APPENDIX K

HYDROLOGY SUPPORTING DOCUMENTS

FEDERAL EMERGENCY MANAGEMENT AGENCY FLOOD HAZARD MAP
PRELIMINARY ON-SITE AND OFF-SITE HYDROLOGY AND FLOOD HAZARD ANALYSIS
RETENTION BASIN INFILTRATION TEST RESULTS
PRELIMINARY GEOTECHNICAL INVESTIGATION
MEMORANDUM RE: SAN FELIPE CREEK/SEVILLE SOLAR COMPLEX RESPONSE

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Federal Emergency Management Agency Flood Hazard Map

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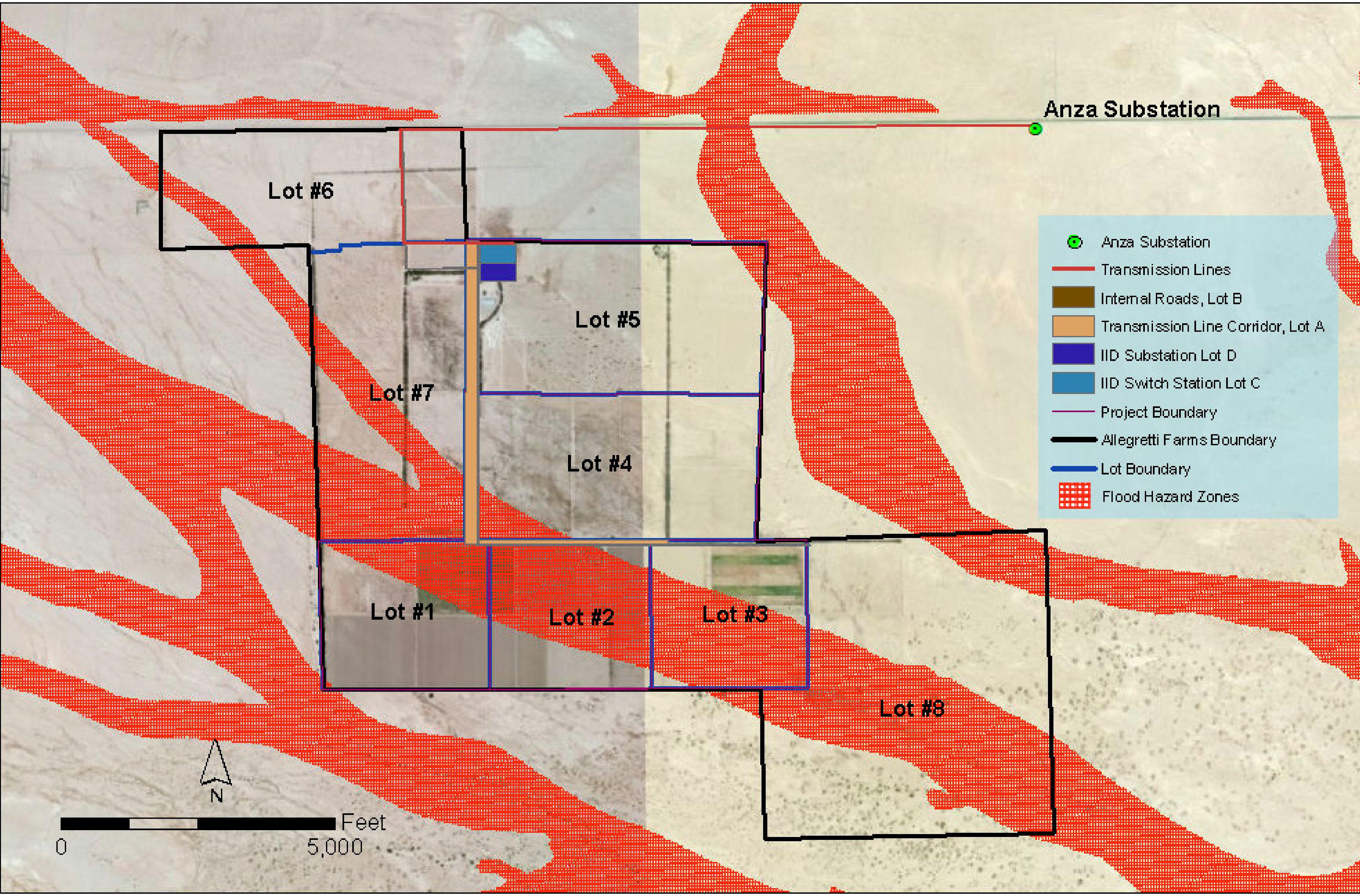

Anza Substation
Lot #6
Lot #5
Lot #7
Lot #4
Lot #1
Lot #2
Lot #3
Lot #8
Anza Substation
Transmission Lines
Internal Roads, Lot B
Transmission Line Corridor, Lot A
IID Substation Lot D
IID Switch Station Lot C
Project Boundary
Allegretti Farms Boundary
Lot Boundary
Flood Hazard Zones
N
Feet
0
5,000

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Preliminary On-Site and Off-Site Hydrology and Flood Hazard Analysis

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A. EXISTING ON-SITE AND OFF-SITE HYDROLOGY ANALYSIS

On-site and off-site hydrology maps with peak flow rate calculations at critical nodal points have been developed for the Allegretti Farms Solar Project Site (Site).

On-Site Hydrology - The on-site hydrology map (Attachment A) and calculations were determined using the Riverside County Rational Method, analyzing the main watercourses throughout the Site at various concentration points.

Off-Site Hydrology - The off-site hydrology map (Attachment B) and calculations were determined using the Riverside County Unit Hydrograph Method for four (4) of the five (5) off-site areas (Areas B, C, D, and E) in order to determine the 100-year, 1-hour peak flows along the upstream boundary of the Site.

The fifth off-site area (Area A) is significantly larger (approximately 400,000 acres) than the aforementioned off-site areas and exceeds the limitations of many traditional hydrologic computation methods. Due to its size, it was necessary to run a separate 100-year, 24-hr study on Area A to find its peak flow rate. Rather than generating a single unit hydrograph, similar to the other off-site areas, Area A was divided into eight (8) subareas, each with a separate set of parameters and its own computed unit hydrograph. These eight (8) unit hydrographs were then routed using the SCS Convex Channel Routing Method based on the parameters of each subarea in order to generate one inclusive unit hydrograph. This was used to determine the peak 100-yr, 24-hr flow rate for Area A.

Flows generated by off-site Area B concentrate at off-site Node 201 (see Off-site Hydrology Map – Attachment B), which is located along an existing natural watercourse that traverses around the northeast corner of the Site.

Off-site Area E, the smallest of the off-site watersheds, drains onto the site at off-site Node 501 via sheet flow. Since off-site Area E is relatively flat and smaller in size, there exists no defined watercourse at the outlet location along the northern boundary of the Site.

Flows generated by off-site Area D are directed via an existing levee along the northern side of Highway 78 to a break location where runoff crosses the highway via surface flow. From here, flows continue southerly approximately 2,100 feet before flowing across the northern boundary of the site.

Flows generated by off-site Areas A and C are concentrated along the western boundary of the site where there is an existing 7-foot-high (approximate) earthen berm that extends from the northwest corner of the site to the southwest corner of the site. The structural integrity of the earthen berm has not been verified.

B. EXISTING EARTHEN BERM ALONG WESTERLY BOUNDARY

AEI-CASC prepared a preliminary hydraulic (normal depth) analysis of the flows impacting the northerly and southerly sections of the earthen berm along the westerly project boundary. The analysis assumed that the berm is structurally sound and capable of withholding the off-site flows. Further studies, including a scour analysis and geotechnical investigation, will likely be required to confirm the structural integrity of the earthen berm.

Northerly Berm Section (off-site Area C and Node 301) - Using United States Geological Survey (USGS) maps and digital aerial topography of the Site, cross sections at critical locations along the earthen berm were generated and analyzed for capacity using the Normal Depth Method. The channel created by the berm has an approximate capacity of 25,740 cfs at off-site Node 301, the concentration point of off-site Area C. Since Area C generates a computed 100-yr, 1-hr peak flow rate of approximately 2,557 cfs, flows are assumed to be fully contained off-site by the earthen berm at this location and subsequently directed southerly via surface flow toward the southwest corner of the Site.

Southerly Berm Section (off-site Area A and Node 108) - Using the Normal Depth Method again, this time at the concentration point of off-site Area A, the cross section of the channel created by the earthen berm at off-site Node 108 was generated and analyzed, and found to have an approximate capacity of 173,496 cfs. Since Area A generates a 100-yr, 24-hr peak flow rate of approximately 68,100 cfs at this location, flows generated by both Area A and Area C are assumed to be fully contained off-site by the berm at this location and directed around the southwest corner of the site. From the southwest corner of the site, runoff continues downstream via surface flow following the existing natural watercourse.

C. PRELIMINARY SAN FELIPE CREEK FLOOD HAZARD ANALYSIS

The FEMA map covering the project area does not recognize the berm and depicts a flood zone traversing the southwesterly corner of the site.

At the client’s request, AEI-CASC prepared a preliminary flood hazard analysis assuming the earthen berm along the westerly property line will not prevent off-site flood flows from entering the property. The purpose of the analysis is to determine a preliminary understanding of potential channel flow velocities and flood depths across the site.

The flood hazard analysis was based upon the preliminary off-site hydrologic analysis that was developed to determine peak flow rates impacting the site during a 100-year storm event. The off-site hydrologic analysis covered an offsite tributary area of nearly 400,000 acres, and resulted in a peak 100-year flow rate of 68,100 cfs that would impact the southwesterly boundary of the site.

This flood hazard study utilized the Normal-Depth Method to determine the flooding limits and depths. Two (2) stream cross-sections (Cross-Section 2 and 5, see Table 1) were developed within the project site (perpendicular to the subject watershed flow line) and were analyzed based upon an assumed Manning's n-value of 0.030 (typical for earthen/sandy bottom floodplains). The results of the analysis are shown in Table 1 (below). Approximate flood depths were determined for Cross-Sections 1, 3, 4, 6 and 7 by interpolating and projecting the results from Cross-Sections 2 and 5. The average flood velocities determined from Cross-Sections 2 and 5 were applied to Cross-Sections 1, 3, 4, 6 and 7. While the flow velocities are estimated to be approximately 4.5 feet/sec in the deepest section of the flood plain, the velocities along the fringe (shallower edges) of the floodplain are expected to be somewhat lower.

TABLE 1: PRELIMINARY HYDRAULIC RESULTS (BASED ON NORMAL DEPTH METHOD)

CROSS-SECTION	FLOOD DEPTH (FT)	FLOOD TOP WIDTH (FT)	FLOOD VELOCITY (FPS)
2	4.61	7,538	4.53
5	5.83	8,022	4.48

As shown on Exhibit "C", the computed flood hazard area encompasses a larger area than FEMA's current floodplain area. In particular, the majority of the southwest corner of the project site is located within the computed flood hazard area. The difference between AEI-CASC's computed flood hazard area and FEMA's floodplain area may be attributed to the topographic mapping and 100-year flow rate used in the analysis. It is assumed that FEMA utilized United States Geological Survey (USGS) maps consisting of 10-foot contour intervals while AEI-CASC used a more detailed topographic mapping consisting of 2-foot contour intervals. AEI-CASC was not able to determine the 100-year flow rate that FEMA used as the basis for their floodplain analysis. However, if FEMA used a significantly lower 100-year flow rate in its study, then this might explain the difference between AEI-CASC's and FEMA's studies.

Limitations: The normal depth methodology used in the analysis is intended to be used for preliminary planning purposes and should not be used for final engineering design. Detailed hydraulic modeling using the Hydrologic Engineering Centers River Analysis System (HEC-RAS) may be performed to determine more accurate modeling of the flood plain limits, velocities and depths across the site.

Attachments:

A. Existing On-site Hydrology Map
B. Existing Off-Site Hydrology Map
C. Preliminary 100-year Flood Hazard Map for Allegretti Farms Tract Map
D. Preliminary Hydraulic Analysis (Normal Depth Method) for Cross-Sections 2 and 5

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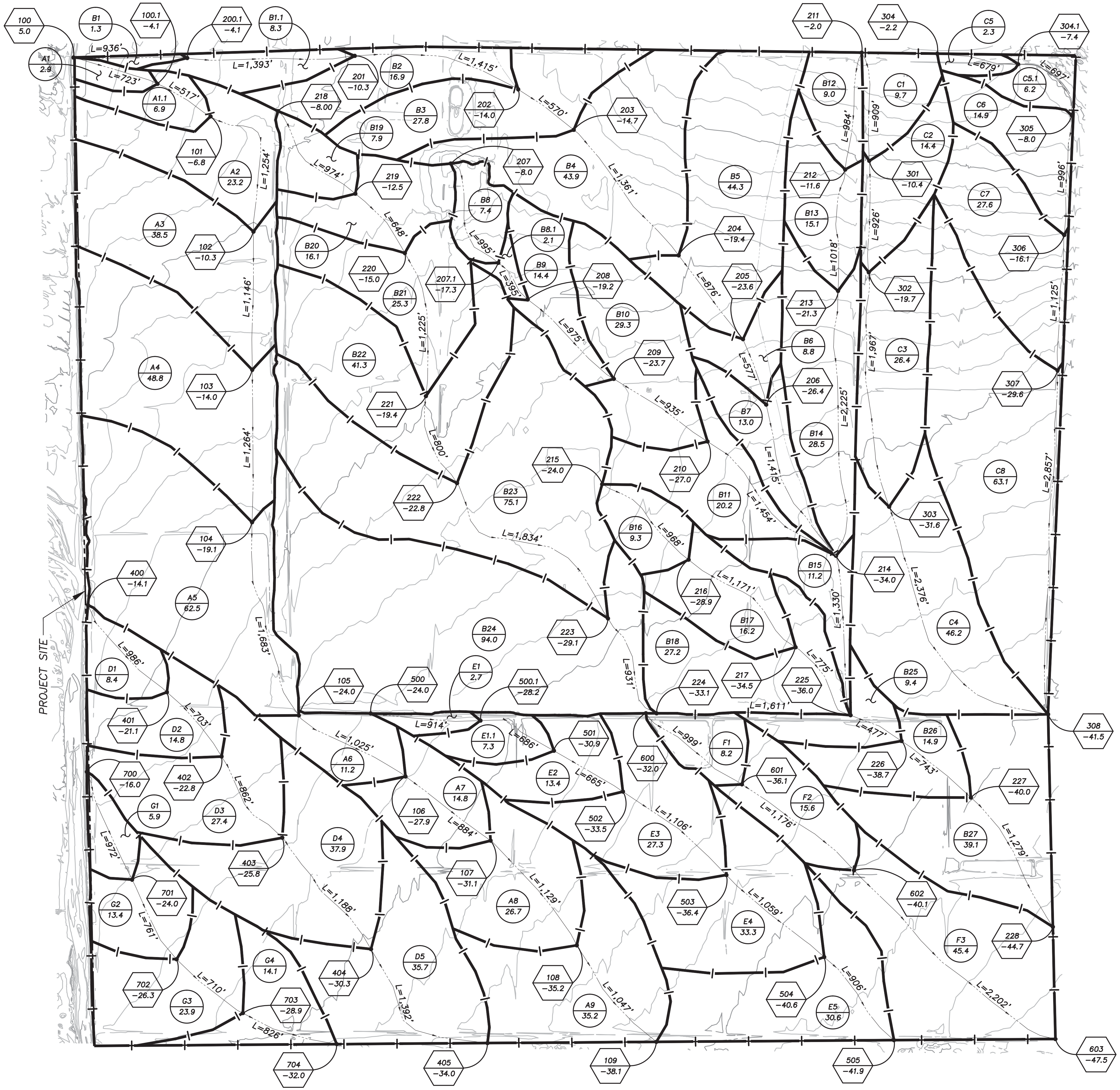
ATTACHMENT A
EXISTING ON-SITE HYDROLOGY MAP



