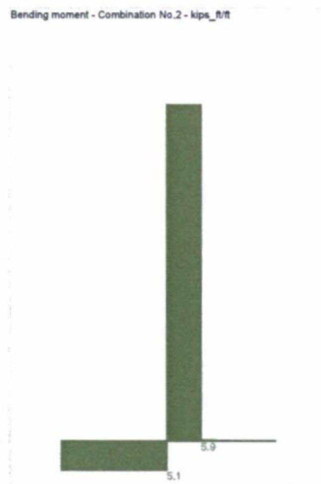
Load combination no.4 — $0.9 \times$ Dead + $1.0 \times$ Earthquake + $1.6 \times$ Lateral earth











Check stem design at base of stem

Depth of section — t = **24** in

Masonry section properties

Gross cross-sectional area — $A = t_b =$ **139.5** in^2/ft

Gross moment of inertia — $I = t_b^3 / 12 =$ **1571** in^4/ft

Gross section modulus — $S = 2 \times I / t_b =$ **270.3** in^3/ft

Gross radius of gyration — $r = \sqrt{(I / A)} =$ **3.4** in

Reinforced masonry - Section 3.3

Design bending moment combination 2 M = **70601** lb_in/ft

Axial load $P = 1.2 \times (\gamma_{stem} \times h_{stem} \times A + P_{D3})$ = **3097** lb/ft

Effective height $h = 2 \times h_{stem}$ = **19** ft

Slenderness ratio h / r = **67.941**

Nominal axial strength - exp.3-18 $P_n = 0.8 \times (0.8 \times (A - A_{sr.prov}) \times f'_m) \times [1 - (h / (140 \times r))^2]$ = **136057** lb/ft

Strength reduction factor - cl.3.1.4 ϕ = **0.9**

Design axial strength $\phi P_n = \phi \times P_n$ = **122451** lb/ft

$P / \phi P_n$ = **0.025**

PASS - Nominal axial strength exceeds axial load

Reinforcement provided No.5 bars @ 8" c/c

Area of reinforcement provided $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr})$ = **0.46** in²/ft

Depth of reinforcement d = **5.81** in

Maximum usable compressive strain of masonry - cl.3.3.2

ε_{mu} = **0.0025**

Tensile strain in reinforcement at balance point $\varepsilon_s = f_y / E_s$ = **0.002069**

Tension reinforcement strain factor α_s = **1.5**

Maximum area of reinforcement $A_{sr.max} = 0.64 \times f'_m \times d \times [\varepsilon_{mu} / (\varepsilon_{mu} + \alpha_s \times \varepsilon_s)] / f_y$ = **0.664** in²/ft

PASS - Area of stem reinforcement provided is less than maximum allowable

Distance from fiber of maximum compressive strain to neutral axis

$c = d \times \varepsilon_{mu} / (\varepsilon_{mu} + \varepsilon_s)$ = **3.179** in

Tensile force at balance point $T_b = A_{sr.prov} \times f_y$ = **27612** lb/ft

β_1 = **0.8**

Compressive force at balance point $C_b = 0.8 \times f'_m \times \beta_1 \times c$ = **48830** lb/ft

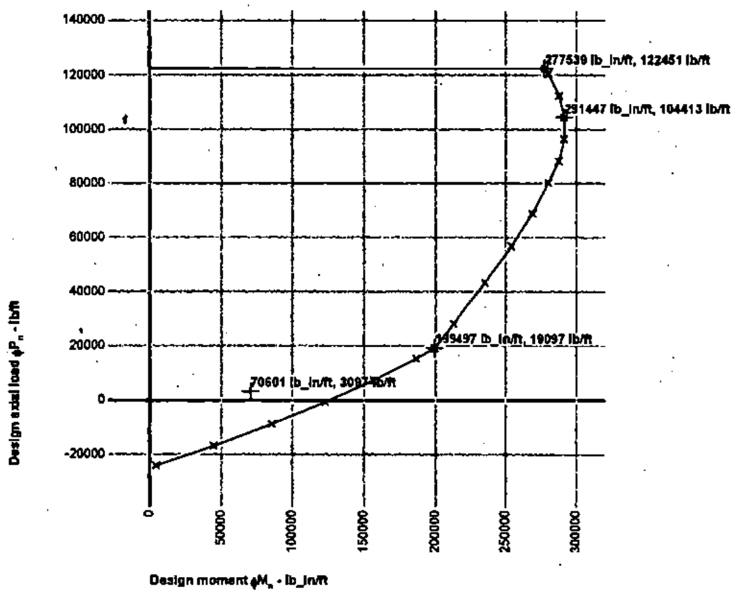
Design axial force at balance point $P_b = \phi \times (C_b - T_b)$ = **19097** lb/ft

