

Appendix O

Drainage Report

Preliminary Drainage Report

Bonanza Peak Solar Power Generating Facility,
Town of Charleston View, Inyo County, California

Prepared for:
Bonanza Peak Solar, LLC

Date:
February 6, 2026

Prepared by:
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Project/File:
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Revision Record

Revision	Description	Author	Date	Quality Check	Date	Independent Review	Date

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Acronyms / Abbreviations

Acronym / Abbreviation	Full Name
2D	Two-dimensional
A	Area
A-T	Adjustment-Transposition
BFE	Base Flood Elevation
CFR	Code of Federal Regulations
CFS	Cubic Feet Per Second
CN	Curve Number
CWB	California Water Boards
DEM	Digital Elevation Model
DTSD	Division of Transportation System Development
DY	Debris Yield
F	Fahrenheit
FF	Fire Factor
FT	Feet
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
FIRM	FEMA Flood Insurance Rate Map
FIS	FEMA Flood Insurance Study
Fr	Froude Number (hydraulic output)
GIS	Geographic Information Systems
HEC-18	Hydraulic Engineering Circular Number 18 (Federal Highway Administration publication)
HEC-23	Hydraulic Engineering Circular Number 23 (Federal Highway Administration publication)
HEC-RAS	U.S. Army Corps of Engineers Institute of Water Resources Hydrologic Engineering Center – River Analysis System
HSG	Hydrologic Soil Group
HUC	Hydrologic Unit Code
LiDAR	Light Detection and Ranging
MSL1912	Mean Sea Level Datum of 1912
MW	Mega Watt
NAD1983	North American Datum of 1983
NAVD88	North American Vertical Datum of 1988
NFIP	National Flood Insurance Program
NGVD29	National Geodetic Vertical Datum of 1929
NHD	National Hydrography Database
NLCD	National Land Cover Database
NOAA	National Oceanic and Atmospheric Administration
NRCS	Natural Resources Conservation Service
NWS	National Weather Service
OS	Open Space
O&M	Operations and Maintenance
P	Precipitation
PV	Photovoltaic



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Introduction

Q	Flow Rate
RAS	River Analysis System
RR	Relief Ratio
SCS	Soil Conservation Service (now the Natural Resources Conservation Service)
SFHA	Special Flood Hazard Area
SSURGO	Soil Survey Geographic Database
SWDV	Surface Water Data Viewer
TR-55	Technical Release 55
USACE	United States Army Corps of Engineers
USDA	United States Department of Agriculture
USDOT	United States Department of Transportation
USGS	United States Geological Survey



1 Introduction

1.1 Project Description

The Bonanza Peak Solar Project (Project) proposes construction of an up to 500 megawatts (MW) photovoltaic solar energy generation facility on up to 2,400 acres (Developed Area) within a larger 3,387-acre area (Project Area). The Project is located on privately-owned land in the unincorporated Town of Charleston View, in Inyo County, California. The Project is located adjacent to the California-Nevada State Line just north of California State Route 178 (Old Spanish Trail Highway) (**Figure 1**).

Bonanza Peak Solar, LLC (the Client) plans to build, operate, and maintain the facilities depicted on the Site Plan (**Figure 2**). Construction of these facilities includes the following components:

- Installing solar panel arrays within the Developed Area (on up to 2,400 acres);
- Installing underground collection lines from each panel to a collection point substation;
- Constructing an operations and maintenance building;
- Upgrading access roads within the facility for construction and maintenance;
- Constructing an on-site collection point electrical substation on 19 acres;
- Installing and maintaining an on-site interconnection facility;
- Constructing an approximately 9-mile-long electrical transmission line from the Project to an existing point of connection near the 161-kV transmission line at Yellow Pine Energy Center in Clark County, Nevada; and
- Preliminary access to the Project would be provided via Old Spanish Trail Highway to Zircon Street, an existing dirt road approximately 2 miles west of the California-Nevada State Line.



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1 Introduction

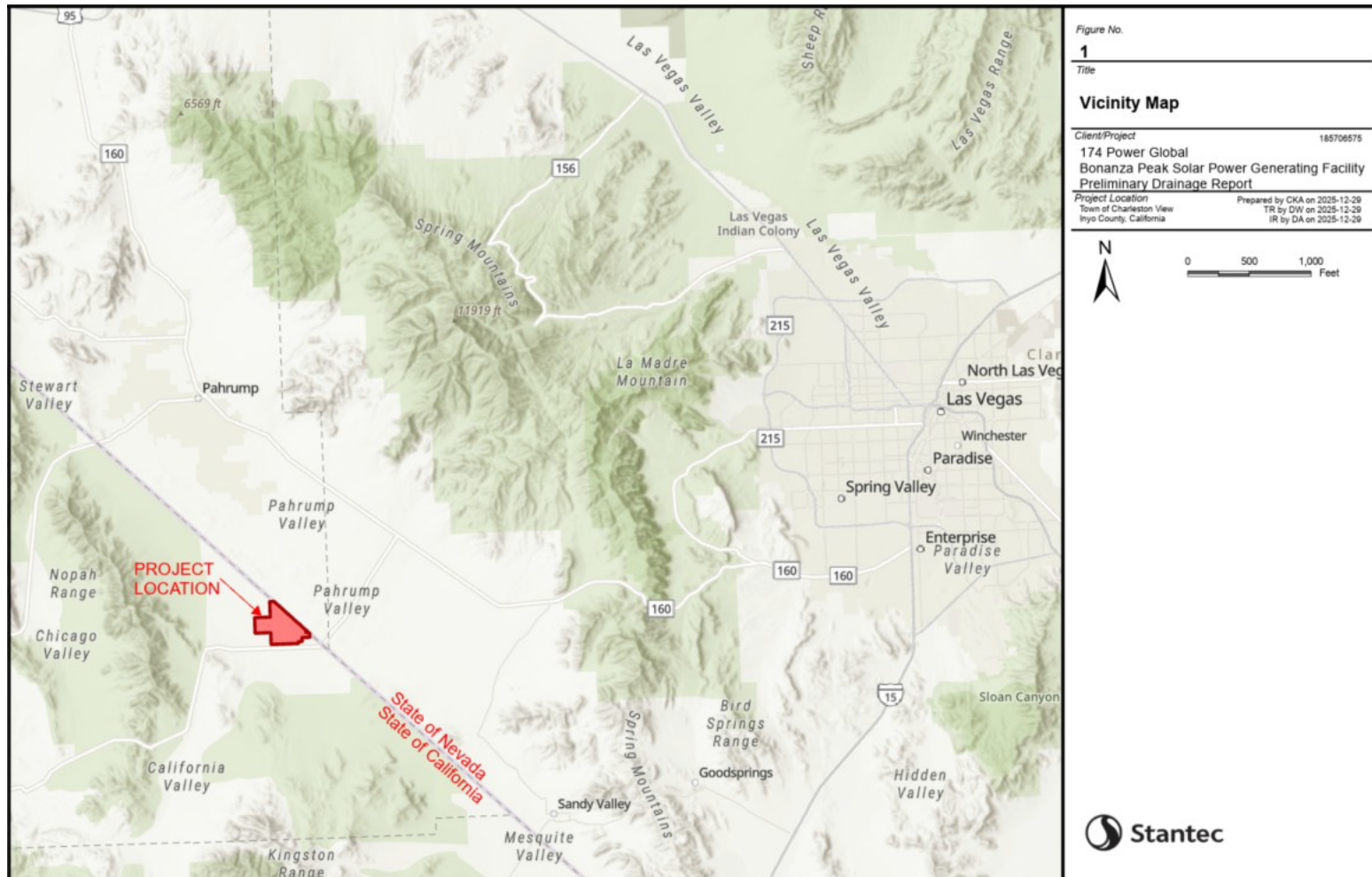


Figure 1. Vicinity Map



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1 Introduction

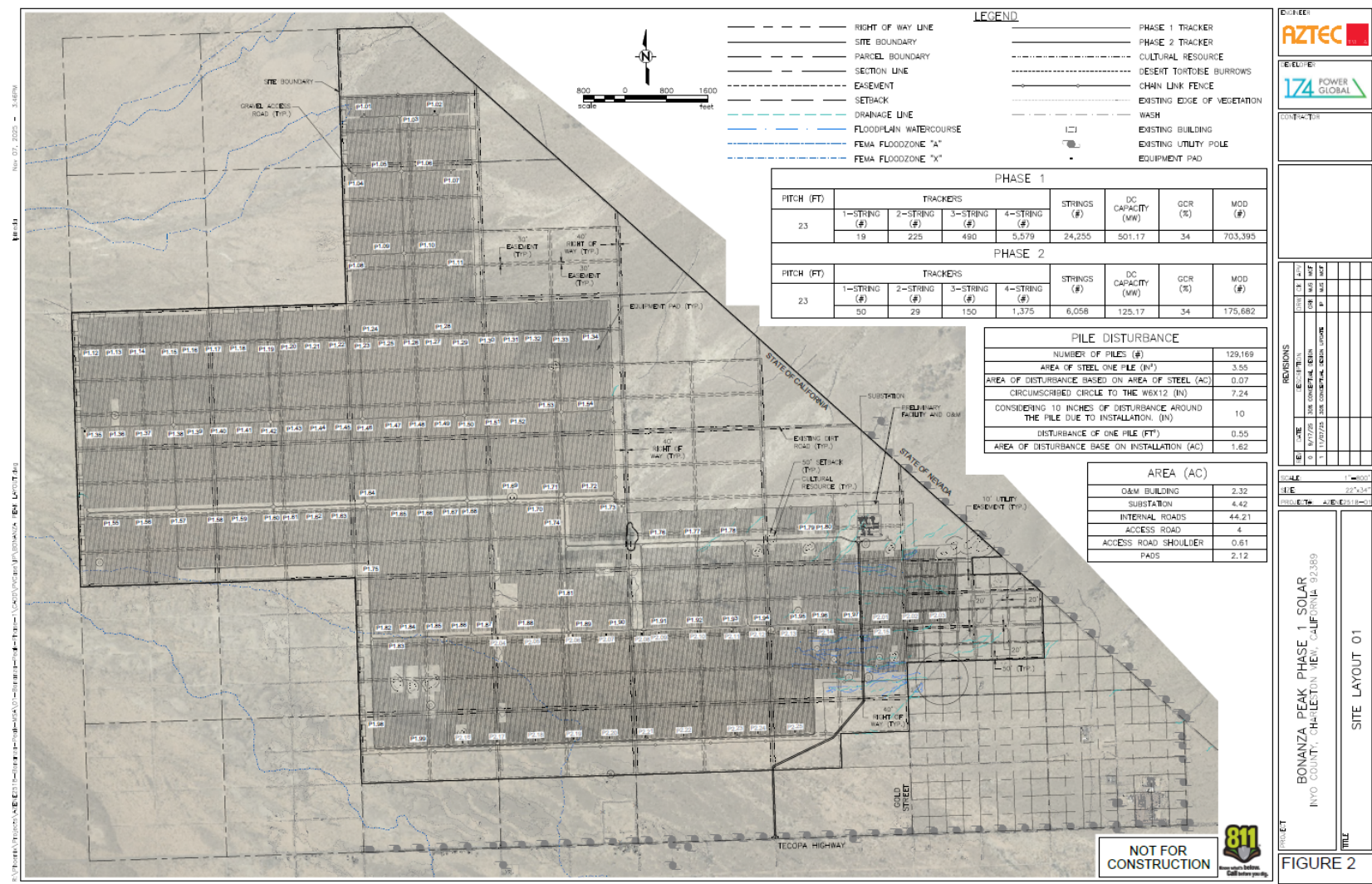


Figure 1. Site Plan



1.1.1 County Zoning

The County's Zoning Map identifies the zoning of the Project Site as Open Space (OS-40), which provides sites for open space and compatible uses. Commercial renewable energy facilities are an allowable use within the OS-40 land use zoning district. Existing development and disturbed areas within the Project Site include rural access roads and scattered abandoned rural buildings. **Table 1** provides a summary of land uses.