HYDROLOGY DATA

NODAL POINT	10-YR, 1-HR (CFS)	100-YR, 1-HR (CFS)
109	340	742
214	342	744
225	696	1,512
228	749	1,636
308	263	563
405	182	381
505	149	317
603	96	201
704	94	194

LEGEND

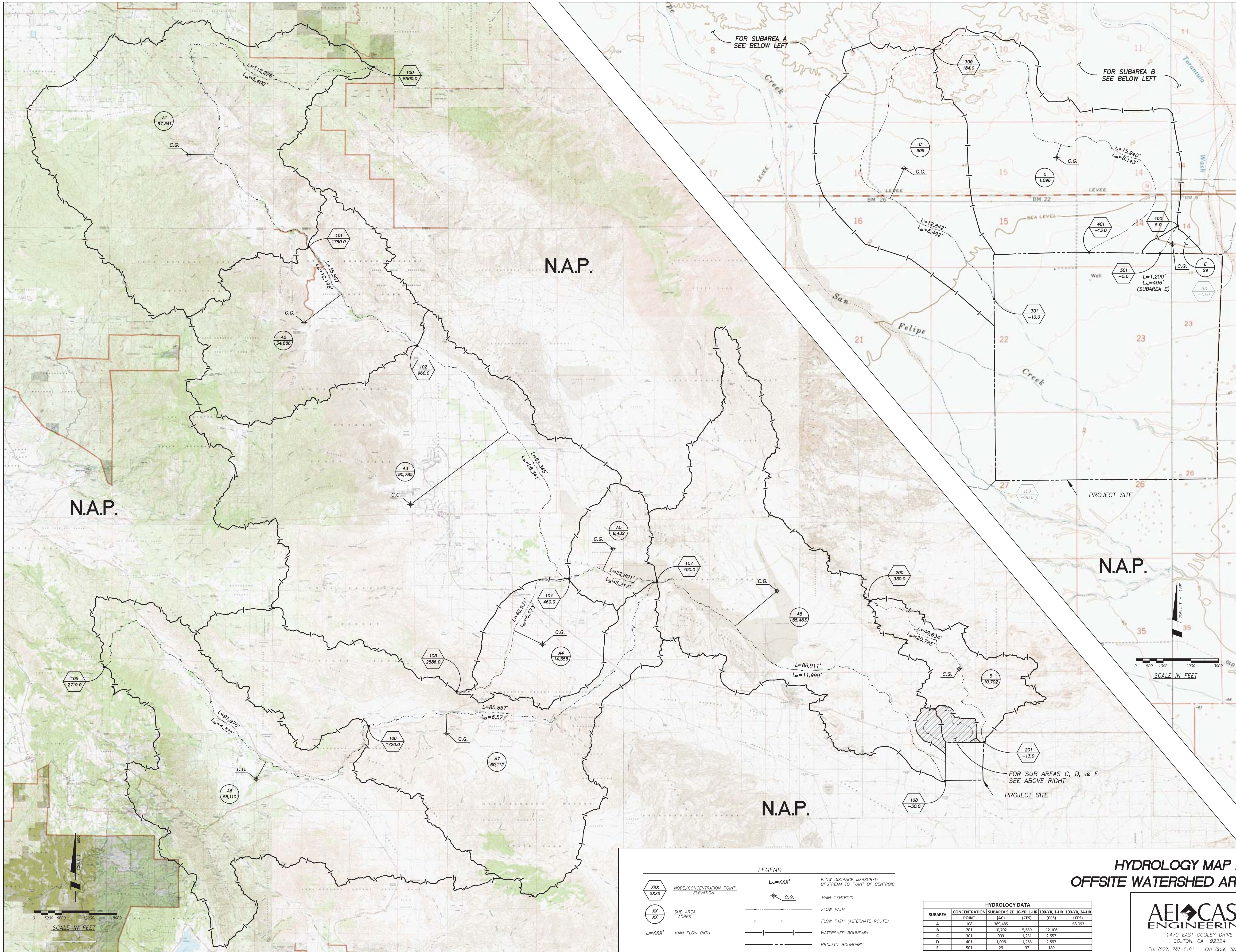
XXX / XXXX	NODE/CONCENTRATION POINT ELEVATION		FLOW PATH
			FLOW PATH (ALTERNATE ROUTE)
XX / XX	SUB AREA ACRES		WATERSHED BOUNDARY
			PROJECT BOUNDARY
L=XXX'	MAIN FLOW PATH		
L_{CA}=XXX'	FLOW DISTANCE MEASURED UPSTREAM TO POINT OF CENTROID		

HYDROLOGY MAP FOR ONSITE WATERSHED AREAS (EXISTING CONDITION)

AEI CASC ENGINEERING
1470 EAST COOLEY DRIVE
COLTON, CA 92324
PH. (909) 783-0101 FAX (909) 783-0108

Drawing Name: Q:\1330-001\Master_Drainage\Exhibits\Hydrology Map On-Site (Existing Condition).dwg
Last Opened: Jan 30, 2013 - 11:50am by amistretta

ATTACHMENT B
EXISTING OFF-SITE HYDROLOGY MAP



HYDROLOGY MAP FOR OFFSITE WATERSHED AREAS

LEGEND

XXX / XXXX	NODE/CONCENTRATION POINT / ELEVATION
XX / XX	SUB AREA / ACRES
L=XXX'	MAIN FLOW PATH
L_{CA}=XXX'	FLOW DISTANCE MEASURED UPSTREAM TO POINT OF CENTROID
C.G.	MAIN CENTROID
	FLOW PATH
	FLOW PATH (ALTERNATE ROUTE)
	WATERSHED BOUNDARY
	PROJECT BOUNDARY

HYDROLOGY DATA

SUBAREA	CONCENTRATION POINT	SUBAREA SIZE (AC)	10-YR, 1-HR (CFS)	100-YR, 1-HR (CFS)	100-YR, 24-HR (CFS)
A	108	389,485	-	-	68,093
B	201	10,702	5,659	12,106	-
C	301	909	1,251	2,557	-
D	401	1,096	1,263	2,597	-
E	501	29	97	189	-

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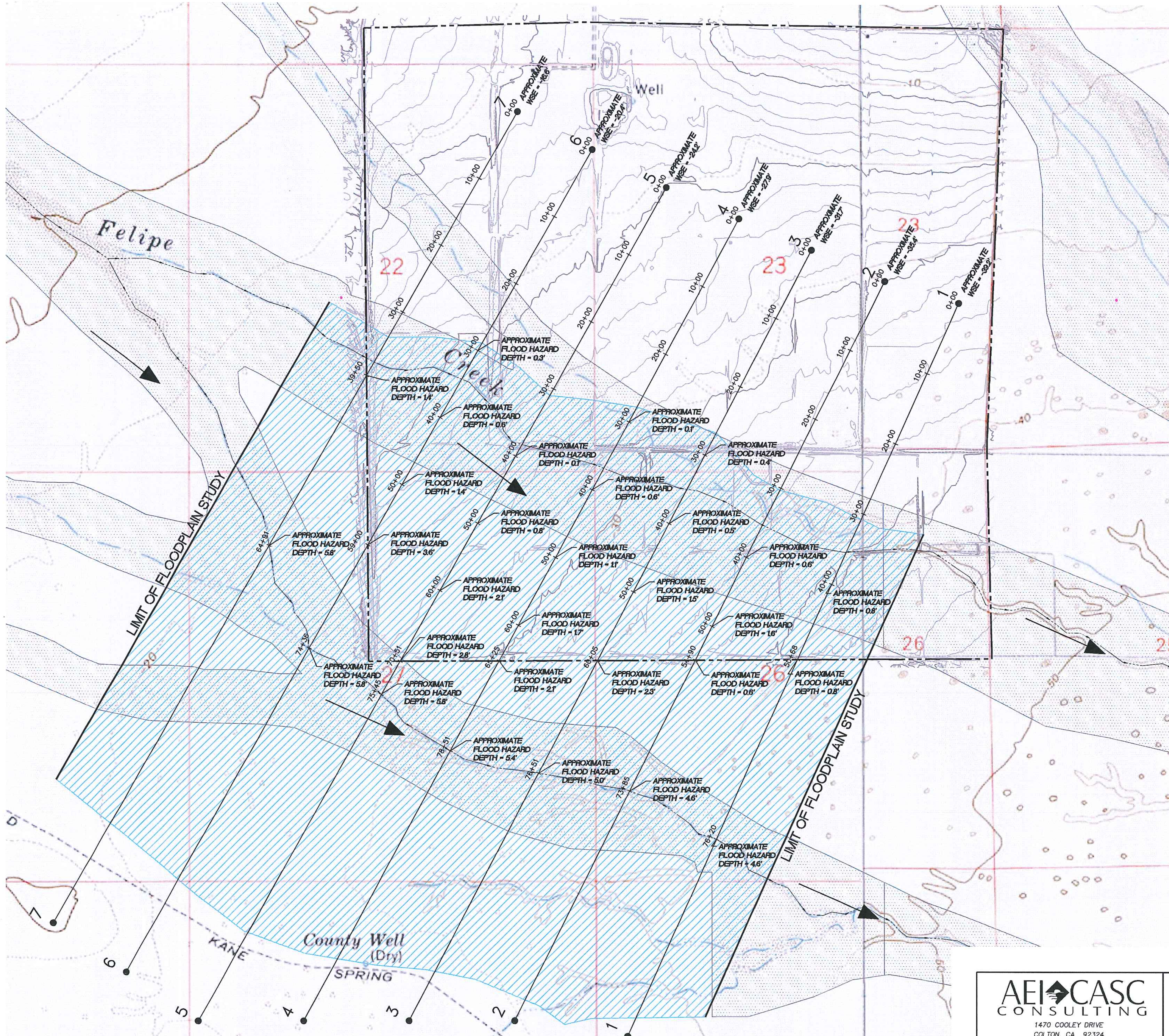
COLTON, CA 92324
PH. (909) 783-0101 FAX (909) 783-0108

ATTACHMENT C
PRELIMINARY 100-YEAR FLOOD HAZARD MAP



HYDRAULIC SUMMARY (EXISTING CONDITION)
(ASSUMMING NO LEVEE ALONG WESTERLY PROPERTY LINE)

CROSS-SECTION	*APPROXIMATE CHANNEL INVERT (FT)	100-YEAR FLOW RATE (CFS)	100-YEAR WATER SURFACE ELEVATION W.S.E (FT)	100-YEAR *DEPTH (FT)	100-YEAR *VELOCITY (FPS)
1	-48.3	68,100	-39.2	4.6	4.5
2	-40.0	68,100	-35.4	4.6	4.5
3	-36.7	68,100	-31.7	5.0	4.5
4	-33.3	68,100	-27.9	5.4	4.5
5	-30.0	68,100	-24.2	5.8	4.5
6	-26.2	68,100	-20.4	5.8	4.5
7	-22.4	68,100	-16.6	5.8	4.5

*NOTE: VALUES IN THE ABOVE TABLE FOR FLOODPLAIN FLOW VELOCITIES, FLOW DEPTHS AND APPROXIMATE CHANNEL INVERTS ONLY CORRELATE TO THE CHANNEL FLOW LINE SOUTH OF THE PROPERTY. FLOW VELOCITIES ALONG THE FRINGE (SHALLOWER EDGES) OF THE FLOODPLAIN ARE EXPECTED TO BE SOMEWHAT REDUCED.

ADDITIONAL NOTE: PRELIMINARY FLOODPLAIN ANALYSIS NORMAL DEPTH METHODOLOGY IS BASED ON MULTIPLE ASSUMPTIONS. APPROXIMATE FLOOD HAZARD DEPTHS AND VELOCITIES SHOULD NOT BE USED FOR DESIGN PURPOSES. THIS EXHIBIT SHOULD BE USED FOR PLANNING PURPOSES ONLY.