Design moment at balance point $M_b = \phi \times (T_b \times (d - t_b / 2) + C_b \times (t_b / 2 - \beta_1 \times c / 2))$ = **199497** lb_in/ft

Strength interaction diagram

c / d	c (in)	C (lb/ft)	T (lb/ft)	f_s (psi)	M (lb_in/ft)	P (lb/ft)
0.01	0.058	892	27612	60000	4588	-24047
0.1	0.581	8924	27612	60000	44756	-16819
0.2	1.162	17848	27612	60000	85841	-8787
0.3	1.743	26772	27612	60000	123192	-755
0.4	2.324	35697	27612	60000	156811	7276
0.5	2.905	44621	27612	60000	186696	15308
0.547	3.179	48830	27612	60000	199497	19097

0.6	3.486	53545	22243	48333	212860	28172
0.7	4.067	62469	14299	31071	235297	43353
0.8	4.648	71393	8341	18125	253996	56747
0.9	5.229	80317	3707	8056	268960	68949
1	5.81	89242	0	0	280187	80317
1.1	6.391	98166	0	0	287674	88349
1.2	6.972	107090	0	0	291427	96381
1.3	7.553	116014	0	0	291447	104413
1.4	8.134	124938	0	0	287734	112444
1.5	8.715	133862	0	0	280288	120476
1.6	9.296	142787	0	0	277539	122451



From strength interaction diagram...

Maximum moment M_{max} = **291447** lb_in/ft

Limiting moment under applied axial load M_{limit} = **139315** lb_in/ft

M / M_{limit} = **0.507**

PASS - Design flexural strength exceeds factored bending moment

Design shear force V = **1114** lb/ft

Nominal shear strength - cl.3.3.4.1.2 $V_n = \min((4 - 1.75 \times \min(M / (V \times t_b), 1)) \times A \times \sqrt{(f'_m \times 1\text{ psi})} + 0.25 \times P, 4 \times A \times \sqrt{(f'_m \times 1\text{ psi})})$ = **14811** lb/ft

Strength reduction factor - cl.3.1.4 — ϕ_v = **0.8**

Design shear strength — $\phi V_n = \phi_v \times V_n$ = **11849** lb/ft

$V / \phi V_n$ = **0.094**

PASS - Design shear strength exceeds applied shear force

Check base design at toe

Depth of section — h = **24** in

Rectangular section in flexure - Chapter 10

Design bending moment combination 2 — M = **5137** lb_ft/ft

Depth of tension reinforcement — $d = h - c_{bb} - \phi_{bb} / 2$ = **20.625** in

Compression reinforcement provided — No.6 bars @ 10" c/c

Area of compression reinforcement provided — $A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt})$ = **0.53** in^2/ft

Tension reinforcement provided — No.6 bars @ 10" c/c

Area of tension reinforcement provided — $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **0.53** in^2/ft

Maximum reinforcement spacing - cl.10.5.4 — s_{max} = min(18 in, 3 × h) = **18** in

PASS - Reinforcement is adequately spaced

Depth of compression block — $a = A_{bb.prov} \times f_y / (0.85 \times f'_c)$ = **1.247** in

Neutral axis factor - cl.10.2.7.3 — β_1 = min(max(0.85 - 0.05 × (f'c - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**

Depth to neutral axis — $c = a / \beta_1$ = **1.468** in

Strain in reinforcement — $\varepsilon_t = 0.003 \times (d - c) / c$ = **0.039163**