Table 1. Land Use Summary

LAND USE	AREA (acres)
Solar Panels, PV Inverters, and Access Roads	2,355
Internal Roads	44.21
Access Road and Shoulder	4.61
Electrical Substation	4.42
Pads	2.12
O&M building	2.32
Drainage Facilities	21.21
DEVELOPED AREA TOTALS =	2,434
Undeveloped Remainder	953
ENTIRE SITE TOTALS =	3,387

1.2 Purpose

This Preliminary Drainage Report (Report) was prepared to evaluate the flood hazard environment within the Project Area. The purpose of this Report is to conduct a preliminary hydrology and hydraulic assessment and quantify any potential impacts and mitigation measures needed for the future final designs.

The primary objectives of this drainage report include the following:

- Identify (1) existing drainage features, (2) flood processes, (3) stable and unstable surfaces., and 4) flood conditions that would constrain the future development of the Site;
- Quantify project-related impacts specific to increases in flood distribution, runoff yield, and flow rates for the 1-percent annual chance flood hazard; and
- Perform preliminary sizing of the proposed local detention basins. Preliminary sizing is based on the 100-year event only. Other events may be evaluated during final engineering.

At the time of this Report, the County of Inyo had not published any formal Hydrology Manual, and conversations with the County Engineer indicate that the County of Inyo has not adopted hydrology manuals from adjacent counties (such as San Bernardino nor Riverside County).



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1 Introduction

At the time of this Report, the County of Inyo has not published any detention basin criteria and conversations with the County Engineer indicate that the County of Inyo has not adopted detention basin design criteria from adjacent counties (such as San Bernardino nor Riverside County).

Given that this Report covers the Project's conceptual design, only the 100-year event is evaluated in this report. Other events will be evaluated during final engineering.



2 Existing Watershed Characteristics

2.1 FEMA Flood Plain

FEMA Zone A floodplains create regulatory limitations on development by setting maximum allowable increases to proposed water surface elevations. FEMA Zone A are areas with a 1% annual chance of flooding. Detailed analyses have not been performed in these areas; therefore, no flood depths or base flood elevations are shown or defined in these zones. The Inyo County floodplain administrator is responsible for enforcing floodplain regulations such as the National Flood Insurance Program (NFIP), the Code of Federal Regulations (CFR) 60, and State regulations, when applicable.

The existing Federal Emergency Management Agency (FEMA) map was used to determine if the Project Area is located within the limits of the 100-year floodplain. Review of the existing FEMA Flood Insurance Rate Map (FIRM), panels 06027C4175D and 06027C4625D indicate that only a small portion of the Project Area (at the north end of the site) is located in Federal Emergency Management Agency (FEMA) Special Flood Hazard Area (SFHA) Zone A floodplain (**Figure 3a, Figure 3b**).

The project does not propose any improvements or grading within the FEMA Flood Zone A.

2.5 miles downstream of the proposed project, flows continue to an existing 4,500-acre dry lakebed, which ultimately discharges to the Death Valley Low Point (undrained low spot).

2.2 Upstream Watershed Boundary Conditions

The region is subject to temperature extremes, ranging from the 20 degrees Fahrenheit (°F) in the winter months to more than 100°F in the summer, with variations based on elevation. The total precipitation normal (average annual rainfall) is approximately 4.97 inches (NOAA 2025), though the region is subject to intense seasonal storms, and thus the annual rainfall in a given year may vary greatly. Desert washes are characterized by uncertain and changing flow paths, high velocity flow, and active sediment transport, scour and deposition.

The majority of the offsite drainage area tributary to the Project Area originates from the Charleston Peak mountains located 5 miles east in the State of Nevada (**Figure 4**). Charleston Peak's highest tributary is at 7,550 feet (ft) in elevation, while the Project Area elevation ranges from 2,670 ft to 2,565 ft. The physical tributary drainage area based on the USGS topographic mapping is many square miles that extend primarily east and south of the Project Area. The total offsite drainage area is approximately 335.6 square miles.

Due to the erratic nature of dry washes, including the potential to change course during major storm events, all watercourses were reviewed using the latest available aerial imagery in Google Earth. The delineation followed the ridges on the contour and concluded that there are six main regional watersheds tributary or adjacent to the Project Area. The characteristics of all six regional watersheds are summarized in **Table 2**.

Table 2. Regional Watershed Characteristics



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2 Existing Watershed Characteristics

WATERSHED	AREA (square miles)	EXISTING 100 YEAR Q (cfs)
A	61.7	3,503
B	134.7	1,674
C	30.1	2,538
D	49.9	3,828
E	31.1	1,834
F	28.1	2,397
TOTAL =	335.6	



Figure 3a. FEMA FIRM North

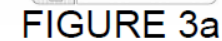
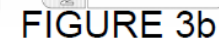


Figure 3b. FEMA FIRM South



Preliminary Drainage Report 2 Existing Watershed Characteristics

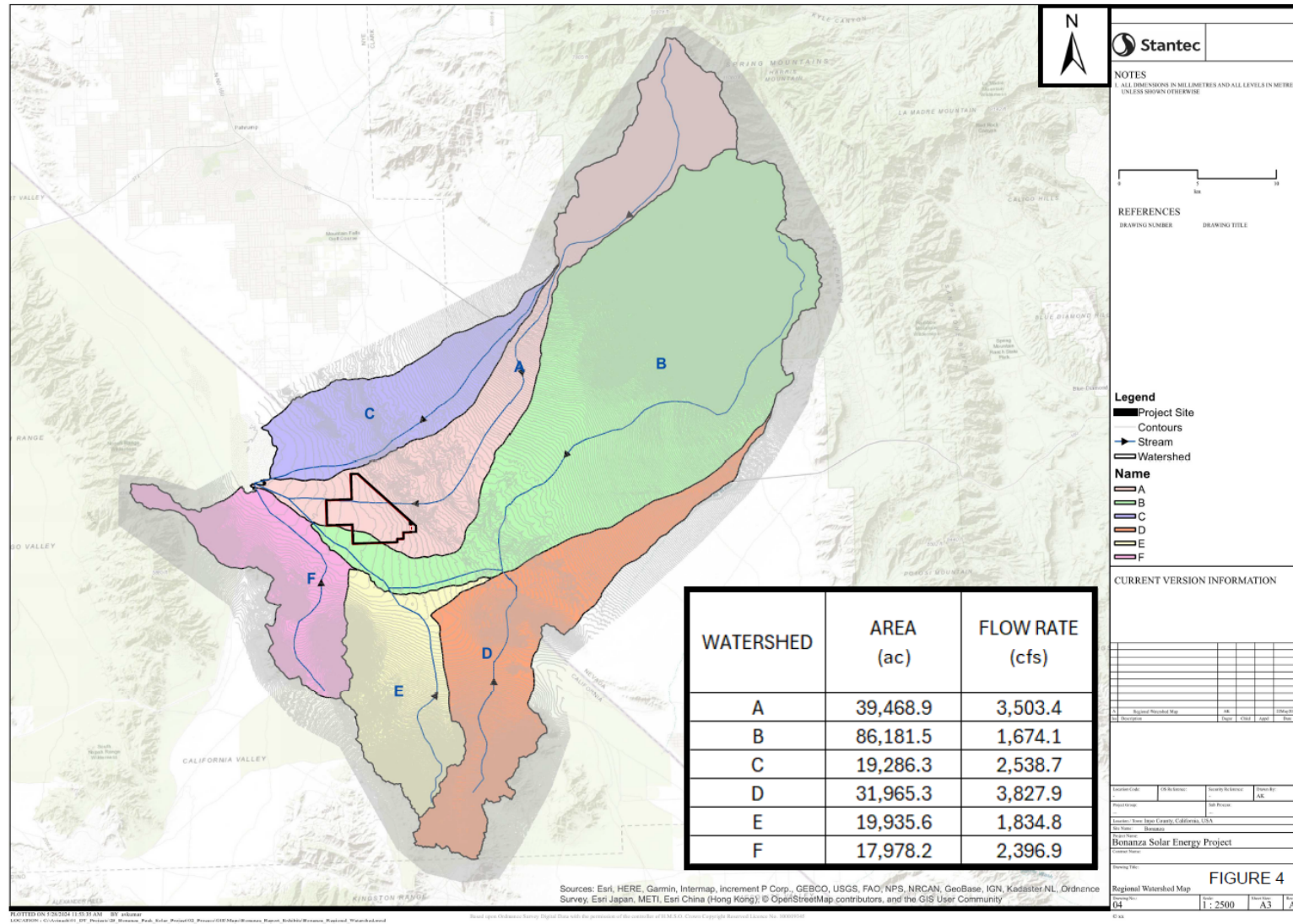


Figure 4. Watershed Map



2.3 Existing On-Site Watershed

The Project Area has two distinct topographies. The “upper” portion of the Project Area has a topography with an average slope of 1.5 %, which encompasses approximately 1,162 acres 34% of the Project Area. The “lower” portion of the Project Area has a flatter topography with an average slope of 0.4%, which encompasses approximately 2,225 acres, or 66% of the Project Area.

Within the Project Area, the terrain generally slopes northwesterly. The topology of the proposed site is dominated by unconfined sheet flooding. Limited drainage infrastructure exists in the drainage area. Vehicular routes become flooded during storm events.

The maximum existing flood depths for the Project Area occurs on the lower, flatter portions of the Project Area and reach a maximum = 1.53 ft deep (**Figure 5**).

The maximum existing velocities for the Project Area occur on the higher, steeper portions of the Project Area and reach a maximum = 2.52 ft per second within the Developed Area (**Figure 6**). For the entire Project Area, the velocities reach over 4 ft per second.

Aside from a few dirt roads graded for maintenance, the Project Area is currently undeveloped and includes mix of native and non-native vegetation. Land cover can be characterized as desert shrubs with poor cover (less than 50% of vegetation cover). There is no impervious development within the Project Area.

The majority of the proposed developed area contains soils of hydrologic soil group B with moderate infiltration, which is discussed further in Section 3.8 (**Figure 7**).

2.4 Downstream Watershed Boundary Conditions

Downstream of the Project Area, the regional watersheds flow to the northwest 2.5 miles to the existing 4,500-acre dry lakebed, with an average elevation is 1,775 ft. This dry lakebed historically collects runoff during major storm events, and ponds for a short while until evaporation and infiltration into the lake bottom.

Since the existing dry lakebed is 790 feet below the Project Area, there is no need to perform a downstream backwater analysis.