LEGEND:

- 1 — 1 CROSS-SECTION
- STREAM 100-YR FLOODPLAIN
- FEMA 100-YR FLOODPLAIN ZONE A
- PROJECT BOUNDARY
- FLOW DIRECTION
- FLOW LINE

N

GRAPHIC SCALE
600 0 300 600 1200
1" = 600'

EXHIBIT "A"

AEI CASC CONSULTING
1470 COOLEY DRIVE
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PRELIMINARY 100-YEAR FLOOD HAZARD MAP FOR ALLEGRETTI FARMS TRACT MAP (NORMAL DEPTH METHOD)

ATTACHMENT D
PRELIMINARY HYDRAULIC ANALYSIS FOR CROSS SECTIONS 2 AND 5

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Worksheet for Irregular Section - 2

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00334	ft/ft
Discharge	68100.00	ft^3/s
Section Definitions		

Station (ft)	Elevation (ft)
7+33	-29.67
8+49	-33.20
9+57	-34.00
11+47	-34.00
12+67	-33.75
13+89	-33.33
15+04	-33.74
16+36	-34.32
17+42	-34.55
18+49	-34.52
19+79	-34.38
21+18	-34.43
22+20	-33.99
23+25	-34.00
24+37	-34.43
25+53	-35.93
26+74	-35.25
27+76	-35.64
28+82	-35.70
30+01	-35.74
31+06	-35.93
32+31	-35.97
33+41	-36.00
34+48	-36.00
35+54	-36.00
36+78	-36.00
37+79	-36.00

Worksheet for Irregular Section - 2

Input Data

Station (ft)	Elevation (ft)
38+79	-36.02
40+19	-36.30
41+27	-36.51
42+54	-36.54
43+73	-36.60
44+98	-36.85
46+02	-36.99
47+26	-37.20
48+40	-37.25
49+53	-37.27
50+63	-37.49
51+93	-37.84
53+00	-38.00
54+03	-36.08
54+67	-36.03
67+02	-37.00
69+43	-40.00
70+62	-37.00
71+96	-37.00
73+41	-40.00
74+58	-38.00
75+83	-40.00
81+25	-38.00
83+99	-38.00
87+38	-40.00
90+79	-38.00
95+71	-38.00
97+54	-40.00
105+28	-30.00

Worksheet for Irregular Section - 2

Input Data

Start Station	Ending Station	Roughness Coefficient
(7+33, -29.67)	(105+28, -30.00)	0.030

Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results

Normal Depth		4.61	ft
Elevation Range	-40.00 to -29.67 ft		
Flow Area		15025.15	ft²
Wetted Perimeter		7537.94	ft
Hydraulic Radius		1.99	ft
Top Width		7537.74	ft
Normal Depth		4.61	ft
Critical Depth		3.80	ft
Critical Slope		0.01115	ft/ft
Velocity		4.53	ft/s
Velocity Head		0.32	ft
Specific Energy		4.93	ft
Froude Number		0.57	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	4.61	ft

Worksheet for Irregular Section - 2

GVF Output Data

Critical Depth	3.80	ft
Channel Slope	0.00334	ft/ft
Critical Slope	0.01115	ft/ft

Messages

Notes

Slope = 6/1797.68 = 0.003338

Area A (Node 101) 100-Year Peak Flow = 68,093 CFC ~ 68,100 CFS

Worksheet for Irregular Section - 5

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00348	ft/ft
Discharge	68100.00	ft^3/s
Section Definitions		

Station (ft)	Elevation (ft)
0+00	-20.00
1+00	-20.00
2+09	-20.00
3+63	-20.15
4+78	-20.37
5+93	-20.50
7+05	-20.50
8+21	-20.67
9+34	-20.75
10+59	-21.00
11+60	-21.22
12+89	-21.60
14+14	-21.85
16+03	-22.33
17+16	-22.41
18+17	-22.46
19+58	-23.20
20+65	-23.25
21+68	-23.27
23+04	-23.26
24+40	-23.30
25+49	-23.46
26+50	-23.73
27+89	-23.93
29+09	-23.99
30+16	-24.00
32+35	-24.49

Worksheet for Irregular Section - 5

Input Data

Station (ft)	Elevation (ft)
33+41	-24.73
34+78	-25.02
36+02	-25.37
37+06	-25.53
38+25	-26.00
39+37	-24.02
41+49	-24.30
42+57	-24.15
43+70	-24.38
45+03	-24.59
46+17	-24.69
47+21	-24.86
48+40	-24.82
49+61	-25.05
51+05	-25.35
52+67	-25.75
53+92	-25.76
54+95	-26.00
56+54	-26.12
57+54	-26.14
58+82	-26.13
60+09	-26.20
61+37	-26.31
62+56	-26.39
63+76	-26.33
65+07	-26.40
66+14	-26.46
67+16	-26.03
68+25	-26.04
69+27	-26.06
70+00	-27.05
75+16	-30.00
91+10	-26.00
114+77	-24.00
127+36	-20.00

Section Definitions

Worksheet for Irregular Section - 5

Input Data

Start Station	Ending Station	Roughness Coefficient
(0+00, -20.00)	(127+36, -20.00)	0.030

Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results

Normal Depth		5.83	ft
Elevation Range	-30.00 to -20.00 ft		
Flow Area		15213.24	ft²
Wetted Perimeter		8022.17	ft
Hydraulic Radius		1.90	ft
Top Width		8022.13	ft
Normal Depth		5.83	ft
Critical Depth		4.97	ft
Critical Slope		0.01112	ft/ft
Velocity		4.48	ft/s
Velocity Head		0.31	ft
Specific Energy		6.14	ft
Froude Number		0.57	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth		0.00	ft
Length		0.00	ft
Number Of Steps		0	

GVF Output Data

Upstream Depth		0.00	ft
Profile Description			

Worksheet for Irregular Section - 5

GVF Output Data

Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	5.83	ft
Critical Depth	4.97	ft
Channel Slope	0.00348	ft/ft
Critical Slope	0.01112	ft/ft

Messages

Notes

Slope = 10/2873.59 = 0.00348

Area A (Node 101) 100-Year Peak Flow = 68,093 CFC ~ 68,100 CFS

Retention Basin
Infiltration Test Results

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Riverside County / Environmental

40880-R County Center Drive
Temecula, CA 92591
T: 951.600.9271 F: 951.719.1499

PETRA

past + present + future
it's in our science

Engineers, Geologists
Environmental Scientists

December 5, 2012
J.N. 332-12

REGENERATE POWER
c/o Cameron Bucher
ZGlobal Engineering
604 Sutter Street, Suite 250
Folsom, CA 95630

Subject: Infiltration Test Results, Seville Solar Site, Ocotillo Wells Area of Imperial County, California

Dear Mr. Bucher,

Petra Geotechnical, Inc. (Petra) has completed infiltration rate testing of the Seville solar facility site near Ocotillo Wells in Imperial County. Testing was conducted using a dual-ring infiltrometer in accordance with ASTM Test Method D3385-09 at locations depicted in the Infiltration Test Location Map, Figure 1.

The tests were conducted at a depth of approximately 1 foot below existing grade. Soils encountered in tests DRI-1 and DRI-2, which were located along the center and eastern end of the southern project boundary respectively, were predominantly silt/clay, which proved to be relatively impermeable. The third test, DRI-3, conducted near the center of the eastern project boundary, encountered more permeable silty sand. Details of the individual test results are attached. The test results and their approximate locations are summarized in the following table:

Infiltration Test Results

Test No.	Approximate Test Location	Infiltration Rate
DRI-1	Center of southern project boundary	0.04 in/hr
DRI-2	East end of southern project boundary	0.00 in/hr
DRI-3	Center of eastern project boundary	2.28 in/hr

This opportunity to be of service is sincerely appreciated. If you have any questions, please contact this office.

Respectfully submitted,

PETRA GEOTECHNICAL, INC.

Grayson R. Walker, GE
Principal Engineer
GE 871

REGISTERED PROFESSIONAL ENGINEER
GRAYSON R. WALKER
GE 871
GEOTECHNICAL
STATE OF CALIFORNIA

Attachments: Infiltration Test Location Map, Figure 1
Infiltration Test Results

Distribution: (1) Addressee (electronic)
(1) AEI-CASC (electronic)
Attention: David Cooke

PETRA GEOTECHNICAL, INC.
40880 County Center Drive, Suite R
Temecula, California 92591
PHONE: (951) 600-9271
COSTA MESA TEMECULA PALM DESERT SAN DIEGO SANTA CLARITA

Infiltration Test Location Map

Proposed Seville Solar Site Project
Allegretti Farms Site
Ocotillo Wells Area, Imperial County, California

DATE: Dec. 2012	J.N.: 332-12	Figure 1
DWG BY: DLJ	SCALE: NTS	

Seville Site
JN 332-12
Tested by L. Holmes, November 14, 2012

Infiltration Test DRI-1

Double-Ring Infiltrometer Test, ASTM D3385-09

Time (m)	Δ Time (m)	Elapsed Time (m)	Inner Ring (cm)	Volume Inner (cm^3)	Rate Inner (cm/h)
9:15	60		58.8	901.2	1.30
10:15		60	53.7		
10:15	60		58.2	583.2	0.84
11:15		120	54.9		
11:15	60		59.4	106.0	0.15
12:15		180	58.8		
12:15	60		59.4	70.7	0.10
1:15		240	59.0		
				0.0	#DIV/0!
				0.0	#DIV/0!
				0.0	#DIV/0!
				0.0	#DIV/0!
				0.0	#DIV/0!
				0.0	#DIV/0!
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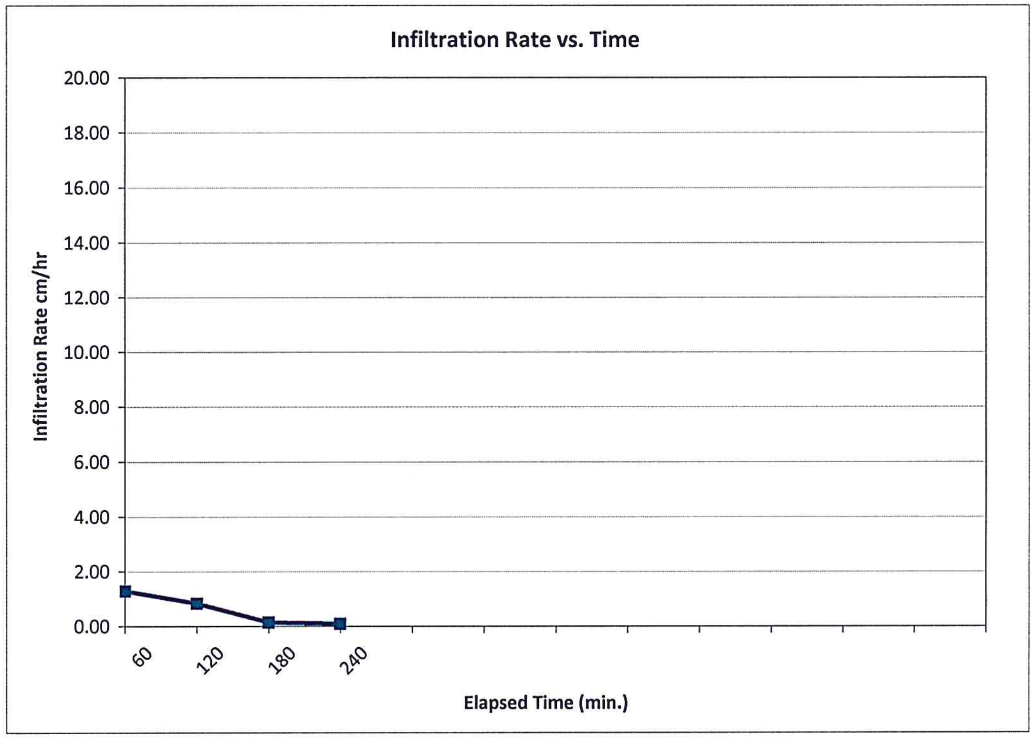
				0.0	#DIV/0!
				0.0	#DIV/0!



Area of Inner Ring (cm^2)
692.8

Area of Annular Space (cm^2)
2120.5

Infiltration Rate	**0.10**	**cm/hr**
	2.8E-05	**cm/sec**
	0.04	**in/hr**
	0.6	**gal/day/ft^2**

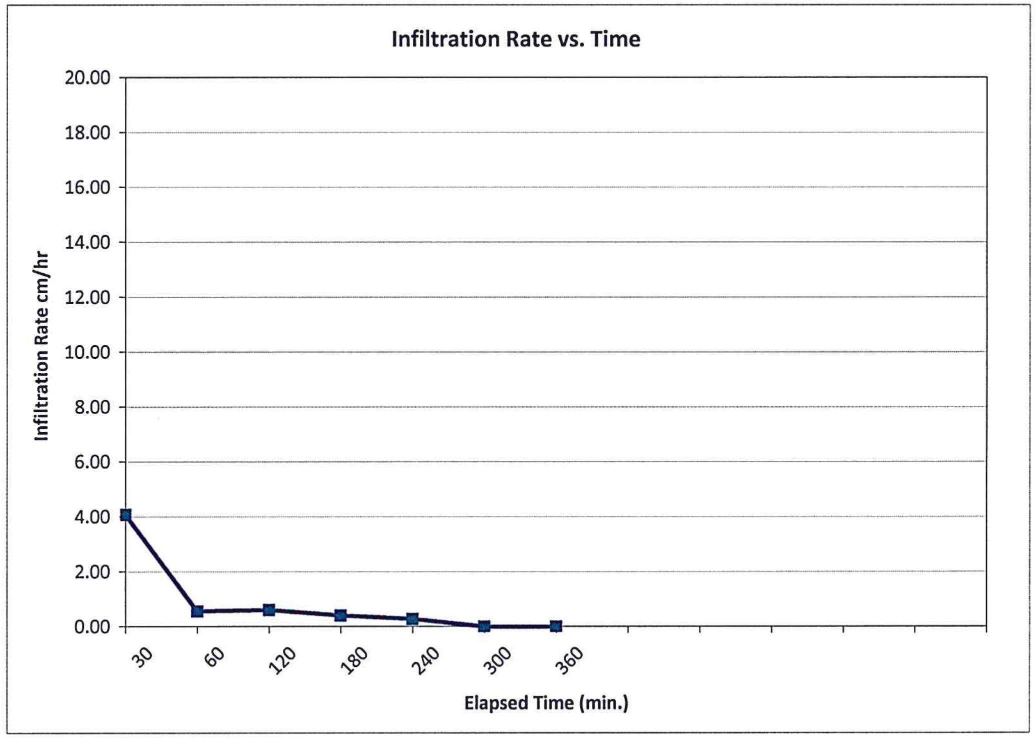
PETRA GEOTECHNICAL, INC.

Seville Site
JN 332-12
Tested by L. Holmes, November 15, 2012

Infiltration Test DRI-2

Double-Ring Infiltrometer Test, ASTM D3385-09

Time (m)	Δ Time (m)	Elapsed Time (m)	Inner Ring (cm)	Volume Inner (cm^3)	Rate Inner (cm/h)
9:25	30		58.9	1413.7	4.08
9:55		30	50.9		
9:55	30		59.0	194.4	0.56
10:25		60	57.9		
10:25	60		58.9	424.1	0.61
11:25		120	56.5		
11:25	60		58.8	282.7	0.41
12:25		180	57.2		
12:25	60		58.9	194.4	0.28
1:25		240	57.8		
1:25	60		58.8	0.0	0.00
2:25		300	58.8		
2:25	60		59.0	0.0	0.00
3:25		360	59.0		
				0.0	#DIV/0!
				0.0	#DIV/0!
				0.0	#DIV/0!
				0.0	#DIV/0!
				0.0	#DIV/0!
				0.0	#DIV/0!