Section is in the tension controlled zone

Strength reduction factor — ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.9**

Nominal flexural strength — $M_n = A_{bb.prov} \times f_y \times (d - a / 2)$ = **53018** lb_ft/ft

Design flexural strength — $\phi M_n = \phi_f \times M_n$ = **47716** lb_ft/ft

$M / \phi M_n$ = **0.108**

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis — $A_{bb.des}$ = **0.056** in^2/ft

Minimum area of reinforcement - cl.7.12.2.1 — $A_{bb.min}$ = 0.0018 × h = **0.518** in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Chapter 11

Design shear force — V = **3140** lb/ft

Concrete modification factor - cl.8.6.1 — λ = **1**

Nominal concrete shear strength - eqn.11-3 — $V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times d$ = **24750** lb/ft

Strength reduction factor — ϕ_s = **0.75**

Design concrete shear strength - cl.11.4.6.1 — $\phi V_c = \phi_s \times V_c$ = **18563** lb/ft

$V / \phi V_c$ = **0.169**

PASS - No shear reinforcement is required

Check base design at heel

Depth of section — h = **24** in

Rectangular section in flexure - Chapter 10

Design bending moment combination 2 — M = **185** lb_ft/ft

Depth of tension reinforcement — $d = h - c_{bt} - \phi_{bt} / 2$ = **21.625** in

Compression reinforcement provided — No.6 bars @ 10" c/c

Area of compression reinforcement provided — $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **0.53** in²/ft

Tension reinforcement provided — No.6 bars @ 10" c/c

Area of tension reinforcement provided — $A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt})$ = **0.53** in²/ft

Maximum reinforcement spacing - cl.10.5.4 — s_{max} = min(18 in, 3 × h) = **18** in

PASS - Reinforcement is adequately spaced

Depth of compression block — $a = A_{bt.prov} \times f_y / (0.85 \times f'_c)$ = **1.247** in

Neutral axis factor - cl.10.2.7.3 — β_1 = min(max(0.85 - 0.05 × (f'c - 4 ksi) / 1 ksi, 0.65), 0.85) = **0.85**

Depth to neutral axis — $c = a / \beta_1$ = **1.468** in

Strain in reinforcement — $\varepsilon_t = 0.003 \times (d - c) / c$ = **0.041207**

Section is in the tension controlled zone

Strength reduction factor — ϕ_f = min(max(0.65 + (ε_t - 0.002) × (250 / 3), 0.65), 0.9) = **0.9**

Nominal flexural strength — $M_n = A_{bt.prov} \times f_y \times (d - a / 2)$ = **55669** lb_ft/ft

Design flexural strength — $\phi M_n = \phi_f \times M_n$ = **50102** lb_ft/ft

$M / \phi M_n$ = **0.004**

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis — $A_{bt.des}$ = **0.002** in²/ft

Minimum area of reinforcement - cl.7.12.2.1 — $A_{bt.min} = 0.0018 \times h$ = **0.518** in²/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Chapter 11

Design shear force — V = **1947** lb/ft

Concrete modification factor - cl.8.6.1 — λ = **1**

Nominal concrete shear strength - eqn.11-3 — $V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1\ \text{psi})} \times d$ = **25950** lb/ft

Strength reduction factor — ϕ_s = **0.75**

Design concrete shear strength - cl.11.4.6.1 — $\phi V_c = \phi_s \times V_c$ = **19463** lb/ft

$V / \phi V_c$ = **0.100**

PASS - No shear reinforcement is required

Transverse reinforcement parallel to base

Minimum area of reinforcement - cl.7.12.2.1 — $A_{bx.req} = 0.0018 \times t_{base}$ = **0.518** in²/ft

Transverse reinforcement provided — No.4 bars @ 8" c/c each face

Area of transverse reinforcement provided — $A_{bx.prov} = 2 \times \pi \times \phi_{bx}^2 / (4 \times s_{bx})$ = **0.589** in²/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required