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2 Existing Watershed Characteristics

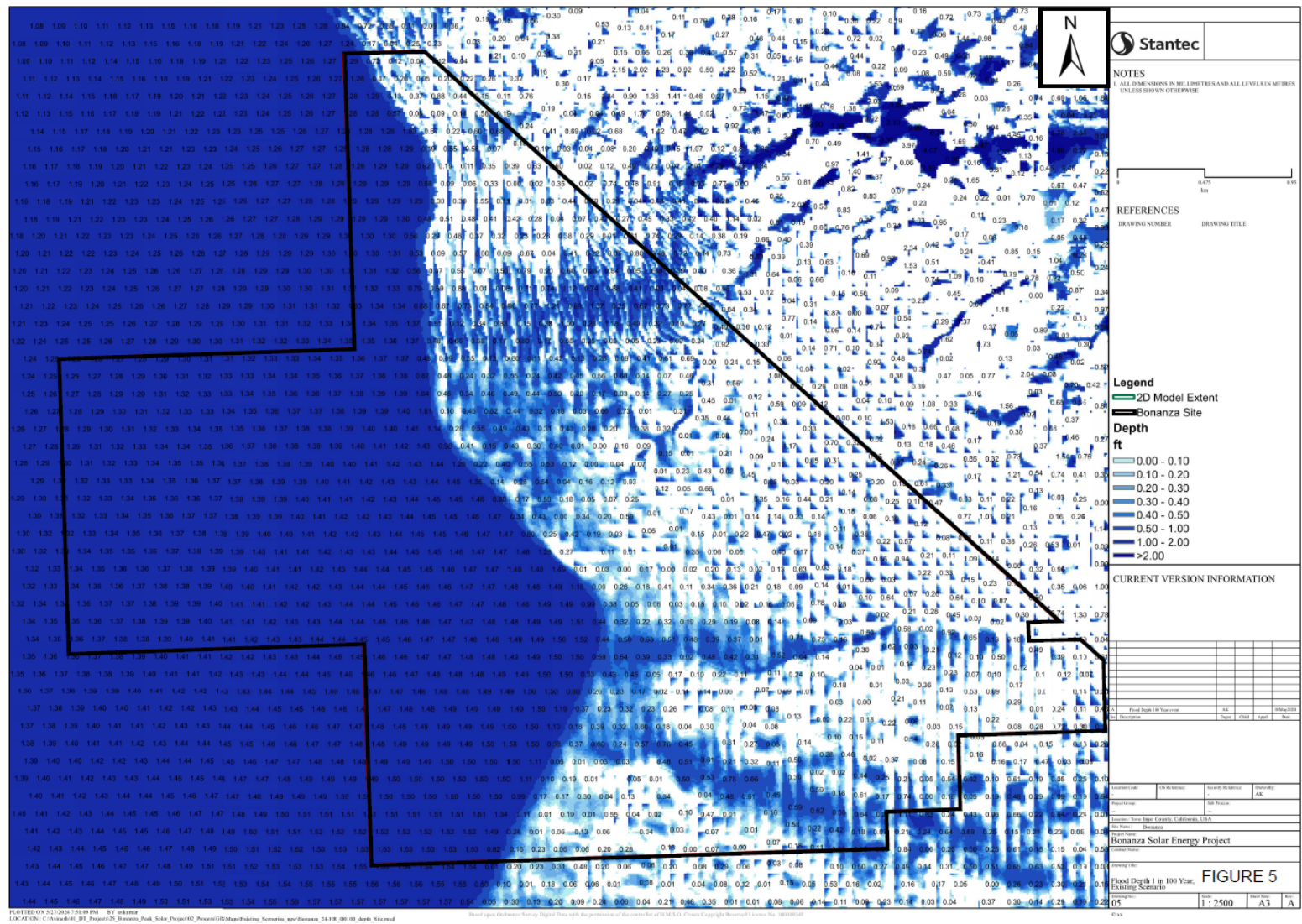


Figure 5. Existing 100 Year Flood Depth Map

Preliminary Drainage Report 2 Existing Watershed Characteristics

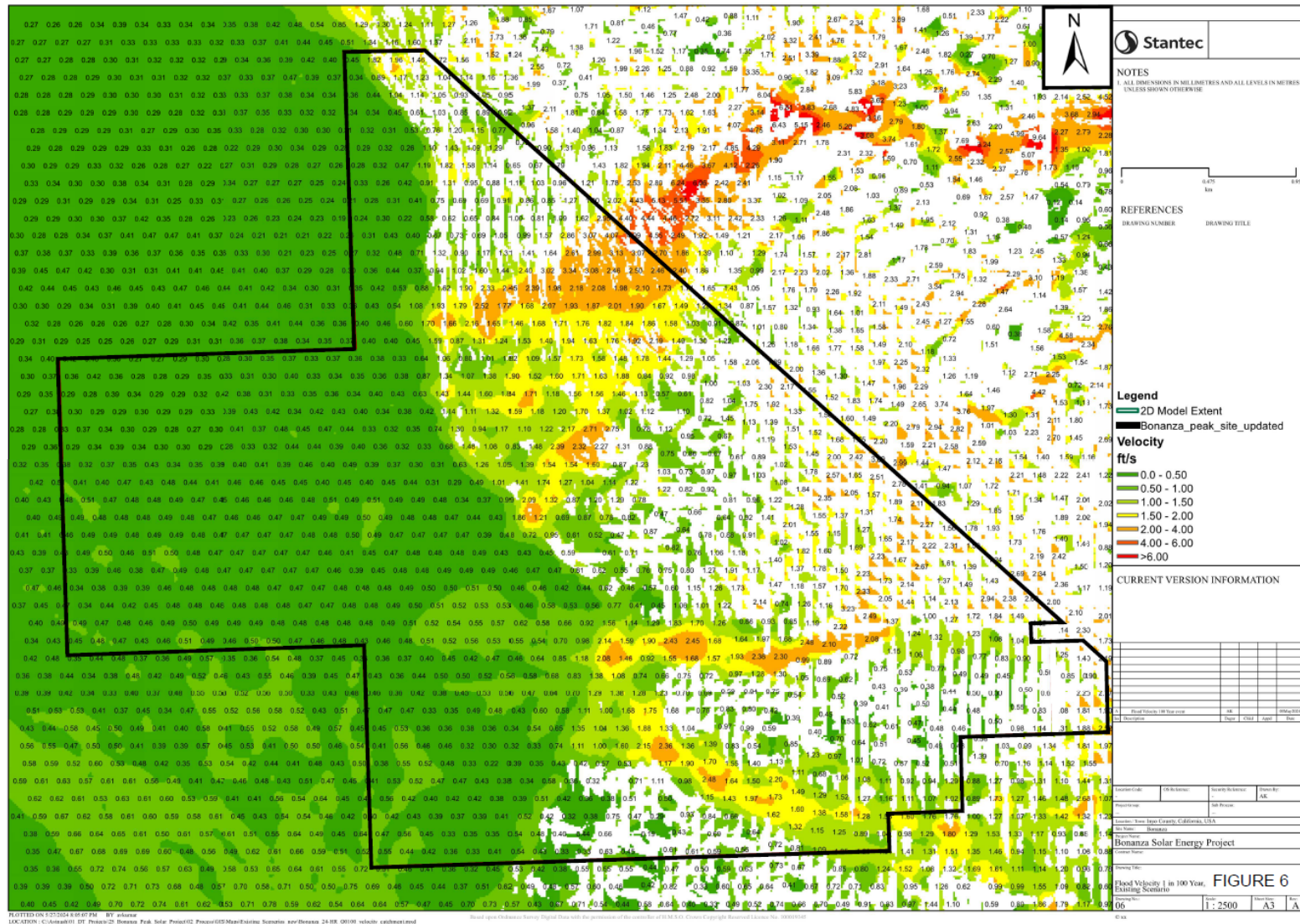


Figure 6. Existing 100 Year Flood Velocity Map

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2 Existing Watershed Characteristics

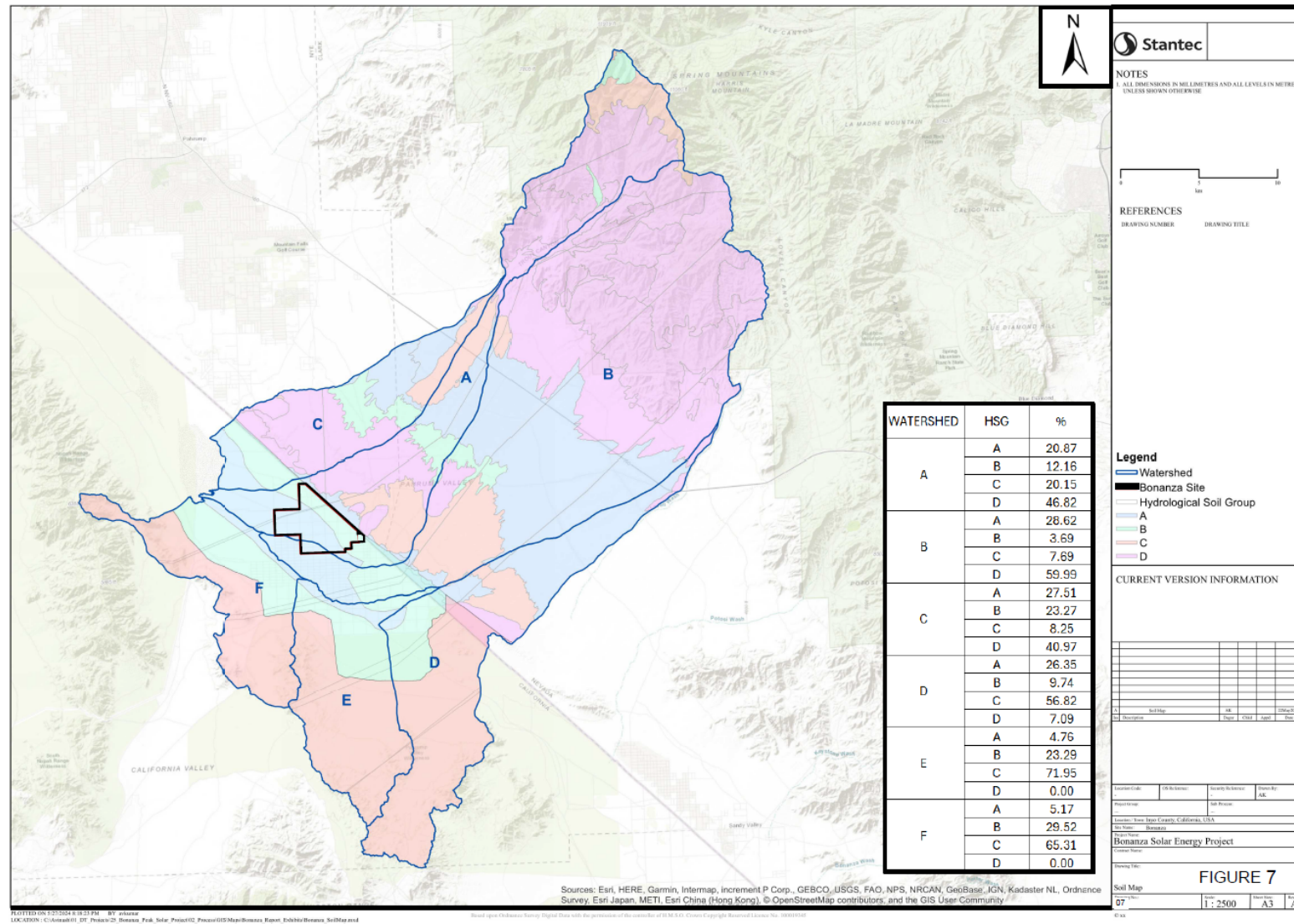


Figure 7. Soils Map

3 Proposed On-Site Grading

3.1 Proposed Grading Goals and Objectives

The Project's primary grading goal is to install the solar facilities and equipment with the minimum disturbance to the natural conditions including minimizing grading disturbances, maintaining existing drainage patterns, avoiding channelization of runoff, maintaining electrical equipment 12 inches above the 100-year flood plain level, and installing detention basins as needed to mitigate increase of flows to downstream neighboring parcels.