Area of Inner Ring (cm^2)
692.8

Area of Annular Space (cm^2)
2120.5

Infiltration Rate	**0.00**	**cm/hr**
	0.0E+00	**cm/sec**
	0.00	**in/hr**
	0.0	**gal/day/ft^2**

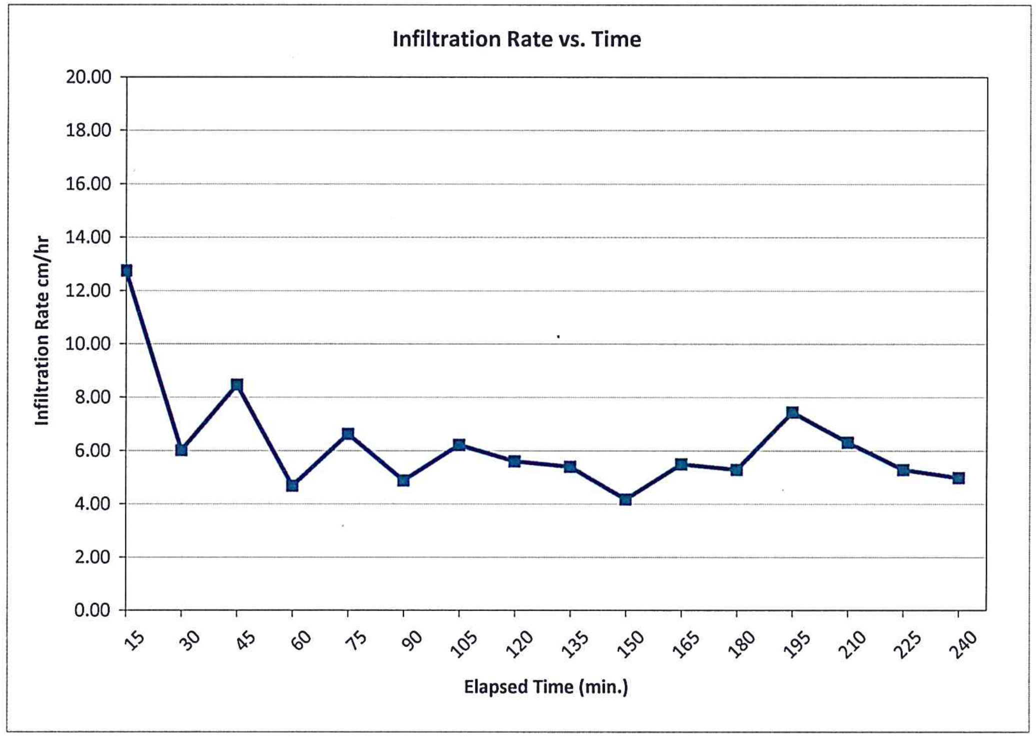
PETRA GEOTECHNICAL, INC.

Seville Site
JN 332-12
Tested by L. Holmes, November 16, 2012

Infiltration Test DRI-3

Double-Ring Infiltrometer Test, ASTM D3385-09

Time (m)	Δ Time (m)	Elapsed Time (m)	Inner Ring (cm)	Volume Inner (cm^3)	Rate Inner (cm/h)
8:55	15		58.7	2208.9	12.75
9:10		15	46.2		
9:10	15		59.3	1042.6	6.02
9:25		30	53.4		
9:25	15		59.3	1466.7	8.47
9:40		45	51.0		
9:40	15		58.7	812.9	4.69
9:55		60	54.1		
9:55	15		58.7	1148.6	6.63
10:10		75	52.2		
10:10	15		59.3	848.2	4.90
10:25		90	54.5		
10:25	15		59.3	1078.0	6.22
10:40		105	53.2		
10:40	15		58.9	971.9	5.61
10:55		120	53.4		
10:55	15		58.5	936.6	5.41
11:10		135	53.2		
11:10	15		58.7	724.5	4.18
11:25		150	54.6		
11:25	15		59.3	954.3	5.51
11:40		165	53.9		
11:40	15		59.3	918.9	5.31
11:55		180	54.1		
11:55	15		59.3	1290.0	7.45
12:10		195	52.0		
12:10	15		59	1095.6	6.33
12:25		210	52.8		
12:25	15		58.8	918.9	5.31
12:40		225	53.6		
12:40	15		59.3	865.9	5.00
12:55		240	54.4		



Area of Inner Ring (cm^2)
692.8

Area of Annular Space (cm^2)
2120.5

Infiltration Rate	**5.80**	**cm/hr**
	1.6E-03	**cm/sec**
	2.28	**in/hr**
	34.2	**gal/day/ft^2**

PETRA GEOTECHNICAL, INC.

Subject: preliminary basin design infiltration rate - Seville Solar Site
From: Grayson Walker <gwalker@petra-inc.com>
Date: 2/14/2013 11:28 AM
To: "'Cameron Bucher'" <cameron@zglobal.biz>
CC: "'David Cooke'" <dcooke@aei-casc.com>, "Grayson Walker" <gwalker@petra-inc.com>

Cameron,

I[1]ve spoken to David Cooke about what would be an appropriate infiltration rate for preliminary sizing of the detention basins at the Seville site. While the infiltration tests yielded rates ranging from 0.00 to 2.28 inches per hour (see attached report), the tests were conducted at shallow, near surface depths. The soils on the site are predominately sandy with isolated layers of finer-grained silts and clays. I suggest using the 2.28 in/hr (to be factored per regulatory requirements) for preliminary basin sizing throughout the site with the understanding that there may be some remedial grading associated with the basin construction to remove exposed silts/clays and replace them with the sandy on-site soils. No import of select materials should be needed.

Please let me know if you have any questions.

Regards,
Grayson

Grayson R. Walker, GE
Vice President
Principal Engineer

PETRA GEOTECHNICAL, INC.
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dir 951.253.4451
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fax 951.719.1499
gwalker@petra-inc.com

Attachments:

332-12 Infiltration Test Results.pdf — 2.8 MB

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PRELIMINARY GEOTECHNICAL INVESTIGATION

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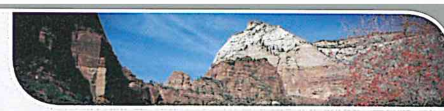

PETRA

PRELIMINARY GEOTECHNICAL INVESTIGATION, PROPOSED SEVILLE SOLAR ENERGY FACILITY, ALLEGRETII FARMS SITE, LOCATED EAST OF OCOTILLO WELLS AND SOUTH OF SR-78, IMPERIAL COUNTY, CALIFORNIA

REGENERATE POWER, LLC

DECEMBER 27, 2012
J.N. 332-12

Riverside County / Environmental

40880-R County Center Drive
Temecula, CA 92591
T: 951.600.9271 F: 951.719.1499

December 27, 2012
J.N. 332-12

REGENERATE POWER, LLC
c/o Mr. Cameron Bucher
ZGLOBAL Engineering, Inc.
604 Sutter Street, Suite 250
Folsom, California 95630

Subject: Preliminary Geotechnical Investigation, Proposed Seville Solar Energy Facility, Allegretii Farms Site, Located East of Ocotillo Wells and South of SR-78, Imperial County, California

Dear Mr. Bucher:

Petra Geotechnical, Inc. (Petra) is pleased to submit herewith our preliminary geotechnical investigation report for the proposed Seville solar energy facility project located approximately 0.4 miles south of SR-78 and approximately 7 miles east of Ocotillo Wells in Imperial County, California. This work was performed in accordance with the scope of work outlined in our Proposal number 1249-12 dated October 24, 2012. This report presents the results of our field exploration and our engineering judgment, opinions, conclusions and recommendations pertaining to geotechnical design aspects for the proposed development.

It has been a pleasure to be of service to you on this project. Should you have questions regarding the contents of this report or should you require additional information, please contact this office.

Respectfully submitted,

PETRA GEOTECHNICAL, INC.

Grayson R. Walker, GE
Vice President

CB/DLJ/GRW/jma
Distribution: (4) Addressee
(1) Mr. David Cooke, AEI-CASC

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PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED SEVILLE SOLAR ENERGY FACILITY
LOCATED SOUTH OF SR-78 AND EAST OF OCOTILLO WELLS
IMPERIAL COUNTY, CALIFORNIA

This report presents the results of Petra Geotechnical, Inc.'s (Petra) preliminary geotechnical investigation for the proposed solar energy facility within the existing Allegretii Farms property located south of SR-78 and east of the Ocotillo Wells area in Imperial County, California. This investigation included a site reconnaissance and subsurface exploration, as well as a review of published and unpublished literature and geotechnical maps pertaining to geologic hazards which may have an impact on the proposed construction.

Purpose and Scope of Services

The purposes of this study were to obtain preliminary information on the subsurface geologic and soil conditions within the project area, evaluate the field and laboratory data and provide conclusions and preliminary recommendations for design and construction of the proposed site improvements as influenced by the subsurface conditions.

The scope of our evaluation consisted of the following.

- Reconnaissance of the site to evaluate existing conditions.
- Review of available published and unpublished geologic data, maps, available online aerial imagery and geotechnical reports concerning geologic and soil conditions within and adjacent to the site which could have an impact on the proposed improvements.
- Excavate twenty exploratory borings, utilizing a hollow-stem auger drill rig, to evaluate the stratigraphy of the subsurface soils and collect representative undisturbed and bulk samples for laboratory testing.
- Advance six cone penetration test (CPT) soundings to evaluate the subsurface soil stratigraphic prolife for a preliminary liquefaction and dynamic settlement analysis.
- Log and visually classify soil materials encountered in the hollow-stem auger borings in accordance with the Unified Soil Classification System.
- Conduct three shallow double-ring infiltrometer tests to evaluate infiltration rate of the near-surface onsite soils for preliminary design of onsite detention basins.
- Conduct appropriate laboratory testing of representative samples (bulk and undisturbed) obtained from the hollow-stem auger borings to determine their engineering properties.
- Perform appropriate engineering and geologic analysis of the data with respect to the proposed improvements.

- Preparation of this report, including pertinent figures and appendices, presenting the results of our evaluation and recommendations for the proposed improvements in general conformance with the requirements of the 2010 California Building Code (CBC), as well as in accordance with applicable local jurisdictional requirements.

Location and Site Description

The subject site is an approximately 1,500-acre, roughly square in shape, property consisting of Section 23 and portions of Sections 22, 26 and 27 of Township 12S and Range 9E in Imperial County, California. The site lies approximately 0.4 miles south of Highway 78 (SR-78), approximately 7 miles east of Ocotillo Wells and approximately 9 miles west of Highway 86. The site is also located about 14 miles from the southern tip of the Salton Sea, 4 miles east of the Imperial and San Diego County line, ½ of a mile east of Cahuilla Trail and ½ of a mile west of Pole Line Road. The general location of the site is shown on Figure 1. The site is situated between Tarantula Wash and San Felipe Creek and the general area surrounding the site is comprised of essentially vacant land consisting of sand dunes and local washes. Access to the site is via a dirt driveway extending south from Highway 78.

The site is essentially comprised of flat-lying, very low gradient agricultural fields that are separated by either dirt access roads or rows of mature windbreaker trees. San Felipe Creek, in its natural state, previously flowed through the southern third of the property in a southeasterly direction; however, the creek has been subsequently diverted at the western property boundary and now flows south. The former creek bottom has been in-filled to create a near level surface for the utilization of the site as agricultural fields. A notable earth berm and man-made drainage channel associated with diversion of San Felipe Creek has been established along the western property boundary. It appears that all agricultural activities had been suspended within the last few years with the only exception being a small area in the extreme southeast corner of the site which contained grain crops at the time of our field exploration. Barbed wire fencing was typically observed along the property lines.

The site exhibits a generally planar and flat-lying topography with an overall inclination to the southeast at an estimated average low gradient of 0.4 percent. The planar topographic feature is generally attributed to previous agricultural activities that included in-filling of the former creek bottom of San Felipe Creek and possible in-filling of other low ground depressions and/or other shallow drainages that previously existed within the site. Site elevations range from a high of approximately 5 feet below mean sea level (msl) at the northwest property corner to a low of approximately 40 feet below msl at the southeast corner. Vegetation within the site is generally limited to the several rows of windbreaker trees, the grain crop located at the southeast corner, and sporadic weeds.

Proposed Construction and Grading

No conceptual development plans are available at this time. However, proposed improvements at the site may consist of numerous panel blocks with photovoltaic (PV) solar arrays, various inverter transformer stations, numerous underground cable raceways, a substation, detention basins for controlling storm water runoff, maintenance access roads, and maintenance buildings. Preliminary designs for the transformers and substation are not known at this time; however the solar arrays are anticipated to be constructed on a shallow pier type foundation.

Grading of the site is also unknown at this time; however, due to the relatively flat-flying topography, site grading is expected to entail minor cuts and fills to provide access roads, site drainage, and building sites for appurtenant structures.

Assumed Foundation Loading Conditions

It is anticipated that the proposed solar panel arrays will be mounted on multi-panel tracker tables mounted at a minimum height of 6 feet above the ground and that a wind load of approximately 20 kips (ultimate) will be applied to each table horizontally. We utilized these assumptions in formulating trial pile and foundation sizes for analysis.

For the proposed inverters, we have assumed that the inverters will impose a load on the order of about 5 tons. For the proposed substation, we have assumed that switchgear and other equipment would range from about 5 to 20 tons. We have also assumed that there will be a large transformer for connection to the transmission lines, and that this transformer will be on the order of 250 to 300 tons. For any proposed control buildings and/or maintenance buildings, we have assumed that the buildings will be of relatively light weight construction.

Literature Review

Petra researched and reviewed available published and unpublished geologic data, maps and aerial imagery pertaining to regional geology, faulting and geologic hazards that may affect the site. The results of this review are discussed under Findings presented in a following section of this report.

Subsurface Exploration

A subsurface exploration program was performed under the direction of an engineering geologist from Petra between November 13 and 16, 2012. The exploration involved the excavation of twenty exploratory borings (B-1 through B-20) to a maximum depth of approximately 51.5 feet below existing grades, utilizing a truck-mounted CME 75 drill rig equipped with 8-inch diameter hollow-stem augers. Earth materials encountered within the exploratory borings were classified and logged by an engineering geologist in accordance with the visual-manual procedures of the Unified Soil Classification System, ASTM Test Standard D2488. In addition, six cone penetration test soundings (CPT-1 through CPT-6) were performed within the site to a maximum depth of approximately 60 feet below existing grades utilizing a 30-ton CPT rig. The CPT soundings were performed by Middle Earth Geo Testing on November 13, 2012 in accordance with ASTM Test Standard D5778. The approximate locations of the exploratory borings and CPT soundings are shown on Figure 2. The logs for the borings are presented in Appendix A and the CPT test data is presented in Appendix C.

Disturbed bulk samples and relatively undisturbed ring samples of in-situ soil materials were collected from the exploratory borings for classification, laboratory testing and engineering analyses. Undisturbed samples were obtained using a 3-inch outside diameter modified California split-spoon soil sampler lined with brass rings. The soil sampler was driven with successive 30-inch drops of a free-fall, 140-pound automatic trip hammer. The central portions of the driven-core samples were placed in sealed containers and transported to our laboratory for testing. The number of blows required to drive the split-spoon sampler 18 inches into the soil were recorded for each 6-inch driving increment; however, the number of blows required to drive the sampler for the final 12 inches was noted in the boring logs as *Blows per Foot*.

Double-Ring Infiltrometer Tests

Three double-ring infiltometer tests were conducted at shallow depths to determine infiltration rates of the near-surface onsite soils for preliminary design of detention basins to manage stormwater runoff. These tests were performed in accordance with ASTM. Test Standard D3385. The test results are presented in a separate report by Petra (Petra, 2012).