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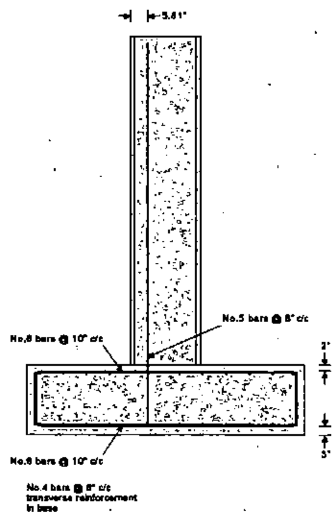
Project	Lomas Verdes Estates	Sheet No.	11
		Project No.	17-050
		Date	5/12/2017
Subject	EDGE Masonry Pier	Computed By	CB



Reinforcement details

Strain in reinforcement $\varepsilon_t = 0.003 \times (d - c) / c =$ **0.041207**

Section is in the tension controlled zone

Strength reduction factor $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) =$ **0.9**

Nominal flexural strength $M_n = A_{bt.prov} \times f_y \times (d - a / 2) =$ **55669** lb_ft/ft

Design flexural strength $\phi M_n = \phi_f \times M_n =$ **50102** lb_ft/ft

$M / \phi M_n =$ **0.001**

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis $A_{bt.des} =$ **0** in^2/ft

Minimum area of reinforcement - cl.7.12.2.1 $A_{bt.min} = 0.0018 \times h =$ **0.518** in^2/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Chapter 11

Design shear force $V =$ **2155** lb/ft

Concrete modification factor - cl.8.6.1 $\lambda =$ **1**

Nominal concrete shear strength - eqn.11-3 $V_c = 2 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times d =$ **25950** lb/ft

Strength reduction factor $\phi_s =$ **0.75**

Design concrete shear strength - cl.11.4.6.1 $\phi V_c = \phi_s \times V_c =$ **19463** lb/ft

$V / \phi V_c =$ **0.111**

PASS - No shear reinforcement is required

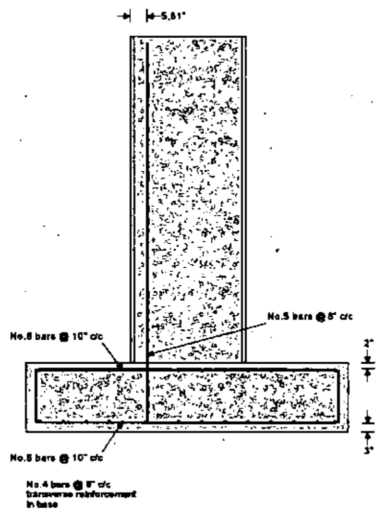
Transverse reinforcement parallel to base

Minimum area of reinforcement - cl.7.12.2.1 $A_{bx.req} = 0.0018 \times t_{base} =$ **0.518** in^2/ft

Transverse reinforcement provided No.4 bars @ 8" c/c each face

Area of transverse reinforcement provided $A_{bx.prov} = 2 \times \pi \times \phi_{bx}^2 / (4 \times s_{bx}) =$ **0.589** in^2/ft

PASS - Area of reinforcement provided is greater than area of reinforcement required



Reinforcement details

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Sheet No.

Project No. 17-050

Project Name Lomas Verdes

Date May 2017

Subject

Computed By CB

GATE ANCHORAGE

15 psf x 8 ft x 10 ft = 1200 pounds

Tension on top anchor:
1200 pounds x 5 ft / 6ft = 1000 pounds of tension (allowable)

Utilize cast-in-place anchors for ease of hardware installation

3/4" diameter anchor with 4" embedment
As = 0.44 in^2

TENSION CAPACITY
Apt = π * 4^2
Apt = 50.2 sq inches

Bab = 1.25 * Apt * f'm^(0.5) = 2.43k < 1.0k - OK!
Bas = 0.6Asfy = 0.6*.392*36 = 9.50k

SHEAR CAPACITY
Apt = π * 4^2 / 2
Apt = 25.1 sq inches

Bvb = 1.25 * Apt * f'm^(0.5) = 1.22k
Bvs = 350 * (f'm * Ab)^1/4 = 1.77k

There are 4 anchors to support the gate weight = 1.2k - OK!