To minimize the potential for concentrated flows and erosion, diversion berms and miscellaneous fill should not be implemented anywhere on the Developed Area, except at the proposed PV inverter islands, the Substation pad, and the O&M Building Pad which need to be elevated to raise the electrical equipment above the flood plain. *(see discussion below for the PV Inverter Island Pads, Electrical Substation Pad, and O&M Building Pad)*

Although the proposed solar panel array areas will not be graded, the inherent use of large equipment to install the solar piles, etc. will impact the perviousness of the existing ground slightly, and for this Report, the solar fields have been assigned a 5% pervious land cover factor.

Local detention basins, of longitudinal shape, will be graded to capture onsite runoff and mitigate it to levels below the 100% existing peak discharge. Mitigated peak discharges will be discharged downstream into dedicated public street right-of-ways along the downstream perimeter of the Project. Local detention basins are anticipated to be relatively shallow (less than 4 ft deep including freeboard).

3.2 Proposed On-Site Inverter Pads

The solar photovoltaic array panels will be installed via steel pipe piles driven directly into the ground, and the panels are elevated to maximize solar efficiency. The scouring potential and mitigation of scouring at these driven piles are discussed further in Sections 5.0 and 6.0.

The solar panel arrays are interconnected via underground cabling to electrical inverters that are located within the solar array field. Each proposed inverter station is to be located atop an elevated "island pad" to maintain freeboard above the maximum flood elevations.

3.3 Proposed On-Site Electrical Substation Pad and O&M Building Pad

The proposed Electrical Substation and O&M Building shall be constructed atop elevated pads (approximately 4.42 and 2.32 acres, respectively) to maintain freeboard for the electrical equipment above the maximum flood elevations. The pads will be graded to catch the existing grade and minimize disturbance of existing slopes/surface surrounding the pads.



4 Hydrologic and Hydraulic Modeling

4.1 HEC-RAS Modeling Software

The U.S. Army Corps of Engineers HEC-RAS software (U.S. Army Corps of Engineers Institute for Water Resources, 2021), was chosen as the computer modeling package for the Project. HEC-RAS version 6.4.1 features utilized during the analysis include rain-on-grid hydrology and two-dimensional (2D) overland flow routing to assess the hydrologic and hydraulic conditions of the watershed surrounding the Project.

4.2 Project Extents and Model Domain

The 3,387-acre Project area is located in the unincorporated Town of Charleston, Inyo County, California. The Project extents are defined by the entire watershed that is tributary to the vicinity of the Project.

The total model domain area is approximately 335.6 square miles (214,784 acres).

4.3 Horizontal Coordinate System and Vertical Datum

The horizontal coordinate system used in the HEC-RAS model and associated files is the North American Datum of 1983 (NAD 1983) State Plane California Zone IV in U.S. Customary Feet. Note, U.S. Customary Feet are also referred to as U.S. Survey Feet, which is distinguished from International Feet.

All elevations and water surfaces used in the HEC-RAS model and reported in this document are reported in the North American Vertical Datum of 1988 (NAVD88). No site data was provided in other vertical datums such as the National Geodetic Vertical Datum of 1929 (NGVD29) or the Mean Sea Level datum of 1912 (MSL1912).

4.4 Topography

The National Hydrography Dataset (NHD) includes a general delineation of the Charleston Peak watershed and its tributary subwatersheds. The NHD delineation was used as a general guideline and supplemented by a computerized watershed delineation process in ArcGIS. The delineation used a combination of two topographic USGS datasets, including:

- 10-foot Digital Elevation Model (DEM) for the entire watershed using the U.S. Geological Survey, 20220419, USGS 1/3 Arc Second n35w115 20220419 dataset.
- 1-meter LiDAR, 2018 contours for the vicinity of the project from the United States Geological Survey (USGS), Southern California Wildfires, dataset.

Due to the erratic nature of dry washes, including the potential to change course during major storm events, all watercourses were reviewed using the latest available aerial imagery in Google Earth. The delineation followed the ridges on the contour and concluded that there are five main regional watersheds tributary or adjacent to the Project Area.

The topographic mapping was converted from meters to feet.



4.5 Hydrologic Methodology

A hydrologic model calculates the water cycle processes, including infiltration, evaporation, evapotranspiration, and runoff. Hydraulic modeling simulates runoff as it is routed/conveyed over the physical landscape, such as over the landscape and through culverts, bridges, and in channels. The rainfall-runoff hydrologic parameters used in the HEC-RAS models for the Project were developed following the methodology established in Technical Release 55 (U.S. Department of Agriculture Natural Resources Conservation Service, 1986), which provides an overview of the use of the Soil Conservation Service (SCS) Curve Number (CN) method.

The SCS CN method determines an approximate amount of runoff from a rainfall event in a particular area. The major inputs of the SCS CN method are:

- *Rainfall* – Design rainfall is discussed in Section 4.9;
- *Initial abstraction* – All losses occur before runoff begins and is generally correlated with soil and land cover parameters; and
- *Potential maximum retention after runoff begins* – Based on the soil and land cover conditions, which define the Curve Number discussed in Section 4.7 and 4.8.

Rain-on-grid hydrology was used for analyzing the Project Area. The rain-on-grid method evaluates each topographic grid cell in the HEC-RAS model as its own watershed. For this method, a composite Curve Number is determined for each cell. HEC-RAS combines the rain-on-grid cell information with precipitation, impervious areas, the slope of the landscape, and hydrologic condition to determine the runoff potential of a cell.

4.6 Land Cover and Manning's Roughness

The existing land cover can be characterized as 100% desert shrub with poor cover for the areas around the Project Area with portions of existing graded roads characterized as barren.

The Project will consist of 3 types of modification:

Solar Arrays: The areas of solar panels that are not graded shall be modeled as remaining generally pervious (less than **5% impervious**).

Inverter Pads: The inverter island fill pads shall be surfaced with all-weather aggregate based and modeled as **15% impervious**.

Electrical Substation Pad: the Electrical Substation Pad shall be surfaced with all-weather aggregate based and modeled as **85% impervious**.

O&M Building Pad: O&M Building Pad shall include the building rooftop with all-weather aggregate base surrounding and modeled as **85% impervious**.

Access: The proposed access roads shall be modeled as remaining generally pervious (less than **5% impervious**).

The proposed land cover summary is depicted in **Table 3**.



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4 Hydrologic and Hydraulic Modeling

Table 3. Impervious Land Cover Summary

AREA	ACREAGE	PERCENT IMPERVIOUS	IMPERVIOUS ACREAGE
Solar Panels and Roads	2,420.67	5%	121.03
Inverter Island Pads	6.59	15%	0.99
Substation Pad	4.42	85%	3.76
O&M Building Pad	2.32	85%	1.97
DEVELOPED AREA TOTALS =	2,434	AVG 5.4%	132.36
Undeveloped Remainder	953	0%	0
ENTIRE SITE TOTALS =	3,387	AVG 3.9%	132.36

The Manning's roughness coefficients (**Table 4**) were assigned based on the landcover identified in the NLCD and technical guidance published in the HEC-RAS 2D Modeling User's Manual (U.S. Army Corps of Engineers, 2021).

Table 4. Model Domain Impervious Percentages and Manning's n Values

FEATURE	SELECTED IMPERVIOUS PERCENTAGE ¹	LITERATURE RANGE OF MANNING'S ROUGHNESS COEFFICIENTS ²	SELECTED MANNING'S ROUGHNESS COEFFICIENT
Undefined (No Data)	100%	0.05	0.05
Developed, Open Space	9.41%	0.03-0.05	0.04
Developed, Low Intensity	29.29%	0.06-0.12	0.09
Developed, Medium Intensity	57.68%	0.08-0.16	0.12
Developed, High Intensity	85.03%	0.12-0.20	0.16
Cultivated Crops	---	0.02-0.05	0.035
Herbaceous	---	0.025-0.05	0.0375
Shrub/Scrub	---	0.07-0.16	0.115
Grassland/Herbaceous	---	0.025-0.05	0.0375
Pasture/Hay	---	0.025-0.05	0.0375
Emergent Herbaceous Wetlands	---	0.05-0.085	0.0675
Barren Lands	---	0.023-0.03	0.0265
Open Water	100%	0.025-0.05	0.0375

¹ U.S. Geological Survey, 2019

² U.S. Army Corps of Engineers, 2021



4.7 Grid Cells and Mesh

A variable-sized mesh (an array of grid cells) was used to define the model domain. The mesh includes the following grid cell zones to optimize the model detail and simulation runtimes:

- The Project extents associated with the planned PV assets was assigned a 200x200-foot grid cell size.
- The critical areas near the substation pad were assigned a 25x25-foot grid cell size.
- The critical areas near the O&M pad were assigned a 25x25-foot grid cell size.
- Remaining areas not described above were assigned a 250x250-foot grid cell size.

4.8 Hydrologic Soils Groups and Curve Numbers

As mentioned in previous sections the *potential maximum retention after runoff begins* is an important factor in the runoff from an area. The Curve Number provides an empirical quantification of the runoff potential.

Curve Numbers are assigned based on:

- The Hydrologic Soil Group (HSG), which is based on the underlying soil's runoff potential;
- The land cover type (derived from the NLCD dataset and discussed above; and
- The condition of the land cover (e.g., fallow farm field, spring plantings, late summer growth, etc.).

Because impervious areas are separately classified in the HEC-RAS runoff calculation routine, HSGs are the primary variable in determining runoff potential. The HSG information was obtained from the National Resources Conservation Service (NRCS) Soil Survey Geographic Database (SSURGO) (Natural Resources Conservation Service, 2020). HSGs are assigned a letter classification by the database. 'A' soils have the highest infiltration potential (lowest runoff potential), 'B' soils have a somewhat middling infiltration potential (middling runoff potential), 'C' soils have a low infiltration potential (high runoff potential), and 'D' soils have a very low infiltration potential (very high runoff potential). The SCS Soil Groups utilized are identified in **Table 5**, below:

Table 5. SCS Soil Groups and Filtration Rates

SCS Soil Group	Description	Range of Loss Rates (inches/hour)
A	Deep sand, deep loess, aggregated silts	0.3 – 0.45
B	Shallow loess, sandy loam	0.15- 0.30
C	Clay loams, shallow sandy loam, soils low in organic content, and soils usually high in clay	0.05 – 0.15
D	Soils that swell significantly when wet, heavy plastic clays, and certain saline soils	0 – 0.05

The HSG Soils are overlaid atop the six regional watersheds (**Figure 4**) and are tabulated in **Table 6**.