Laboratory Testing

The laboratory testing program included the determination of in-situ dry density and moisture content, maximum dry density and optimum moisture content, expansion index, Atterberg Limits, consolidation potential, direct shear strength, and preliminary soil corrosivity screening (soluble sulfate and chloride content, pH and minimum resistivity). A description of laboratory test methods and summaries of the laboratory test data are presented in Appendix B and the in-situ dry density and moisture content results are presented on the boring logs (Appendix A).

FINDINGS

Regional Geologic Setting

The proposed solar energy facility is located near the eastern boundary of the Imperial Valley, which is part of the Salton Trough geomorphic province of California. The Salton Trough encompasses the Coachella, Imperial and Mexicali Valleys, which extend from northeast of Palm Springs near San Gorgonio Pass to the Gulf of California. The geologic structure of the trough is a result of extensional forces within the earth's crust. The Imperial Valley is generally bounded by the Chocolate Mountains to the east, the Salton Sea to the north, the Peninsular Ranges to the west, and Mexicali Valley to the south. Lacustrine and alluvial sediments are the dominant geologic units of the Imperial Valley. Unexposed succession of Tertiary- and Quaternary-aged sedimentary rocks lies below the alluvial and lake sediments ranging in depth from 11,000 feet or more at the margins to more than 20,000 feet in the central portion of the Salton Trough. Basement rocks consisting of Mesozoic granite and probably Paleozoic metamorphic rocks are estimated to exist at depths between 15,000 and 20,000 feet.

The watershed of the Salton Trough empties into the Salton Sea at the lowest part of the basin. This basin was periodically filled with water to form the ancient Lake Cahuilla, depending on which side of its delta the Colorado River would drain. The sediments of the delta form a topographic high that separates the Salton basin, which is below sea level, from the Gulf of California. More specifically, the site lies near the western boundary of the old meandering shoreline of ancient Lake Cahuilla and approximately 14 miles from the southern tip of the present-day Salton Sea. The current level of the Salton Sea is about 226 feet below msl.

Local Geology and Subsurface Soil Conditions

The site is underlain by alluvial and eolian deposits consisting of interbedded clean sands, silty sands, silts and sandy silts. A thin, isolated layer of plastic silty clay, 1 to 2 feet thick, was encountered in eight of the exploratory borings. The top of the thin clay layer was encountered at depths varying from about 3 to 11 feet below grade. The clean sands are prevalent in the upper 20 to 25 feet over the northerly and northeasterly portions of the site while the finer-grained silty sands, silts and sandy silts are more prevalent in the upper 10 to 25 feet over the southerly and southwesterly portions of the site. The alluvial soils were generally found to be loose in the upper approximately 2 feet, medium dense at a depth interval of approximately 2 to 5 feet, and dense to very dense to the depths explored.

Undocumented artificial fill of undetermined depth associated with the in-filling of San Felipe Creek exists within the southern portion of the site along the previous alignment of San Felipe Creek. Based on a review of old topographic maps, the estimated location of the previous alignment of San Felipe Creek is shown on Figure 2. Minor amounts of shallow undocumented artificial fill also exist in several areas along the existing dirt access roads. In addition, a tilled horizon related to farming activities exists within the agricultural fields, and the depth of the tilled surface is estimated to be approximately 2 feet.

Groundwater

Free groundwater was not encountered within the exploratory borings; however, perched groundwater was encountered in B-2 at a depth of approximately 43 feet. The regional ground water table is estimated to be approximately 150 feet or greater below the ground surface.

Faulting

The Salton Trough is a seismically active area and in particular within the Imperial Valley with numerous northwest-trending active faults. However, the site is not located within a *Fault Hazard Zone*, as defined by the state of California in the Alquist-Priolo Earthquake Fault Zoning Act, and no faults are known to project through the project site. The closest active faults in proximity to the site include: the Coyote Creek fault, approximately 1 mile to the southwest; the Borrego Mountain fault, approximately 5 miles to the northwest; the Superstition Hills fault, approximately 6.1 miles to the southeast; and the Elmore Ranch fault, approximately 10 miles to the southeast. The Coyote Creek and Borrego Mountain faults are segments of the San Jacinto Fault Zone (SJFZ) and the Superstition Hills fault is also believed to be a segment of the SJFZ. An "active" fault is defined as a fault that has had displacement within the Holocene epoch, or last 11,000 years. A "potentially active" fault is a fault that does not have evidence of movement within the last 11,000 years, but has moved within the last 1.6 million years.

CONCLUSIONS AND RECOMMENDATIONS

General

From a geotechnical engineering and engineering geologic point of view, the subject property is considered suitable for the proposed development provided the following conclusions and recommendations are incorporated into the design criteria and project specifications.

GEOLOGIC CONSIDERATIONS

Groundwater

Adverse effects on the proposed construction due to shallow groundwater are not anticipated.

Fault Rupture

The site is not located within a currently designated State of California Alquist-Priolo Earthquake Fault Zone (Hart, 1999). In addition, no known active faults have been identified on the site. While fault rupture would most likely occur along established fault traces, fault rupture could occur at other locations. However, the potential for active fault rupture at the site is considered to be very low.

Seismic Shaking

The site is located within an active tectonic area with several significant faults capable of producing moderate to strong earthquakes. The Coyote Creek fault, the Borrego Mountain fault, the Superstition Hills fault and the Elmore Ranch fault are all in close proximity of the site and capable of producing strong ground motions.

Historically, the Imperial fault generated the 1979 and 1940 earthquakes and the Elmore Ranch fault generated the November 23, 1987 earthquake that is thought to have triggered the November 24, 1987 earthquake that occurred on the Superstition Hills and Wienert faults. Table 1 lists select recorded earthquakes felt at the site area.

TABLE 1
Significant Historic Earthquakes

Earthquake Events	Moment Magnitude (Mw)
El Mayor/Cucapah Mexicali (April 4, 2010)	7.2
Superstition Hills (Nov. 24, 1987)	6.6
Elmore Ranch (Nov. 23, 1987)	6.2
Mexicali (June 9, 1980)	6.1
Imperial Valley (Oct. 15, 1979)	6.4
Borrego Mountain (April 8, 1968)	6.5
Imperial Valley (May 18, 1940)	6.9
Laguna Salada (Feb. 23 1892)	7.0

Secondary Effects of Seismic Activity

Secondary effects of seismic activity normally considered as possible hazards to a site include several types of ground failure, as well as earthquake-induced flooding. Various general types of ground failures, which might occur as a consequence of severe ground shaking at the site, include ground subsidence, ground lurching and lateral spreading. The probability of occurrence of each type of ground failure depends on the severity of the earthquake, distance from faults, topography, subsoil and groundwater conditions, in addition to other factors. Based on the site conditions and relatively flat topography, ground subsidence, ground lurching and lateral spreading is considered unlikely at the site.

Seismically induced flooding that might be considered a potential hazard to a site normally includes flooding due to tsunami or seiche (i.e., a wave-like oscillation of the surface of water in an enclosed basin that may be initiated by a strong earthquake) or failure of a major reservoir or retention structure upstream of the site. The Salton Sea is situated approximately 14 miles from the site with an elevation differential greater than approximately 180 feet. In addition, no major reservoir is located near, or upstream of the site. Therefore, the potential for seiche or inundation is considered negligible. Because of the inland location of the site, flooding due to a tsunami is also considered negligible at the site.

Landslides and Slope Instability

The site exhibits a generally flat topography and no landslides exist within or near the site. Based on the topography across the site, the potential for landsliding is considered negligible.

Flooding

Storm water is periodically channeled within the adjacent San Felipe Creek and Tarantula Wash, located near the westerly and easterly boundaries of the site, respectively. As such, there may be a potential for local sheet flooding to occur at the site. Therefore, a detailed drainage study should be performed by the project civil engineer.

Expansive Soils

The expansion potential of the surface and subsurface soils across the site vary from very low to low. Soils exhibiting a low expansion potential can affect the performance of concrete slabs or structures with shallow foundations. Soils exhibiting a low expansion potential are present in the upper 10 to 25 feet over the southerly and southwesterly portions of the site. Recommendations to mitigate the potential effects of expansive soils are provided in the *Preliminary Foundation Recommendations* section of port.

Areal Subsidence

The site is not known to be located in an area with a potential for ground subsidence due to withdrawal of fluids such as groundwater or oil.

Ground Motion Analysis

Since the site is located within a seismically active area, the potential exists for ground motion to affect future improvements. Petra has thus assessed free-field horizontal ground acceleration (PGA) using currently accepted methodology as mandated by the CBC (2010) and Special Publication 117A (CGS 2008). Current standards of practice and regulatory agencies dictate such assessments - for example, the State of California Seismic Hazards Mapping Act (SHMA) of 1990 (Division 2, Chapter 7.8, Public Resources Code).

Selection of the appropriate design seismic parameters depends upon the kinds of geotechnical or structural analyses (for example, static or dynamic), the kind and sensitivity (for example, schools, hospitals, essential services facilities vs. normal-risk) of proposed structures, and the level of "acceptable risk" deemed suitable for the project. Normal-risk structures usually include those where the CBC (ICBO, 2010) concern is primarily life and safety during an earthquake. Accordingly, the required estimation of ground motion is the Design-Basis Earthquake (**DBE**) that has a 10-percent chance of being exceeded in 50-years.

Probabilistic Analysis

A probabilistic analysis incorporates uncertainties in time, recurrence intervals, size, and location (along faults) of hypothetical earthquakes. This method therefore accounts for the likelihood (rather than certainty) of occurrence and provides levels of ground acceleration that might be more reasonably hypothesized for a finite exposure period. The DBE ground-motion with a recurrence interval of about 475 years is used (10 percent chance of being exceeded in 50 years).

Probabilistic ground motions for the site can be computed by use of computer programs from the USGS 2008 PSHA Interactive Deaggregation web site and the "Design Maps" web application. Motions with a 10-percent probability of being exceeded in 50-years (475-years return period), 100 years (975-year return period), and 2-percent probability in 50-years (2475-years return period) were computed. Normal risk structures typically include single-family and multiple family residences, and commercial buildings. More critical structures include schools, and other important facilities that would fall within occupancy category III such as power generation facilities. These more critical facilities include an importance factor in the structural engineer's use of the ground motion values.

It should be noted that classification of the project site as Site Class D (according to Table 20.3-1 of ASCE 7-05) is consistent with the blow counts measured in the exploratory borings and inferred in the CPT soundings.

Outputs from the modeling are provided in Appendix E. The ground motion modeling included deaggreagation of the fault magnitude and acceleration pairing. The magnitude and acceleration were scaled within the liquefaction analysis programs to appropriate weighting equal to a scale magnitude of 7.5. The Following Table summarizes the results of the expected peak ground acceleration and magnitude that we have concluded are probable for the site for the DBE. Spectral acceleration values for the structural engineering are discussed in a later section and are based on similar computational methodologies.

Maximum Credible Magnitude	Peak Horizontal Ground Acceleration
6.62	0.511g

Liquefaction and Seismically-Induced Settlement

Assessment of liquefaction potential for a particular site requires knowledge of a number of regional as well as site-specific parameters, including the estimated design earthquake magnitude, the distance to the assumed causative fault and the associated probable peak horizontal ground acceleration at the site, subsurface stratigraphy and soil characteristics. Parameters such as distance to causative faults and estimated probable peak horizontal ground acceleration can readily be determined using published references, or by utilizing a commercially available computer program specifically designed to perform a probabilistic analysis. On the other hand, stratigraphy and soil characteristics can only be accurately determined by means of a site-specific subsurface investigation combined with appropriate laboratory analysis of representative samples of onsite soils.

Liquefaction occurs when dynamic loading of a saturated sand or silt causes pore-water pressures to increase to levels where grain-to-grain contact is lost and material temporarily behaves as a viscous fluid. Liquefaction can cause settlement of the ground surface, settlement and tilting of engineered structures, flotation of buoyant buried structures and fissuring of the ground surface. A common manifestation of liquefaction is the formation of sand boils – short-lived fountains of soil and water that emerge from fissures or vents and leave freshly deposited conical mounds of sand or silt on the ground surface.

The compressional vector forces of the earthquake waves induces compressional stresses and strains in the soil during strong ground shaking. The process causes the sandy deposit to rearrange the grain structure so that there is an increase in density, thus decrease in volume which leads to vertical settlements. Dynamic settlement has been well documented in wet sandy deposits undergoing liquefaction (see Tokimatsu and Seed, 1987) and in relatively dry sediments as well (Stewart et al, 1996). Specific methods to analyze potential wet and dry dynamic settlement are reported in Tokimatsu and Seed (1987) and dry settlement in Pradel (1998) and Stewart et al. (2001; 2002) respectively. Most of the referenced papers study the seismic effects on dry clean sands of a uniform size, though several reports extend the literature to fine grained soils (Stewart et al., 2001 & 2002). State guidelines for evaluating dynamic settlement are provided in the California Geological Survey Special Publication 117A (CGS, 2008).

Liquefaction Analyses Using CPT Results

A variety of computer programs are available for liquefaction and seismically-induced settlement analyses. For our liquefaction study, we utilized the procedure by Dr. Moss (Moss 2006 – Cliq 2011) developed from the methods originally recommended by Seed and Idriss (1982) using the computer program Cliq (Version 1.5.1.16, Geologismiki, 2011) to best determine the likelihood and ramifications of liquefaction at this site.

Our analyses were performed solely using CPT data due to the fact that the CPT provides *continuous* penetration resistance data rather than borehole data that must be averaged over discrete sampling increments (e.g., 5 or 10 feet). In our analyses, we utilized a PGA of 0.511g, a moment magnitude Mw of 6.62 and a conservative, assumed groundwater depth of 33 feet. Based on our analyses, **no** liquefiable soil layers were identified at the six CPT sounding locations. Furthermore, seismically-induced (dynamic) settlements were determined to be on the order of 0.02 to 0.04 inches. The results of our analyses for all six CPT soundings are provided in Appendix D.

EARTHWORK

General Earthwork Recommendations

Earthwork should be performed in accordance with the applicable provisions of the 2010 CBC. Grading should also be performed in accordance with the following site-specific recommendations prepared by Petra based on the proposed construction.

Geotechnical Observations and Testing

Prior to the start of earthwork, a meeting should be held at the site with the owner, contractor and geotechnical consultant to discuss the work schedule and geotechnical aspects of the grading. Earthwork, which in this instance will generally entail overexcavation and re-compaction of low density near surface soils for structures supported by mat or shallow foundations, should be accomplished under full-time observation and testing of the geotechnical consultant. Grading and re-compaction of the near surface soils along access roads and in areas to be graded to a sheet flow condition should be accomplished under part-time observation and testing of the geotechnical consultant. A representative of the project geotechnical consultant should be present onsite during earthwork operations to document proper placement and adequate compaction of fills, as well as to document compliance with the other recommendations presented herein.

Clearing and Grubbing

All vegetation and any trash or debris in areas to be graded should be removed from the site. During site grading, fill soils should be cleared of any deleterious materials that are missed during the initial clearing and grubbing operations. Any cavities or excavations created upon removal of subsurface structures should be cleared of loose soil, shaped to provide access for backfilling and compaction equipment and then backfilled with properly compacted fill.