Table 6. HSG Percentages Per Watershed

WATERSHED	HSG	%	WATERSHED	HSG	%
A	A	20.87	D	A	26.35
	B	12.16		B	9.74
	C	20.15		C	56.82
	D	46.82		D	7.09
B	A	28.62	E	A	4.76
	B	3.69		B	23.29
	C	7.69		C	71.95
	D	59.99		D	0
C	A	27.51	F	A	5.17
	B	23.27		B	29.52
	C	8.25		C	65.31
	D	40.97		D	0

Table 7. Selected Curve Number Adapted From TR-55

LAND COVER	CURVE NUMBER			
	HSG A	HSG B	HSG C	HSG D
Developed, Open Space	39	61	74	80
Developed, Low Intensity	54	70	80	85
Developed, Medium Intensity	61	75	83	87
Developed, High Intensity	89	92	94	95
Cultivated Crops	65	75	82	86
Herbaceous	30	58	71	78
Shrub/Scrub	35	56	70	77
Deciduous Forest	36	60	73	79
Evergreen Forest	44	65	76	82
Mixed Forest	36	60	73	79
Hay/Pasture	49	69	79	84
Woody Wetlands	98	98	98	98
Emergent Herbaceous Wetlands	98	98	98	98
Barren Land	77	86	91	94
Open Water	99	99	99	99

Areas with undefined HSGs were classified using available data and engineering judgement. (U.S. Department of Agriculture Natural Resources Conservation Service, 1986)

4.9 Design Storm Precipitation Events

Design storms are statistically derived pattern precipitation-frequency events published for the use of water resources engineers to design infrastructure. NOAA Atlas 14 was used to obtain precipitation data for the site for the 100-year event (**Table 8**).



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Table 8. Selected Curve Number Adapted From TR-55

 NOAA Atlas 14, Volume 6, Version 2 Location name: Tecopa, California, USA* Latitude: 35.9267°, Longitude: -115.914° Elevation: 2908 ft** * source: ESRI Maps ** source: USGS 										
POINT PRECIPITATION FREQUENCY ESTIMATES Sanja Perica, Sarah Dietz, Sarah Heim, Lillian Hiner, Kazungu Maitaria, Deborah Martin, Sandra Pavlovic, Ishani Roy, Carl Trypaluk, Dale Unruh, Fenglin Yan, Michael Yekta, Tan Zhao, Geoffrey Bonnin, Daniel Brewer, Li-Chuan Chen, Tye Parzybok, John Yarchoan NOAA, National Weather Service, Silver Spring, Maryland PF_tabular PF_graphical Maps & aeriels										
PF tabular										
PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)¹										
Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.087 (0.072-0.107)	0.135 (0.111-0.166)	0.200 (0.164-0.246)	0.253 (0.206-0.314)	0.327 (0.258-0.419)	0.385 (0.297-0.504)	0.445 (0.335-0.598)	0.510 (0.373-0.703)	0.600 (0.421-0.862)	0.673 (0.457-1.00)
10-min	0.125 (0.103-0.153)	0.194 (0.160-0.238)	0.286 (0.235-0.352)	0.363 (0.295-0.450)	0.469 (0.369-0.601)	0.552 (0.426-0.723)	0.639 (0.481-0.857)	0.730 (0.535-1.01)	0.859 (0.604-1.24)	0.964 (0.655-1.43)
15-min	0.151 (0.124-0.185)	0.235 (0.193-0.288)	0.346 (0.284-0.426)	0.439 (0.357-0.544)	0.567 (0.446-0.727)	0.668 (0.515-0.874)	0.772 (0.581-1.04)	0.883 (0.647-1.22)	1.04 (0.730-1.49)	1.17 (0.792-1.73)
30-min	0.213 (0.176-0.261)	0.331 (0.273-0.407)	0.489 (0.401-0.601)	0.619 (0.504-0.768)	0.800 (0.630-1.03)	0.943 (0.727-1.23)	1.09 (0.821-1.46)	1.25 (0.913-1.72)	1.47 (1.03-2.11)	1.65 (1.12-2.45)
60-min	0.294 (0.242-0.360)	0.456 (0.376-0.560)	0.673 (0.552-0.828)	0.853 (0.694-1.06)	1.10 (0.868-1.41)	1.30 (1.00-1.70)	1.50 (1.13-2.01)	1.72 (1.26-2.37)	2.02 (1.42-2.90)	2.27 (1.54-3.37)
2-hr	0.398 (0.328-0.487)	0.585 (0.481-0.717)	0.835 (0.685-1.03)	1.04 (0.850-1.30)	1.34 (1.05-1.71)	1.57 (1.21-2.05)	1.81 (1.36-2.43)	2.07 (1.52-2.85)	2.43 (1.71-3.50)	2.73 (1.86-4.06)
3-hr	0.458 (0.378-0.562)	0.660 (0.543-0.810)	0.931 (0.764-1.14)	1.16 (0.942-1.44)	1.47 (1.16-1.89)	1.73 (1.33-2.26)	1.99 (1.50-2.67)	2.27 (1.66-3.14)	2.68 (1.88-3.84)	3.00 (2.04-4.47)
6-hr	0.570 (0.470-0.699)	0.800 (0.658-0.982)	1.11 (0.911-1.36)	1.37 (1.12-1.70)	1.73 (1.37-2.22)	2.03 (1.56-2.65)	2.34 (1.76-3.13)	2.66 (1.95-3.68)	3.14 (2.20-4.51)	3.52 (2.39-5.24)
12-hr	0.688 (0.567-0.844)	0.951 (0.782-1.17)	1.30 (1.07-1.60)	1.60 (1.30-1.98)	2.02 (1.59-2.59)	2.35 (1.81-3.08)	2.70 (2.04-3.63)	3.08 (2.26-4.25)	3.63 (2.55-5.21)	4.07 (2.76-6.06)
24-hr	0.889 (0.782-1.03)	1.22 (1.07-1.42)	1.66 (1.46-1.94)	2.04 (1.77-2.38)	2.56 (2.16-3.08)	2.97 (2.47-3.65)	3.41 (2.77-4.28)	3.87 (3.07-4.98)	4.53 (3.47-6.05)	5.07 (3.76-6.98)
2-day	1.03 (0.910-1.20)	1.42 (1.25-1.65)	1.93 (1.69-2.24)	2.35 (2.05-2.76)	2.95 (2.49-3.55)	3.42 (2.84-4.20)	3.90 (3.17-4.90)	4.42 (3.51-5.69)	5.15 (3.94-6.87)	5.74 (4.26-7.90)
3-day	1.12 (0.985-1.30)	1.54 (1.35-1.78)	2.09 (1.83-2.43)	2.54 (2.21-2.98)	3.18 (2.69-3.83)	3.68 (3.05-4.52)	4.20 (3.41-5.26)	4.74 (3.76-6.10)	5.51 (4.21-7.35)	6.12 (4.54-8.43)
4-day	1.19 (1.05-1.38)	1.63 (1.43-1.89)	2.20 (1.93-2.56)	2.68 (2.34-3.14)	3.34 (2.83-4.03)	3.86 (3.21-4.74)	4.40 (3.58-5.52)	4.97 (3.94-6.39)	5.76 (4.40-7.69)	6.40 (4.74-8.80)
7-day	1.32 (1.16-1.52)	1.78 (1.56-2.06)	2.37 (2.08-2.76)	2.86 (2.49-3.36)	3.54 (3.00-4.27)	4.08 (3.38-5.01)	4.63 (3.77-5.81)	5.22 (4.14-6.72)	6.04 (4.62-8.06)	6.71 (4.98-9.24)
10-day	1.42 (1.25-1.64)	1.90 (1.67-2.20)	2.51 (2.20-2.92)	3.02 (2.63-3.53)	3.71 (3.14-4.47)	4.25 (3.53-5.22)	4.82 (3.92-6.04)	5.42 (4.30-6.97)	6.25 (4.78-8.34)	6.93 (5.14-9.54)
20-day	1.65 (1.45-1.91)	2.19 (1.92-2.54)	2.86 (2.51-3.33)	3.40 (2.96-3.98)	4.12 (3.49-4.97)	4.68 (3.88-5.74)	5.25 (4.27-6.59)	5.85 (4.64-7.53)	6.72 (5.13-8.96)	7.42 (5.50-10.2)
30-day	1.91 (1.68-2.22)	2.53 (2.23-2.94)	3.28 (2.88-3.82)	3.87 (3.37-4.54)	4.63 (3.92-5.59)	5.21 (4.33-6.40)	5.82 (4.73-7.30)	6.45 (5.12-8.30)	7.36 (5.63-9.82)	8.13 (6.03-11.2)
45-day	2.20 (1.93-2.54)	2.90 (2.55-3.37)	3.75 (3.29-4.36)	4.39 (3.82-5.14)	5.19 (4.39-6.26)	5.78 (4.80-7.10)	6.39 (5.20-8.02)	7.04 (5.58-9.06)	7.97 (6.09-10.6)	8.76 (6.49-12.1)
60-day	2.53 (2.23-2.94)	3.33 (2.93-3.87)	4.24 (3.72-4.94)	4.92 (4.28-5.76)	5.74 (4.86-6.92)	6.32 (5.25-7.76)	6.89 (5.60-8.65)	7.54 (5.98-9.70)	8.46 (6.47-11.3)	9.27 (6.88-12.8)
¹ Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS). Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values. Please refer to NOAA Atlas 14 document for more information.										

The HEC-RAS model simulates these design storm precipitation events as rain-on-grid precipitation.



5 Hydrologic and Hydraulic Results

5.1 Proposed On-Site Flood Depths

The maximum existing flood depths for the Project Area occur on the lower, flatter portions of the Project Area and reach a maximum = 1.53 ft deep (**Figure 5**). The proposed flood depths are similar and reach a maximum = 1.74 ft deep (**Figure 8**).

The proposed inverter “islands pads” and the substation pads are all safely located a min 2 ft above this flood level.

5.2 Proposed On-Site Velocities

The maximum existing velocities for the Project Area occur on the higher, steeper portions of the Project Area and reach a maximum = 2.52 ft per second (**Figure 6**). The proposed velocities are similar and reach a maximum = 2.65 ft per second. (**Figure 9**).

5.3 Proposed On-Site Detention Basins

The Project must comply with hydromodification guidelines (CWB, 2012) which require that the downstream off-site properties should not experience an increase of run-off due to the Project.

Local detention basins, of longitudinal shape, will be graded to capture onsite runoff and mitigate it to levels below the 100% existing peak discharge. Mitigated peak discharges will be discharged downstream into dedicated public street right-of-ways along the downstream perimeter of the project. Local detention basins are anticipated to be relatively shallow (less than 4 feet deep including freeboard).

Three detention basin areas are identified in **Figure 10**. The unit hydrographs for each detention basin were modeled and the difference in volume between existing hydrograph and proposed is summarized in **Table 9 and Figure 10**. The calculated debris yield (see Section 7.0) is included in the sizing of the detention basins.