The project geotechnical consultant should provide periodic observation and testing services during clearing and grubbing operations to document compliance with the above recommendations. In addition, should any unusual or adverse soil conditions be encountered during grading that are not described herein, these conditions should be brought to the immediate attention of the project geotechnical consultant for corrective recommendations.

Ground Preparation – Foundation Areas

Based on soil conditions observed in the exploratory borings and data from the CPT soundings, surface soils over a majority of the site are loose to medium dense in the upper approximately 2 to 3 feet but locally increase to depths of approximately 5 to 6 feet. In addition, the depth of undocumented in-fill soils placed within the previous alignment of San Filipe Creek is unknown but conceivably could be on the order of 10 feet or more. In areas where structures are to be supported by conventional shallow slab-on-grade foundations, spread footings and/or mat foundations, the existing ground should be over-excavated to depths that expose competent native soils exhibiting an in-place relative compaction of 90 percent or more, based on Test Method ASTM D1557. As noted above, the required depths of over-excavation are anticipated to vary from approximately 2 to 6 feet with deeper removals along the previous alignment of San Filipe Creek. The horizontal limits of over-excavation should extend to a minimum distance of 5 feet beyond the proposed perimeter foundation lines or to a horizontal distance equal to the depth of over-excavation, whichever is greater.

Due to the variability of the surficial soil conditions, the required depths of over-excavation will have to be determined during grading on a case-by-case basis. Therefore, prior to placing compacted fill, the exposed bottom surfaces in all over-excavated areas should be observed and approved by the project geotechnical consultant. Following this approval, the exposed bottom surfaces should be scarified to a depth of approximately 6 inches, watered or air-dried as necessary to achieve a moisture content that is equal to or slightly above optimum moisture content, and then compacted in-place to a minimum relative compaction of 90 percent.

In areas where tracker table pole supports are founded on spread footings excavated directly into native ground, the exposed bottom surface should be observed and approved by the geotechnical consultant to assure that all loose or unsuitable soils are removed prior to concrete placement. The deeper native soils over a majority of the site are anticipated to be suitable to support the spread footings provided the footing is founded no less than approximately 3 feet below existing grade.

Ground Preparation - Parking Lots, Access Roads and Sheet-Graded Areas

The existing ground in proposed parking lot and access road areas to be paved with asphaltic concrete should be over-excavated and recompacted in a similar manner as recommended above. In areas where access roads are to be covered with gravel only, and in areas to be graded to a sheet flow condition for drainage purposes and where no structures are planned, the existing ground should be scarified to a depth of 8 to 12 inches, watered or air-dried as necessary to achieve a moisture content that is equal to or slightly above optimum moisture content, and then compacted in-place to a minimum relative compaction of 90 percent.

Cut Areas

Cuts that extend to depths greater than approximately 2 to 3 feet below existing grade are anticipated to expose dense competent native soils. Where these materials are exposed at finish grade in areas of proposed construction no special remedial grading will be required provided the exposed grades are not disturbed as a result of the grading operations.

Fill Placement and Testing

All fill should be placed in lifts not exceeding 6 inches in thickness, watered or air- dried as necessary to achieve a moisture content that is equal to or slightly above optimum moisture content, and then compacted in-place to a minimum relative compaction of 90 percent. Each fill lift should be treated in a similar manner. Subsequent lifts should not be placed until the preceding lift has been approved by the project geotechnical consultant. The laboratory maximum dry density and optimum moisture content for each change in soil type should be determined in accordance with Test Method ASTM D1557.

Geotechnical Observations

The project geotechnical consultant should be present on site during grading operations to observe proper placement and adequate compaction of fill, as well as to document compliance with the other recommendations presented herein.

Shrinkage and Subsidence

Volumetric changes in earth quantities will occur when excavated onsite soils are replaced as properly compacted fill. Accordingly, it is estimated that a shrinkage factor on the order of 15 to 20 percent will occur when onsite soils are excavated and placed as compacted fill.

Subsidence from scarification and recompaction of exposed bottom surfaces in over-excavated areas is expected to be on the order of approximately 0.10 to 0.15 feet.

The above estimates of shrinkage and subsidence are intended as aids for the project planners in determining earthwork quantities. However, these values should not be considered as absolute values and some contingencies should be made for balancing earthwork quantities on the basis of actual shrinkage and subsidence that occur during grading.

SEISMIC DESIGN CONSIDERATIONS

Earthquake Loads

Structures within the site should be designed and constructed to resist the effects of seismic ground motions as provided in Section 1613 of the 2010 California Building Code (CBC). The method of design is dependent on the seismic zoning, site characteristics, occupancy category, building configuration, type of structural system and on the building height.

For structural design in accordance with the 2010 CBC, a computer program, Earthquake Ground Motion Parameters Version 5.1.0, developed by the United States Geological Survey (USGS, 2007) was utilized to provide ground motion parameters for the subject site. The program includes hazard curves, uniform hazard response spectra and design parameters for sites in the 50 United States, Puerto Rico and the United States Virgin Islands. Based on the latitude, longitude and site classification, seismic design parameters and spectral response for both short periods and 1-second periods are calculated including Mapped Spectral Response Acceleration Parameter, Site Coefficient, Adjusted Maximum Considered

Earthquake Spectral Response Acceleration Parameter and Design Spectral Response Acceleration Parameter. The program is based on USGS research and publications in cooperation with the California Geological Survey for evaluation of California faulting and seismicity (USGS, 1996a; 1996b; 2002; 2007).

The following 2010 CBC seismic design coefficients should be used for the proposed structures. These criteria are based on the site class as determined by existing subsurface geologic conditions, on the proximity of the site to the nearby causative fault and on the maximum moment magnitude and slip rate of the nearby fault.

2010 CBC Section 1613, Seismic Design Coefficients	
Mapped Spectral Response Acceleration, S_S, unadjusted for site class (Figure 1613.5(3) for 0.2 second)	1.787
Mapped Spectral Response Acceleration, S_1, unadjusted for site class (Figure 1613.5(4) for 1.0 second)	0.721
Site Class Definition (Table 1613.5.2)	D
Site Coefficient, F_a (Table 1613.5.3 (1) short period)	1.0
Site Coefficient, F_v (Table 1613.5.3 (2) 1-second period)	1.5
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{MS} (Eq. 16-36)	1.787
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{M1} (Eq. 16-37)	1.081
Design Spectral Response Acceleration Parameter, S_{DS} (Eq. 16-38)	1.191
Design Spectral Response Acceleration Parameter, S_{D1} (Eq. 16-39)	0.721

PRELIMINARY FOUNDATION RECOMMENDATIONS

General – Foundation Types

In consideration of the dense to very dense nature of the alluvial soils underlying the site, conventional shallow foundations may be used for support of proposed control and equipment maintenance buildings, and heavy equipment such as inverters, the substation and switchgear, transformer and other heavy equipment.

The pole supporting the solar panel tracker assemblies may be supported on either spread footings or driven pipe piles. Spread footings for solar tracker tables would most likely be cast in excavations dug directly into the native soils. The transient nature of the wind load that would provide the controlling conditions for solar panels would require that the spread footing be designed on the basis of preventing sliding and overturning. Therefore, the footings design would not be settlement-controlled as is the case with most other spread footing situations.

Conventional Foundations

Allowable Bearing Values

An allowable bearing value of 2,000 pounds per square foot (psf) may be used for 24-inch square pad footings and 12-inch wide continuous footings founded at a minimum depth of 18 inches below the lowest adjacent final grade. This value may be increased by 10 percent for each additional foot of width or depth, to a maximum value of 3,000 psf. Recommended allowable bearing values include both dead and live loads and may be increased by one-third when considering short-duration wind, but not seismic forces due to the reduction in soil strength during strong seismic shaking.

For larger or deeper footings the settlements will increase further and reduce the usable bearing pressure. We should examine proposed large footing locations further to better ascertain likely settlements at the specific location and with the specific loads imposed.

Static Settlement

Based on the general settlement characteristics of the in situ alluvial soils and compacted fills comprised of soils that are similar to those that exist on the site, as well as the recommended allowable-bearing value, it is estimated that the total settlement of conventional footings for a static loading condition will be less than approximately 1 inch. Maximum differential settlement is estimated to be about 3/4 inch over a horizontal distance of 40 feet. The anticipated differential settlement may be expressed as an angular distortion of 1:640. It is anticipated that the majority of the settlement would occur during construction or shortly thereafter as foundation loads are applied.

Dynamic Settlement

Liquefaction calculations yielded an estimated earthquake-induced dynamic settlement of approximately 0.02 to 0.04 inches for the onsite alluvial soils. In addition, no liquefiable soil layers were identified in the analyses. Therefore, dynamic settlement can generally be ignored in the design of foundations.

Lateral Resistance

A passive earth pressure of 250 psf per foot of depth, to a maximum value of 2,500 psf pounds, may be used to determine lateral bearing resistance for footings. In addition, a coefficient of friction of 0.30 times the dead load forces may be used between concrete and the supporting soils to determine lateral sliding resistance. The above values may be increased by one-third when designing for transient wind or seismic forces. It should be noted that the above values are based on the condition where footings are cast in direct contact with compacted fill or competent native soils. In cases where the footing sides are formed, all backfill placed against the footings upon removal of forms should be compacted to at least 90 percent of the applicable maximum dry density.

Expansive Soil Conditions

The results of our laboratory tests performed on representative samples of near-surface soils within the site indicate that the soils exhibit expansion potentials that are within the Very Low to Low range (Expansion Index from 0 to 50). As such, the site soils are classified as "expansive" as defined in Section 1803.5.3 of the 2010 CBC. The design of foundations and slabs-on-ground should therefore be performed in accordance with the procedures outlined in Sections 1808.6.1 and 1808.6.2 of the 2010 CBC, respectively.

The design and construction recommendations that follow are based on the above soil conditions and may be considered for reducing the effects of variability in composition and behavior within the site soils and long-term differential settlement. These recommendations have been developed on the basis of the previous experience of this firm on projects with similar soil conditions. Although construction performed in accordance with these recommendations has been found to reduce post-construction movement and/or distress, they generally do not positively eliminate all potential effects of variability in soils characteristics and future settlement.

It should also be noted that the recommendations for reinforcement provided herein are performance-based and intended only as guidelines to achieve adequate performance under the anticipated soil conditions. The project structural engineer, architect and/or civil engineer should make appropriate adjustments to reinforcement type, size and spacing to account for internal concrete forces (e.g., thermal, shrinkage and expansion) as well as external forces (e.g., applied loads) as deemed necessary. Consideration should also be given to minimum design criteria as dictated by local building code requirements.

Conventional Slab-on-Grade Systems

As noted above, onsite soils within the subject site should be considered expansive per Section 1803.5.3 of the 2010 CBC. Section 1808.6.2 of the 2010 CBC specifies that non-prestressed slab-on-grade foundations (floor slabs) constructed on expansive materials should be designed in accordance with the latest edition of the Wire Reinforcement Institute (WRI) publication "Design of Slab-on-Ground Foundations". The design procedures outlined in the WRI publication are based on the weighted plasticity index of the various soil layers existing within the upper 15 feet of the building site.

Based on laboratory testing by our firm, a weighted plasticity index of 10 can be assumed for the subject site. The WRI publication states that the weighted plasticity index of each building site should be modified (multiplied) by correction factors that compensate for the effects of sloping ground and the unconfined compressive strength of the supporting soil or bedrock materials. Since the proposed buildings and structures will be constructed on level building pads, and in consideration of the estimated unconfined compressive strength of the onsite soils, it is recommended that the weighted plasticity index, as provided herein be multiplied by a factor of 1.2 in order to determine the value of the effective plasticity index (per Figure 9 of the WRI publication). In summary, it is recommended that an effective plasticity index of 12 be utilized by the project structural engineer to design slabs-on-ground with an interior grade beam system in accordance with the WRI publication.

Footings

1. Minimum footing widths and depths should be determined by the project structural engineer based on total foundation loads. However, we recommend a minimum footing width of 12 inches and a minimum depth of 18 inches below the lowest adjacent final grade.

2. All continuous footings should be reinforced with a minimum of two No. 4 bars, one top and one bottom.

3. Interior isolated pad footings should be a minimum of 24 inches square and founded at a minimum depth of 12 inches below the bottoms of the adjacent floor slabs. Pad footings should be reinforced with No. 4 bars spaced a maximum of 18 inches on centers, both ways, placed near the bottoms of the footings.

4. Exterior isolated pad footings should be a minimum of 24 inches square, and founded at a minimum depth of 18 inches below the lowest adjacent final grade. The pad footings should be reinforced with No. 4 bars spaced a maximum of 18 inches on centers, both ways, placed near the bottoms of the footings. Exterior isolated pad footings may need to be connected to adjacent pad and/or continuous footings via tie beams at the discretion of the project structural engineer.

5. Spacing and locations of additional interior concrete grade beams that may be required below floor slabs should be determined by the project architect or structural engineer in accordance with the WRI publication.

Building Floor Slabs

1. The thickness and reinforcement for concrete floor slabs should be determined by the project structural engineer based on total loads. However, we recommend a minimum floor slab thickness of 4 inches and reinforcement consisting of No. 3 bars spaced a maximum of 18 inches on centers, both ways. Alternatively, the structural engineer may recommend the use of prefabricated welded wire mesh for slab reinforcement. For this condition, the welded wire mesh should be of sheet type (not rolled) and should consist of 6x6/W2.9xW2.9 (per the Wire Reinforcement Institute [WRI] designation) or stronger. All slab reinforcement should be properly supported to ensure the desired placement near mid-depth. Care should be exercised to prevent warping of the welded wire mesh between the chairs in order to ensure its placement at the desired mid-slab position.

2. Moisture-sensitive area concrete floor slabs should be underlain with a moisture vapor retarder consisting of a minimum 10-mil-thick polyethylene or polyolefin membrane that meets the minimum requirements of ASTM E96 and ASTM E1745 for vapor retarders (such as Husky Orange Guard®, Stego® Wrap, or equivalent). All laps within the membrane should be sealed, and at least 2 inches of clean sand should be placed over the membrane to promote uniform curing of the concrete. To reduce the potential for punctures, the membrane should be placed on a pad surface that has been graded smooth without any sharp protrusions. If a smooth surface cannot be achieved by grading, consideration should be given to lowering the pad finished grade an additional inch and then placing a 1-inch-thick leveling course of sand across the pad surface prior to the placement of the membrane.