Table 9. Detention Basin Summary

BASIN	EXISTING PEAK FLOW Q100 (cfs)	PROPOSED MITIGATED PEAK FLOW Q100 (cfs)	PROPOSED DETENTION BASIN VOLUME (Ac-ft)	BASIN DRAW DOWN TIME (hours)
DET1	785	45	71	2.4
DET2	1,460	11	29	2.4
DET3	821	76	111	2.4



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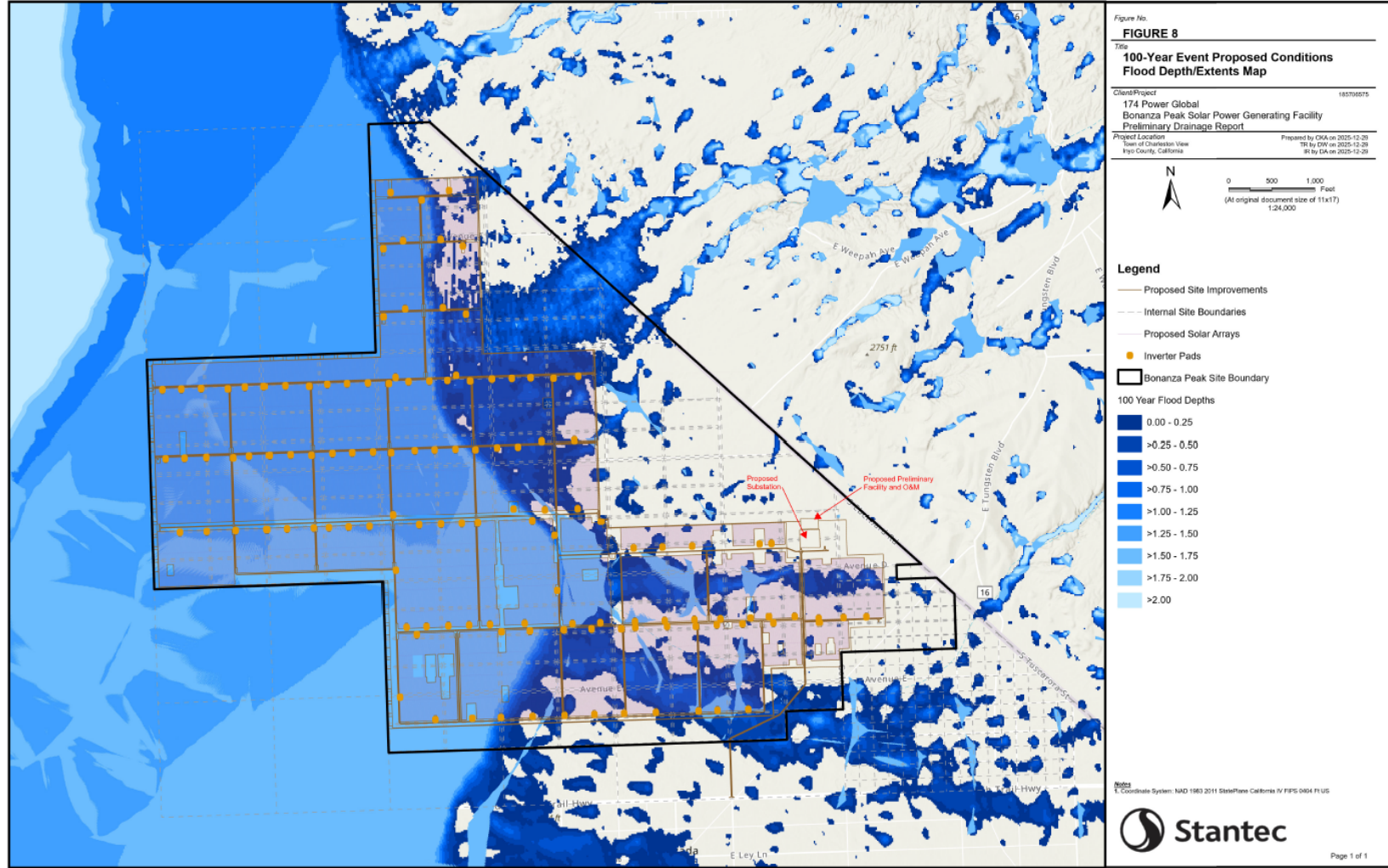


Figure 8. Proposed 100-Year Flood Depth Map



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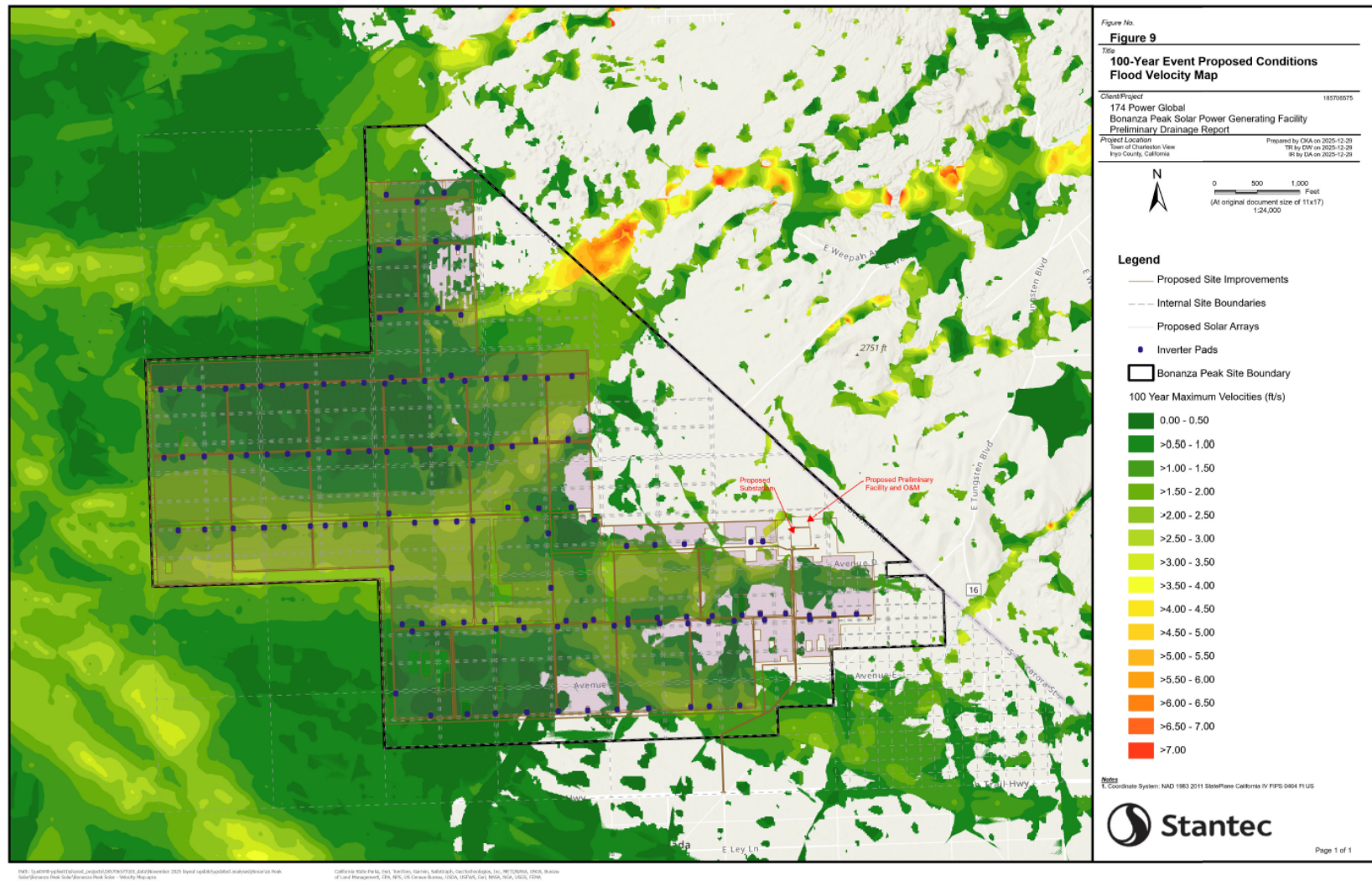


Figure 9. Proposed 100-year Flood Velocity Map



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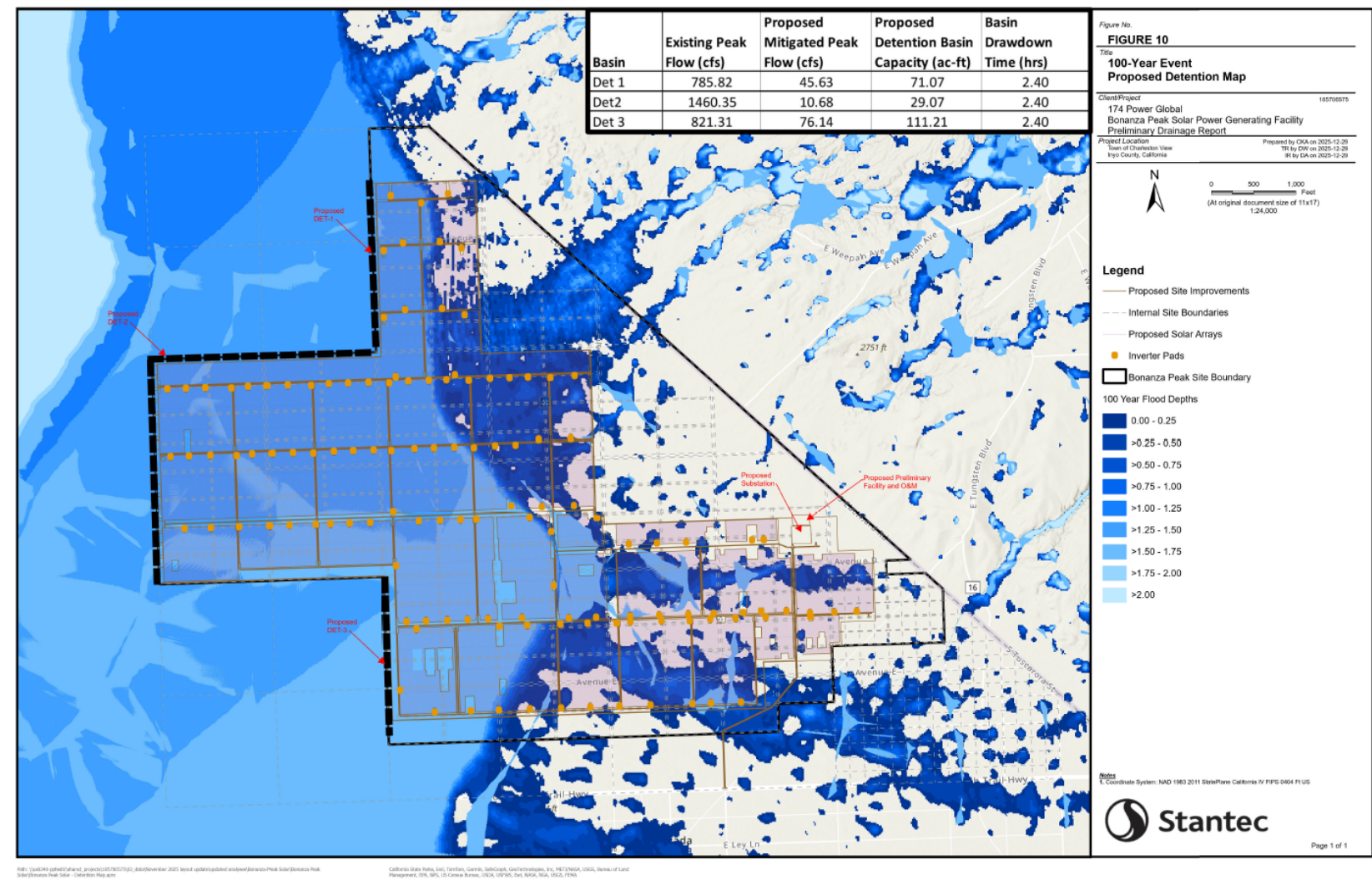


Figure 10. Proposed Detention Map



6 Scour Analysis

6.1 Scour Overview

There are several different types of scour identified in Hydraulic Engineering Circular 18 (HEC-18). For this Report, only local scour was reviewed which is scour that occurs when flow interacts with a structure, such as a pier or pile. **Figure 11**, from HEC-18, provides a graphical representation of the processes that occur during local scour. The important flow characteristics that affect local scour are velocity, depth upstream of the feature, the angle of flow velocity vector as it intersects the feature (angle of attack), and free surface or pressure flow. The feature geometry also plays a role in scour.

The type, dimensions, and shape of piles must be considered when analyzing scour. There are two different subcategories within local scour which are clear-water and live-bed scour. Clear-water scour occurs when “there is no movement of the bed material in the flow upstream of the crossing or the bed material being transported in the upstream reach is transport in suspension through the scour hole at the pier or abutment [pile] at less than the capacity of the flow”. Live-bed scour occurs when “there is transport of bed material from the upstream reach into the crossing” (Federal Highway Administration, 2012).

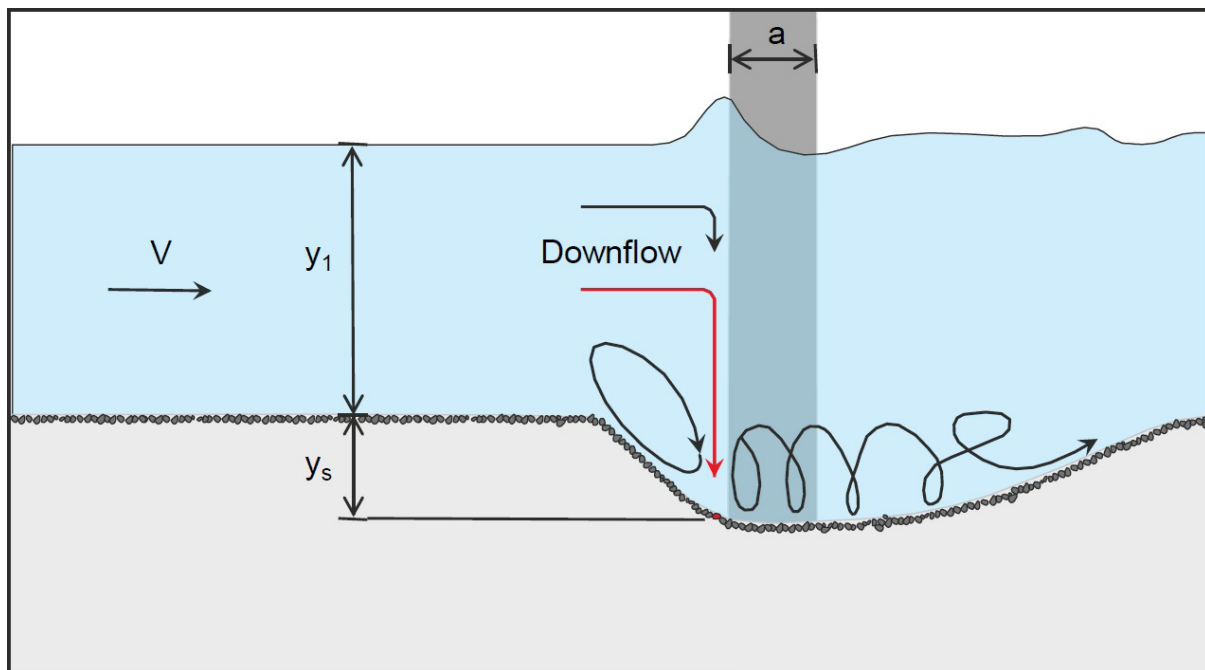


Figure 11. Illustrative Sketch of Local Scour (as shown in HEC-18)

Clear-water scour conditions were assumed for this analysis because there is not a channel bed upstream of the piles. The calculations were completed for an assumed W6x12 pile shape. HEC-18 provides an equation to calculate theoretical maximum scour depths and according to field research, frequently overpredicts the amount of observed scour, which encourages the design of conservative scour revetment (Federal Highway Administration, 2012).

The following sections present the model results and findings of this report. Closed depressions, perennial streams, ephemeral swales, drainage ditches, and drainage structures with limited conveyance capacities exist within and around the Project extents. These topographical features can cause areas of ponding and high velocities. Permissible velocity and shear stress values used for interpreting model results were based on the typical practice of post-construction seeding with short bunch grasses.

- Stantec's experience on other 2D hydrologic and hydraulic analyses for solar projects indicate that modeled inundation depths of greater than or equal to 2.0 feet should be viewed as areas of concern related to the installation of solar piles.
- Velocities higher than 3.0 fps exceed the permissible velocities for short native bunch grasses which roughly corresponds with the native vegetation that will be planted at the Project site in post-construction conditions. Modeled areas showing velocities exceeding 3.0 fps should be viewed as areas of concern for maintaining vegetation.
- Theoretical maximum scour depth calculated results less than 1.0 foot have been omitted from the provided figures. Scour depth less than 1.0 foot is manageable and does not inhibit the functionality of the Project. Furthermore, the calculated scour shown in the results are theoretical scour depths based on the hydraulic forces which results in a suitably conservative estimate of anticipated scour.

6.2 Calculated Scour Depth

The simulated maximum scour depths for the 100-year events are presented in **Figure 12**. Maximum depth, maximum velocity, and maximum scour depth are all parameters that determine the usability of the site. Stantec used guidance defined by the USACE and FEMA to conduct the analyses that were used to develop the figures. The intent of the scour depth analysis is to inform the client of areas of concern where regional drainage and floodwaters could impact PV asset installation, access road construction, and electrical distribution infrastructure locations. The 100-year maximum scour depths shown in **Figure 12** reflect the areas of greatest concern where the Client should consider the incorporation of additional scour protection measures or relocation of the proposed PV assets out of these high-risk areas in final plans for the Project. These areas generally correspond with regional drainage features in the model domain such as ephemeral drainageways, perennial streams, closed depressions, and drainage ditches. While some scour impact can be mitigated by regrading development areas and relocating drainage features in strategic locations away from the PV assets, additional details of potential scour protection techniques that can be pursued are described in Section 6.3.

6.3 Scour Protection

Scour protection will be based on the maximum depth of scour and the level of protection desired. The following sections discuss commonly used scour protection measures. Based on the model results from the existing conditions analysis, there may be localized acceleration (increased velocities) and shear stresses that form due to the construction of the piles. These localized impacts are not considered in this study.



6.4 Gravel/Cobbles and Vegetation

For areas with minimal scour potential, low velocities, and low shear stress, the level of protection provided by low-cost alternatives may be sufficient. These low-cost alternatives include providing scour protection using small gravel/cobbles or vegetation. **Table 10** Permissible Shear Stress and Velocity of Soils, Gravel/Cobbles, and Vegetation Adapted from *Fischenich, 2001*, is adapted from *Stability Thresholds for Stream Restoration Materials* (Fischenich, 2001), and provides the permissible shear stress and velocities for a variety of soils, gravel/cobble diameters, and vegetation types. When the shear stress and velocity adjacent to the pile are modeled at less than the permissible shear stress and velocity, there is minimal scour potential.

Table 10. Permissible Shear Stress and Velocity of Soils, Gravel/Cobbles, and Vegetation Adapted from Fischenich, 2001

CATEGORY	SUB-TYPE	PERMISSIBLE SHEAR STRESS (lbs/ft ²)	PERMISSIBLE VELOCITY (ft/s)
Soils	Fine colloidal sand	0.02-0.03	1.5
	Sandy loam (noncolloidal)	0.03-0.04	1.75
	Alluvial silt (noncolloidal)	0.045-0.05	2
	Silty loam (noncolloidal)	0.045-0.05	1.75-2.25
	Firm loam	0.075	2.5
	Fine gravels	0.075	2.5
	Stiff clay	0.26	3-4.5
	Alluvial silt (colloidal)	0.26	3.75
	Grade loam to cobbles	0.38	3.75
	Graded silts to cobbles	0.43	4
	Shales and hardpan	0.67	6
Gravel/Cobbles	1-inch	0.33	2.5-5
	2-inch	0.67	3-6
	6-inch	2.0	4-7.5
	12-inch	4.0	5.5-12
Vegetation	Class A Turf	3.7	6-8
	Class B Turf	2.1	4-7
	Class C Turf	1.0	3.5
	Long Native Grasses	1.2-1.7	4-6
	Short Native and Bunch Grass	0.7-0.95	3-4



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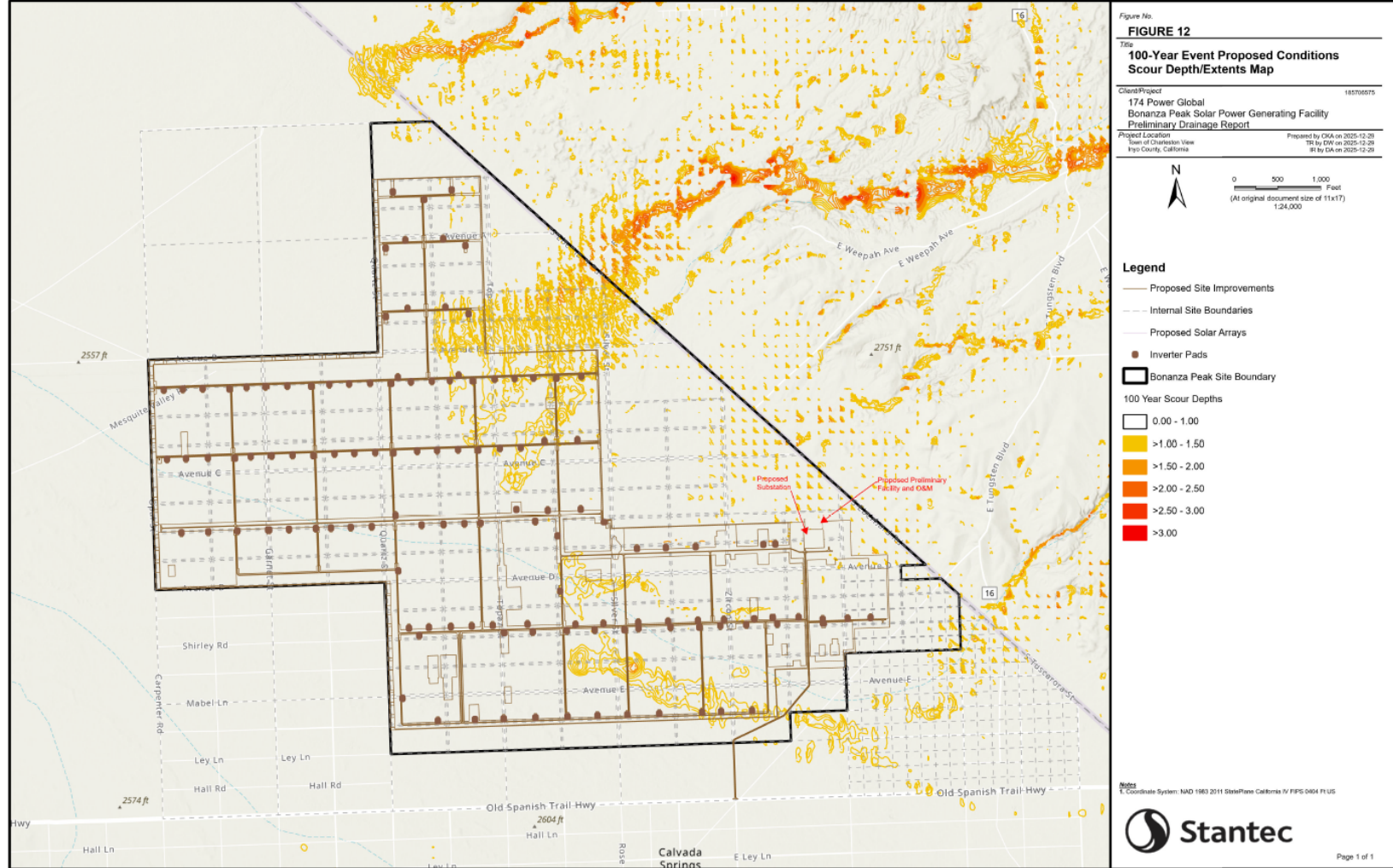