   At the present time, some slab designers, geotechnical professionals and concrete experts view the sand layer below the slab (blotting sand) as a place for entrapment of excess moisture that could adversely impact moisture-sensitive floor coverings. As a preventive measure, the potential for moisture intrusion into the concrete slab could be reduced if the concrete is placed directly on the vapor retarder. However, if this sand layer is omitted, appropriate curing methods must be implemented to ensure that the concrete slab cures uniformly. A qualified materials engineer with experience in slab design and construction should provide recommendations for alternative methods of curing and supervise the construction process to ensure uniform slab curing. Additional steps would also need to be taken to prevent puncturing of the vapor retarder during concrete placement.

3. Prior to placing concrete, the subgrade soils below floor slabs should be prewatered to achieve a moisture content that is at least 1.2 times the optimum moisture content. This moisture should penetrate to a depth of approximately 12 inches into the subgrade.

4. The minimum dimensions and reinforcement recommended herein for building floor slabs may be modified (increased or decreased) by the structural engineer responsible for foundation design based on his/her calculations and engineering experience and judgment.

Spread Footings – PV Trackers

Footing Size and Embedment

Where the PV tracker support poles are to be founded on shallow spread footings, the footings should be embedded a minimum of 2 feet below the lowest adjacent grade. The footings may be either square or rectangular with the long axis of the footing perpendicular to the longitudinal axis of the table. The minimum size of a square footing should be 5 feet. Rectangular footings should have a minimum width of 3 feet and a minimum length of 6.5 feet, and a minimum thickness of 1.5 feet. The remaining 1.5 feet above the footing may be covered with excavated soils. Smaller footings may be possible with additional embedment below grade. Loads for the tracker table arrays should be provided so that the spread footings can be sized accordingly.

Allowable Bearing Value

An allowable bearing value of 2,500 pounds psf may be used for the tracker footings founded in undisturbed native soils. The allowable bearing value includes both dead and live loads and may be increased by one-third when considering short-duration wind loading. The bearing value may not be increased during strong seismic shaking, due to the reduction in soil strength.

Settlement

Based on the anticipated settlement characteristics of the native soils, it is estimated that the total settlement of the spread footings for a static loading condition will be less than approximately 1 inch.

Lateral Resistance

A passive earth pressure of 250 pounds psf per foot of depth, to a maximum value of 2,500 psf pounds, may be used to determine lateral bearing resistance for footings. In addition, a coefficient of friction of 0.30 times the dead load forces may be used between concrete and the supporting soils to determine lateral sliding resistance. The above values may be increased by one-third when designing for transient wind or seismic forces. It should be noted that the above values are based on the condition where footings are cast in direct contact with compacted fill or competent native soils.

Driven Pile Foundations

Solar panels may also be mounted on driven piles that extend above the ground to the desired level of panel mounting. We have assumed that the panels would be mounted at a level of about 4 feet above the ground in order to compare methods of foundation support. Piles could be driven a predetermined length so that the pile heads were at the same elevation, and then a carrier beam could be attached to the pile heads and the solar panel tracker table mounted to the carrier beam. If it would not interfere with solar panel rotation in sun tracking, the piles could be driven at even a slight batter to increase the lateral load resistance through transference of the lateral forces into axial resistance. Battered piles would then result in an "A" frame to support the solar panels most efficiently.

Utilizing soil parameters derived from testing at this site, and utilizing an average value for soil strength properties from these tests, we can present a generic driven pile design that can be used for comparison purposes. The pile type utilized in our analysis consists of a nominal 4-inch diameter hollow steel pipe pile (schedule 40 pipe – 4.5-inch diameter with a 0.237-inch wall thickness), weight of 10.8 lbs/ft, and driven with a closed end to displace the soil as it is driven. Additional soil testing would be required to determine a final pile design. We have presented designs based on vertical alignment of the piles at this time. We can study battered piles further once additional operational and design information can be supplied to us.

Our analyses of pile capacity were based on procedures of the American Petroleum Institute (API) as given in Reese (2006), and Salgado (2006). Strength parameters for our analysis were obtained from laboratory test results and our experience with similar materials in the area. The material properties used for design are presented in Table 1.

TABLE 1

Subsurface Material Types	Total Unit Weight (pcf)	Cohesion (psf)	Angle of Internal Friction (degrees)	e50% Strain Above and Below the Water Table
Sand and Sandy Silt	103 to 109	360	22	NA

The following Table 2 presents the vertical downward and uplift capacities for the piles as well as lateral capacity. A lateral load of 10 kips per pile was applied at the pile head at a height of 6 feet above the ground. The resulting lateral deflection was determined and is shown in the table. Lateral loads for lower levels of lateral deflection can be obtained from the graphs and tables included with Appendix F showing deflections at various lesser loads. All values in the following table are Ultimate Load Conditions.

TABLE 2

(Pile Head 4' Above Ground Line, Free Head Condition)

Pile Type	Total Pile Length (Feet)	Pile Depth Below Ground (Feet)	Ultimate Vertical Capacity (Kips)	Ultimate Uplift Capacity (Kips)	Lateral Pile Head Deflection (In) at 10 Kips Shear at Pile Head	Pile Deflection at Ground Line (Inches)	Depth to First Point of Zero Deflection from Pile Head (Feet)	Maximum Moment (Kip-Ft)	Maximum Shear (Kips)
4" Steel Pipe Pile, Driven Closed Ended, Hollow, Schedule 40 pipe with 0.237 inch wall Thickness, not filled with Concrete	14	10	6.9	4.7	1.08	0.29	7.9	4.5	1.9

The pile type considered was based on the efficiency of soil pile structure interaction effects and the amount of area of the pile to bear against the soil versus the amount of weight of the pile. Increased stiffness for the steel pile could be achieved by filling the pile with concrete after driving. Piles should not be placed any closer than 3.75B without consideration of group effects in the direction perpendicular to the pile row, and not any closer than 7B in the direction parallel to the pile row (Reese 2006).

Depending on operational and load considerations various pile types should be considered for this project. The soils at the site are corrosive to metals and concrete therefore the pile should be protected from corrosion as specified by a qualified corrosion engineer. Corrosion test results are discussed later in the report text.

Driven-Pile Construction

Piles should be constructed and driven in accordance with the applicable subsections of Sections 49, 50, 51 and 90 of the Caltrans Standard Specifications (Caltrans, 2006) and the following recommendations. Piles should be checked for alignment and plumbness. The amount of acceptable misalignment of a pile is usually on the order of approximately 2 to 3 inches from the exact location; however closer alignment may be required for proper solar panel mounting; this should be determined by the structural engineer. It is usually acceptable for a pile to be out of plumb one percent of the depth of the pile. If alignment is a concern then piles should be driven with the use of a template to help control the drift during driving. Piles should be spaced no closer than 2 times the nominal diameter or maximum dimension (center-to-center) but not less than 3 feet.

The pile hammer should be an approved steam, air or diesel hammer that develops sufficient energy to drive piles at a penetration rate of not less than 1/8-inch per blow at the design load.

Indicator Piles and Load Testing

Do to the many unknowns yet remaining at this time, we recommend that an indicator pile program be considered for investigating the actual load capacity of various piles of several types, and locations throughout the project, and the results utilized to make final pile design decisions. An indicator pile program would investigate the site soils further and additional field explorations and soil sampling would be combined with the results of pile load tests at various locations across the site to determine site-specific pile design parameters. This would allow for a refined pile design that would provide an efficient use of project resources.

SOLAR PANEL FOUNDATION CONSTRUCTION CONSIDERATIONS

We have provided two types of foundation options for consideration of solar panel foundation support. The panels could be supported on spread footings or driven pipe pile foundations. Although not included herein, a third option could consist of drilled piers. There are several advantages and disadvantages to each design. The following construction considerations should be used to compare the relative value of between each type.

Spread footings

- Spread footings can be constructed easily from the ground surface with or without overexcavation of the native soils.
- Shallow spread footings would locally be subject to expansive soil conditions. However, in view of the recommended footing embedment, the soils would likely expand in a more uniform manner due to the reduction in wetting and drying extremes. There could be some differential movement between footings and adjacent buried conduits that could be mitigated by designing the cable systems to accommodate some movement.
- Spread footings would generate a significant amount of spoil soils.
- Backfilling over the spread footing would be required.

Drilled Piers

- Drilled piers would generate a large amount of spoil soils.

- Drilled piers are based on a rigid body design and may not be as efficient in lateral load resistance per amount of material utilized as the other options.

- The piers must be extended from ground elevation to solar panel height in a second construction sequence by placement of a concrete pier or steel post.

Driven Piles

- Piles can be driven from the ground surface, no excavation is required.

- Piles can be of such a length that they are driven with the pile head at the required mounting height of the solar panel.

- Piles will not require disposal of displaced spoil soils.

- Large quantities of piles will have to be procured and handled on site.

- A few larger piles or many smaller piles could be utilized depending on material, handling, and driving costs per unit. It is our experience that pile handling and setup would most impact overall efficiency.

- Full displacement piles should be utilized to increase the load capacity of the soils. Steel pipe piles should be driven with a closed end.

- With the implementation of an indicator program production piles could be procured in a specified length and driven to desired final grade.

ACCESS ROADS

The proposed site improvements may include construction of new asphalt-paved parking areas and maintenance roads, as well as improvements to the existing access road. Alternatively, the access and maintenance roads may be constructed of aggregate base entirely without asphalt. We have developed the following preliminary recommendations for flexible pavement design based on an assumed R-value of 40 and using Traffic Index (TI) values of 5.0 and 6.0. The pavement section thicknesses presented in Table 4 are considered as minimums for the subject site, and may be superseded by the requirements of the client or jurisdictional agency if more stringent.

TABLE 4
Suggested Minimum Flexible Pavement Thickness

Traffic Index	R-Value	Hot Mix Asphalt (inches)	Aggregate Base (inches)
5.0	40	3	4
5.0	40	0	10.5
6.0	40	3	6.5

All aggregate base material should be compacted to a minimum relative compaction of 95 percent (ASTM D1557-07) prior to placing asphalt pavement. Base material should conform to the requirements for Untreated Base Materials, Section 200-2 of the latest edition of Standard Specifications for Public Works Construction (Greenbook).

Subgrade drainage is an important factor that enhances pavement performance. Subgrade surfaces below the flexible pavement structural section should be sloped to direct run-off to suitable collection points and to prevent ponding. The roadways should be raised above the surrounding ground surface to facilitate drainage from the roadway.

The asphalt pavement design presented herein is based on the assumption that the pavement will be placed directly over engineered, compacted fill. R-value and traffic index parameters presented herein have also been assumed. We recommend that bulk samples of the actual subgrade materials be retrieved and tested after rough grading is completed. Once actual as-graded conditions are confirmed, additional testing and modified design recommendations may be presented.

CONCRETE FLATWORK

General

We recommend that all exterior concrete flatwork be designed by the project structural engineer with consideration given to mitigating the potential cracking and uplift that can develop in soils exhibiting expansion index values that fall in the very low or low category.

The guidelines that follow should be considered as minimums and are subject to review and revision by the project structural engineer and/or landscape consultant as deemed appropriate.

Thickness and Joint Spacing

To reduce the potential of unsightly cracking, concrete walkways and patio-type slabs should be at least 4 inches thick and provided with construction joints or expansion joints every 6 feet or less.

Reinforcement

All concrete flatwork having their largest plan-view panel dimension exceeding 10 feet should be reinforced with a minimum of No. 3 bars spaced 24 inches on centers, both ways. Alternatively, the slab reinforcement may consist of welded wire mesh of the sheet type (not rolled) with 6x6/W1.4xW1.4 designation in accordance with the Wire Reinforcement Institute (WRI). The reinforcement should be properly positioned near the middle of the slabs.

> *The reinforcement recommendations provided herein are intended as guidelines to achieve adequate performance for anticipated soil conditions. The project architect, civil and/or structural engineer should make appropriate adjustments in reinforcement type, size and spacing to account for concrete internal (e.g., shrinkage and thermal) and external (e.g., applied loads) forces as deemed necessary.*

Subgrade Preparation

To reduce the potential for distress to concrete flatwork, the subgrade soils below concrete flatwork areas to a minimum depth of 12 inches should be moisture conditioned to at least equal to, or slightly greater than, the optimum moisture content and then compacted to a minimum relative compaction of 90 percent.

Pre-Moistening

As a further measure to reduce the potential for concrete flatwork cracking, subgrade soils should be thoroughly moistened prior to placing concrete. The moisture content of the soils should be at least 1.2 times the optimum moisture content and penetrate to a minimum depth of 12 inches into the subgrade.

GENERAL CORROSIVITY SCREENING

The following sections represent an interpretation of current codes and specifications that are commonly used in our industry as they relate to the adverse impact of chemical components of the site soils on various components of the proposed structures. As a screening level study, limited chemical testing was performed on representative samples of onsite soils to identify potential corrosive characteristics of these soils. A variety of test methods are available to quantify the corrosive potential of soils. The testing procedures referred to herein are considered to be typical for our industry and have been adopted and/or approved by many public or private agencies

Petra does not practice corrosion engineering; therefore, the opinion and engineering judgment provided herein should be considered as general guidelines only. Further analyses would be warranted for cases where buried metallic building materials such as copper and ductile iron are planned for the project. For these conditions, we recommend that the project design professionals (i.e., the architect and/or structural engineer) consider recommending a qualified corrosion engineer to conduct additional sampling and testing of near-surface soils during the final stages of site grading to provide a complete assessment of soil corrosivity. Recommendations to mitigate the detrimental effects of corrosive soils on buried metallic and other building materials that may be exposed to corrosive soils should be provided by the corrosion engineer as deemed appropriate.

Concrete in Contact with Site Soils

Soils containing soluble sulfates beyond certain threshold levels as well as acidic soils are considered to be detrimental to integrity of concrete placed in contact with such soils. For the purpose of this study, soluble sulfates concentration in soils determined in accordance with California Test Method No. 417. Soil acidity, as indicated by hydrogen-ion concentration (pH), was determined in accordance with California Test Method No. 643.

The results of our laboratory tests indicate that on-site soils within the subject site contain water soluble sulfate contents of 0.006 to 0.072 percent. Based on Section 1904.3 of the 2010 CBC, concrete that will be exposed to sulfate-containing soils should comply with the provisions of Section 4.3 of ACI 318.

According to Table 4.2.1 of ACI 318-08 (a precursor to Section 4.3), an exposure class of S0 is appropriate for onsite soils. As such, a **Not Applicable** exposure to sulfate may be expected for concrete placed in contact with the onsite soil materials. As directed by Table 4.3.1 of ACI 318-08, no restriction for cement or maximum water-cement ratio for the fresh concrete would be required for this condition. However, the concrete minimum unconfined compressive strength should not be less than 2,500 psi.

The results of limited in-house testing of representative samples indicate that soils within the subject site are moderately alkaline with respect to pH (pH of 8.0 to 8.3). Based on this finding and according to Section 8.22.2 of Caltrans' 2003 Bridge Design Specifications (2003 BDS) requirements (which consider the combined effects of soluble sulfates and soil pH), a commercially available Type II Modified cement may be used.

These recommendations should be verified by the project structural engineer and the contractor responsible for concrete placement for concrete used in footings and interior slabs-on-ground, foundation walls and concrete exposed to weather.

Metals Encased in Concrete

Soils containing a soluble chloride concentration beyond a certain threshold level are considered corrosive to metallic elements such as reinforcement bars, cables, bolts, etc. that are encased in concrete that, in turn, is in contact with such soils. For the purpose of this study, soluble chlorides in soils were determined in accordance with California Test Method No. 422.

The results of limited screening tests performed indicate that onsite soils contain a water-soluble chloride concentrations of 112 to 362 parts per million (ppm). Section 1904.4 of CBC 2010 requires that reinforcement in concrete be protected from the corrosive effects of chloride exposure in accordance with Section 4.4 of ACI 318. It should be noted that Section 4.4 of ACI 318-08 pertains to freeze-and-thaw conditions that are not applicable to the subject project; however, regardless of the level of chlorides in soils in contact with concrete, Table 4.2.1 of ACI 318-08 assigns an exposure class of C1 for concrete that will be exposed to moisture but not necessarily to external sources of chlorides. As such, a **Moderate** exposure to chloride may be expected for metallic elements encased in concrete, which is, in turn, placed in contact with the onsite soil materials.

One method of protecting reinforcement in concrete where elevated chloride concentrations are present in the soils is to increase the thickness of the concrete cover over the reinforcement. However, Table 8.22.1 of Caltrans BDS 2003 provides no minimum concrete cover when chloride concentration is less than 500 ppm (as is the case for the subject site). This recommendation should be verified by the project structural engineer.

Metallic Elements in Contact with Site Soils

Elevated concentrations of soluble salts in soils tend to induce low level electrical currents in metallic objects in contact with such soils. This process promotes metal corrosion and can lead to distress to building components that are in contact with site soils. The minimum electrical resistivity indicates the relative concentration of soluble salts in the soil and, therefore, can be used to estimate soil corrosivity with regard to metals. For the purpose of this investigation, the minimum resistivity in soils is measured in accordance with California Test Method No. 643.

The minimum electrical resistivity for onsite soils was found to be 480 to 3,700 ohm-cm based on limited testing. This result indicates that on-site soils are moderately to **Severely Corrosive** to ferrous metals and copper. As such, any ferrous metal or copper components of the subject buildings or panel foundations that are expected to be placed in direct contact with site soils should be protected against detrimental effects of severely corrosive soils.

POST-GRADING RECOMMENDATIONS

Site Drainage

Positive-drainage devices, such as sloping flatwork, graded-swales and/or area drains, should be provided around buildings to collect and direct water away from the structures. Neither rain nor excess irrigation water should be allowed to collect or pond against building foundations. Drainage should be directed to an appropriate discharge area. The ground surface adjacent to the structures should also be sloped at a gradient of 2 percent or more away from the foundations for a horizontal distance of 5 feet or more.

Utility Trenches

Utility-trench backfill materials to placed within access roads, utility easements, cable raceways, and under building-floor slabs should be compacted to a relative compaction of 90 percent or more. Where onsite soils are utilized as backfill, mechanical compaction should be used. Density testing, along with probing, should be performed by the project geotechnical consultant or his representative to document adequate compaction.

Utility-trench sidewalls deeper than about 3 feet should be laid back at a ratio of 1:1 (h:v) or flatter or shored. A trench box may be used in lieu of shoring. If shoring is anticipated, the project geotechnical consultant should be contacted to provide design parameters.

For trenches with vertical walls, backfill should be placed in approximately 1- to 2-foot thick loose lifts and then mechanically compacted with a hydra-hammer, pneumatic tampers or similar compaction equipment. For deep trenches with sloped walls, backfill materials should be placed in approximately 8- to 12-inch-thick loose lifts and then compacted by rolling with a sheepsfoot tamper or similar equipment.

Where utility trenches are proposed in a direction that parallels any structural footing (interior and/or exterior trenches), the bottom of the trench should not be located within a 1:1 (h:v) plane projected downward from the outside bottom edge of the adjacent footing.

PLAN REVIEW AND CONSTRUCTION SERVICES

This report has been prepared for the exclusive use of Regenerate Power, LLC, to assist the project team in the design of the proposed development. It is recommended that Petra be engaged to review the final-design drawings and specifications prior to construction. This is to document that the recommendations contained in this report have been properly interpreted and are incorporated into the project specifications. If Petra is not accorded the opportunity to review these documents, we can take no responsibility for misinterpretation of our recommendations.

We recommend that Petra be retained to provide soil-engineering services during grading and construction of the excavation and foundation phases of the work. This is to observe compliance with the design, specifications or recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated prior to start of construction.

If the project plans change significantly (e.g., structural loads or types), we should be retained to review our original design recommendations and their applicability to the revised construction. If conditions are encountered during construction that appears to be different than those indicated in this report, this office should be notified immediately. Design and construction revisions may be required.

LIMITATIONS

This report is based on the project, as described, and the preliminary geologic/geotechnical field data obtained from the limited field tests performed at the locations shown. The materials encountered on the project site and utilized in our laboratory evaluation are believed representative of the total area, and the conclusions and recommendations contained in this report are presented on that basis. However, soil materials and groundwater levels can vary in characteristics between points of excavation, both laterally and vertically.

The conclusions and opinions contained in this report are based on the results of the described geotechnical evaluations and represent our professional judgment. The contents of this report are professional opinions and as such, are not to be considered a guaranty or warranty. The findings, conclusions and opinions contained in this report are to be considered tentative only and subject to confirmation by the undersigned during the construction process. Without this confirmation, this report is to be considered incomplete and Petra or the undersigned professionals assume no responsibility for its use. In addition, this report should be reviewed and updated after a period of 1 year or if the site ownership or project concept changes from that described herein.

The professional opinions contained herein have been derived in accordance with current standards of practice and no warranty is expressed or implied. This report has not been prepared for use by parties or projects other than those named or described herein. This report may not contain sufficient information for other parties or other purposes.

We sincerely appreciate this opportunity to be of service. Please do not hesitate to call the undersigned if you have any questions regarding this report.

Respectfully submitted,

PETRA GEOTECHNICAL, INC.

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Memorandum
RE: San Felipe Creek/Seville Solar Complex Response

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M E M O R A N D U M

DATE: JULY 8, 2013

FROM: RICK SIDOR, P.E., AEI-CASC

TO: DWIGHT CAREY, ENVIRONMENTAL MANAGEMENT ASSOCIATES, INC.

RE: San Felipe Creek/Seville Solar Complex response

This memorandum has been prepared to clarify the questions regarding AEI-CASC's preliminary analysis of flood depths which may impact the southerly section of the Seville Solar Complex Project during a major storm event assuming that the existing berm along the westerly project boundary does not exist. Exhibit A of the AEI-CASC flood hazard assessment "Preliminary On-Site and Off-Site Hydrology and Flood Hazard Analysis for Allegretti Farms Solar Project Site", dated May 22, 2013 depicts the flooding limits for the preliminary 100-year, 24-hour flood of through the southwestern corner of the site along San Felipe Creek (attached). The exhibit includes seven cross sections through the site. These cross sections, taken perpendicular to the direction of flow, indicate the estimated water surface elevations (WSE) based upon normal depth calculations. Along each cross section, at approximately 1,000 ft stations, the approximate flood hazard depth is shown. The flood hazard depths are also shown where the cross sections intersect the property boundary.

Subsequent to producing the report, AEI-CASC was asked by the Regenerate Power team to review the cross sections and underlying topographic information and confirm that two points, depicting depths of 3.6 ft and 2.8 ft, are not indicative of the depths within the area that would be utilized for the solar arrays. AEI-CASC was asked to supplement the flood hazard assessment by approximating flood hazard depths at points within the solar development area proximate to Cross Section 6, station 59+00, and Cross Section 5, Station 70+51.

In response to Regenerate's requests, AEI-CASC reviewed the cross sections and estimated the flood depths adjacent to the boundary but within the likely solar development area. On the attached close-up of the Exhibit A map we have labeled the contours and the estimated flood depth within the adjacent development area. The findings are summarized as follows:

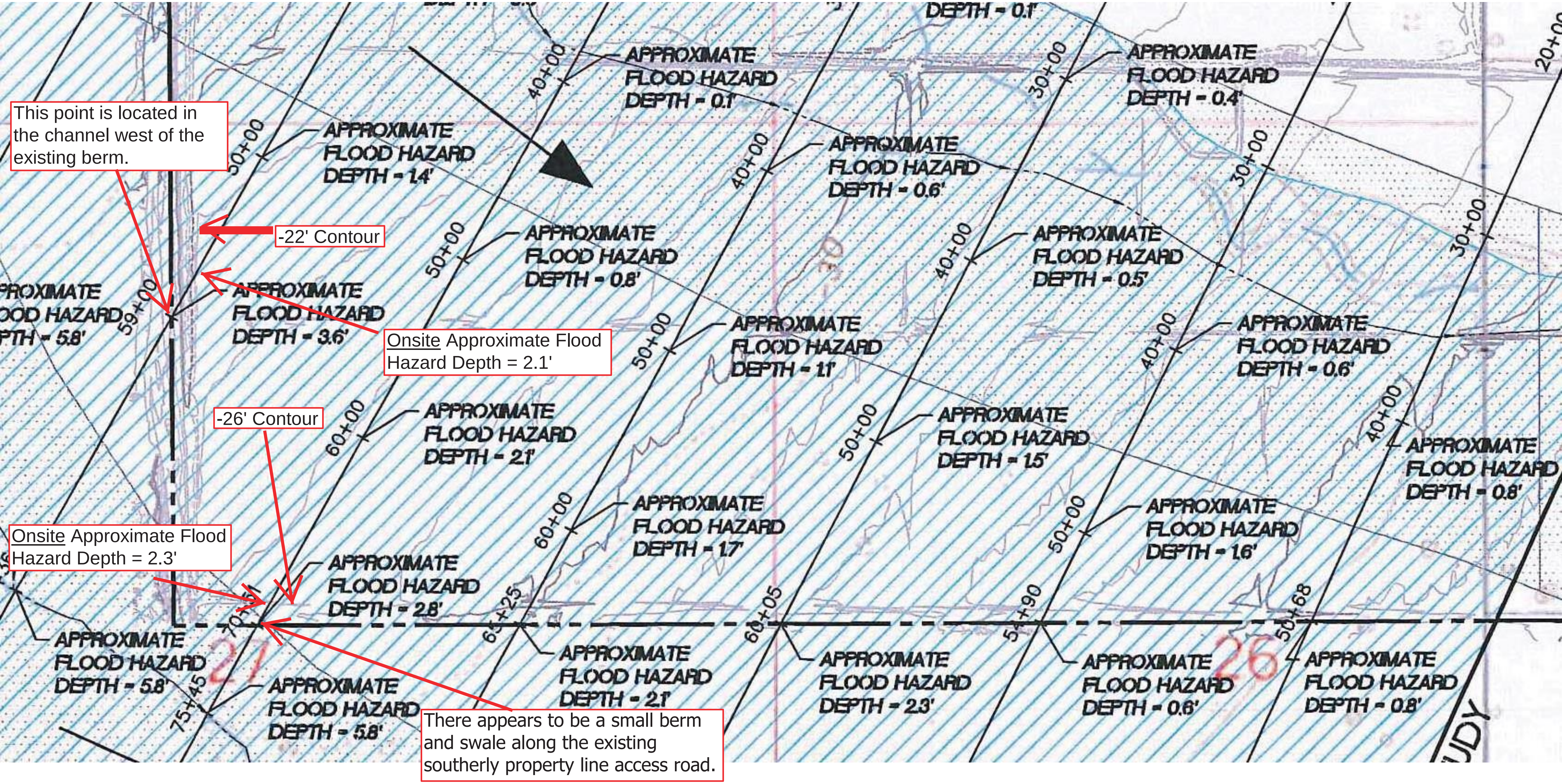
1. Cross Section 6, station 59+00, shows a depth of 3.6 ft. However, this point is located in the bottom of an existing swale along the west side of the earthen berm that has been graded along the western edge of the property. This is not indicative of the depth of flow within the proposed solar development area east of the berm. The depth of flow within the adjacent development area east of the berm appears to be approximately 2.1 ft and decreasing to the northeast.

2. Cross Section 5, Station 70+51, shows a depth 2.8 ft. However, this point appears to be located in a swale running adjacent to the south of the southern perimeter road. This area

also appears to be a small earthen berm adjacent to the swale. The 2.8ft depth is not indicative of the depth of flow in the solar development area to the north. Flood depths in the adjacent area appear to be closer to 2.3 ft in depth at the most, and decreasing northeasterly along the cross section.

As previously stated, it should be noted that this analysis is a planning level study. Detailed hydraulic analysis of the flood plain utilizing computer models will provide more accurate information.

Attachment: Modified Exhibit A


DEPTH = 0.1'
APPROXIMATE FLOOD HAZARD DEPTH = 0.1'
APPROXIMATE FLOOD HAZARD DEPTH = 0.4'
This point is located in the channel west of the existing berm.
APPROXIMATE FLOOD HAZARD DEPTH = 1.4'
APPROXIMATE FLOOD HAZARD DEPTH = 0.6'
-22' Contour
APPROXIMATE FLOOD HAZARD DEPTH = 0.8'
APPROXIMATE FLOOD HAZARD DEPTH = 0.5'
APPROXIMATE FLOOD HAZARD DEPTH = 5.8'
APPROXIMATE FLOOD HAZARD DEPTH = 3.6'
Onsite Approximate Flood Hazard Depth = 2.1'
APPROXIMATE FLOOD HAZARD DEPTH = 1.1'
APPROXIMATE FLOOD HAZARD DEPTH = 0.6'
-26' Contour
APPROXIMATE FLOOD HAZARD DEPTH = 2.1'
APPROXIMATE FLOOD HAZARD DEPTH = 1.5'
APPROXIMATE FLOOD HAZARD DEPTH = 0.8'
APPROXIMATE FLOOD HAZARD DEPTH = 1.7'
APPROXIMATE FLOOD HAZARD DEPTH = 1.6'
Onsite Approximate Flood Hazard Depth = 2.3'
APPROXIMATE FLOOD HAZARD DEPTH = 2.8'
APPROXIMATE FLOOD HAZARD DEPTH = 5.8'
APPROXIMATE FLOOD HAZARD DEPTH = 2.1'
APPROXIMATE FLOOD HAZARD DEPTH = 2.3'
APPROXIMATE FLOOD HAZARD DEPTH = 0.6'
APPROXIMATE FLOOD HAZARD DEPTH = 0.8'
APPROXIMATE FLOOD HAZARD DEPTH = 5.8'
There appears to be a small berm and swale along the existing southerly property line access road.
27
26
59+00
50+00
40+00
30+00
20+00
60+00
75+45
65+25
60+05
54+90
50+68

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