Figure 12. Proposed Scour Depth Map



6.5 Riprap Revetment

Riprap could be used as an effective countermeasure to protect piles from local scour to increase the buildable footprint for the PV assets at locations that are deemed to have water depth, velocities, shear stresses, and/or scour potential that exceed the cautionary thresholds discussed in the preceding sections. The marginal cost of the additional riprap revetment would need to be calculated against the additional expected lifetime revenue increase from the increased number of PV assets installed over the buildable area. Riprap provides significantly more protection than permissible shear stresses from other methods of protection. Permissible shear stresses and velocities associated with riprap are shown in **Table 11**.

Table 11. *Permissible Shear Stress and Velocity of Riprap, Adapted from Fischenich, 2001*

CATEGORY	SUB-TYPE	PERMISSIBLE SHEAR STRESS (lbs/ft ²)	PERMISSIBLE VELOCITY (ft/s)
Riprap	d ₅₀ of 6-inch	2.5	5-10
	d ₅₀ of 9-inch	3.8	7-11
	d ₅₀ of 12-inch	5.1	10-13
	d ₅₀ of 18-inch	7.6	12-16
	d ₅₀ of 24-inch	10.1	14-18

6.6 General Scour Protection Summary

Based on the level of protection (i.e., design event) desired by Cambria PV I, LLC, scour protection recommendations may differ. Regardless of the level of protection desired, the following recommendations apply:

1. Restoration using native and non-native grasses or small cobbles/gravel should provide sufficient scour protection for most arrays located outside of the ephemeral creeks and ditches and their floodplains.
2. If possible, PV assets should be built outside of the ephemeral creeks, swales, ditches, closed depressions, and floodplains.
3. If it is not possible to relocate planned PV assets, riprap revetment design should be provided as set forth in the Federal Highway Administration (FHWA) Hydraulic Engineering Circular No. 23 (HEC-23).
4. Localized acceleration (increasing velocities) and shear stresses that form due to the construction of the piles may require additional scour protection measures.

7 Debris Yield

7.1 Methodology

The USACE Los Angeles District Debris Method (Gatwood et al. 2000) consists of a set of predictive equations expressing the single event unit debris yield of a watershed as a function of physiographic, hydrologic, and meteorologic parameters. These predictive equations were developed by multiple regression analyses of single event debris data observed in the San Gabriel Ranges of southern California.

As defined in this method, the “total debris yield” is the total debris outflow from a watershed measurable at a specific concentration point for a specified event. It may include clay, silt, sand, gravel, boulders, tree stumps, and other organic materials. The “debris production” is the gross erosion within a watershed while the “debris yield” is the quantity of debris actually delivered to a concentration point of interest.

The entire debris production of the watershed may not necessarily reach its outlet because it is stored temporarily within the watershed due to the lack of transporting capacity of the conveyance system.

There are five empirical equations that were derived on the basis of watershed size ranging from 0.1 to 200 square miles. The multiple regression analyses indicated that the unit debris yield (DY) for a watershed is highly correlated with the following basin parameters: relief ratio (RR) analogous to watershed slope, drainage area (A), unit peak flow (Q), and the non-dimensional fire factor (FF).

Equation 2 is usually applied to drainages 3 to 10 square miles in area. Equation 1, which is a function of precipitation rather than runoff, is used for basins 0.1 to 3; however, if frequency discharge information is available, Equation 2 may be used for areas less than 3 square miles.

Equation 1 was applied herein to the drainages of interest less than 3 square miles in size since frequency discharge information was available; thus, Equation 1 was applied to every subbasin:

$$\log DY = 0.65 \log P + 0.62 \log RR + 0.18 \log A + 0.12 FF$$

where:

DY = unit debris yield (yd³/mi²), RR = relief ratio (feet/mile),

A = drainage area (acres),

FF = non-dimensional fire factor

P = max 1-hour precipitation (inches, taken two places after the decimal point, times 100)



7.2 Limitations

The general limitations related to the applications of the USACE Los Angeles District Debris Method in the prediction of debris yield are as follows: (1) geographic constraints, (2) drainage area constraints, (3) topographic constraints, (4) frequency constraints, and (5) input constraints. The frequency and input constraints pertain to small events less than 20-percent annual chance and low runoff or precipitation.

Since the recurrence interval in this study is 100-year, only the geographic, drainage area and topographic constraints remain. This method is intended to be used for the estimation of debris yield mainly from coastal-draining mountainous watersheds located in southern California. Since the predictive equations were derived from data observed in the San Gabriel Range, the use of these equations for watershed conditions different from those of the San Gabriel Range must be specifically addressed. The method is applicable only to watersheds with areas ranging from 0.1 to 200 square miles and with a high proportion of their total area in steep, mountainous terrain. The use of this method to compute debris yields for watersheds in mild-sloped valley areas with a high percentage of piedmonts and alluvial fans or valley fill areas may result in estimates that are higher than actual yield. If the sediment transport capacity is less than the statistical debris method results, the sediment transport capacity governs the debris yield.

7.3 Adjustment-Transposition (A-T) Factor

There are four different techniques used to find the A-T factor of a given watershed. As there were no records available for the regional and onsite watersheds Table B-1 in the USACE Los Angeles District Debris Method was used to find each watershed's A-T factor. A weighted average of all four subfactor groups were used to get the most accurate A-T factor.

7.4 Fire Factor

To determine the Fire Factor for the watersheds, **Figure 2** in the Los Angeles District Debris Method was used. It is known that it has been ten or more years since a wildfire has occurred in the area, which correlates to a fire factor of 3 for regional and onsite watersheds.

7.5 Computational Results

The Debris yield calculations were performed for all proposed pads that were within the onsite watersheds, bypassing all debris and sediment offsite. The debris yield was used to calculate the bulked volume, and the bulking factor is used to calculate the peaked bulk flow rate. The bulked flow rate was used to help design the channel needed in order to have a max depth of 3 feet with 1 foot of freeboard.

Consistent with the Detention Basin Design Criteria for San Bernardino County (SBC 1987), local detention basins were sized to capture the supplemental debris yield volume plus an additional 20% in volumetric capacity for maintenance purposes. An additional one foot of freeboard is also included in the preliminary sizing.



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The debris yield and bulking factor results can be seen in **Table 12**. These debris yield volumes have been incorporated into the detention basin sizing.

Table 12. Debris Yield Volumes

WATER-SHED	DRAINAGE AREA (sq.mi)	RELIEF RATIO (ft/mi)	FIRE FACTOR	A-T FACTOR	MAX 1 HR PRECIP (in)	Qp (cfs)	qp (cfs/sq.mi)	FLOOD VOLUME (ac-ft)	DEBRIS YIELD (ac-ft)	BULKED VOLUME (ac-ft)	BULKING FACTOR	Qp BULKED (cfs)
Det 1	61.67	563.47	3.00	0.7	1.50	3,503	56.81	71.07	0.27	100.40	1.00	71.26
Det 2	61.67	563.47	3.00	0.7	1.50	3,503	56.81	29.07	0.27	100.40	1.00	29.15
Det 3	134.66	219.21	3.00	0.7	1.50	1,674	12.43	111.21	0.56	111.77	1.01	111.77



8 Conclusions and Recommendations

8.1 PV Inverter Island Pads, Electrical Substation Pad, and O&M Building Pad

The proposed grading and drainage system for the Project avoids channelization of existing flows, and only proposes earthwork fills at the PV Inverter Island Pads, the Substation Pad, and the O&M Building Pad to maintain the electrical equipment elevated above the flood plain. These fill areas are to be protected with rip rap revetment along the exposed edges where velocities exceed 2 ft per second and scouring depths exceed 18 inches.

8.2 Solar Panel Piles and Access Roads

The remainder of the Project Area is not being modified, and the proposed solar panel piles and access roads shall be installed atop the existing ground with only the following modifications:

- Increasing pile heights/embedment for the solar panel piles in the array blocks across the eastern portion of the site (for the scour areas shown on **Figure 12**) to mitigate the scour depths and velocities in these particular areas.
- Installing rip rap at the base of piles of PV assets where the modeling shows areas of higher potential scour (**Figure 12**); and
- Upgrading access road structural sections where the modeling shows areas of higher potential scour (**Figure 12**).

8.3 General Discussion

The Project Area is subject to flooding from desert washes. Areas subject to desert wash flooding are considered more hazardous than areas subject to riverine conditions and generally require a higher degree of site engineering. Based on the flooding maps presented above, flood constraints for the development of the proposed project pads include:

- **Uncertain flow paths of desert washes.** Project design should reflect the potential for upstream flow paths to change course.
- **Debris or mud flows.** Desert washes may have the potential to entrain large amounts of sediments and debris. The elevation of debris entrapment deemed the potential for debris flows as medium.
- **Flow concentration.** Highly erodible soils may be adversely impacted if sheet flow is concentrated. The Project design should mitigate for impacts from flow concentration. Particular care should be applied to the design of any road network or obstructions to natural desert washes that are not oriented perpendicular to the primary flow direction.
- **Large Storm Events.** Pedestals and foundations are the equipment pads and panels should be raised at least one foot above the said anticipated 100-year flooding depths. Deeper foundations should be designed to account for erosion where velocities exceed 2 feet per second.



The grading of access roads can lead to the concentration of runoff and subsequently result in localized erosion along and across these roads, and in some cases, trigger the formation of flow-cut watercourses downstream. Local scour can occur around installed obstructions, e.g. panel supports and equipment pads, if exposed to concentrated runoff.

Added impervious cover and storm water conveyance improvements associated with planned developments are the typical drivers of impacts associated with storm water runoff. However, the development of the Project will likely add less than 8 percent of impervious cover. For this particular type of site development, changes in flood patterns, increases in flow intensity, and increases in runoff yield are typically related to any combination of changes in landform, soil composition, and soil compaction as a consequence of grading/construction activities.

Changes to soil compaction conditions can influence runoff development. An increase in soil compaction will either initiate or increase runoff development if the precipitation exceeds the infiltration capacity during a storm sequence. Unless the soil composition has a low baseline infiltration capacity, limiting increases in compaction will assist in limiting the mitigation needs associated with increased runoff volume.

8.4 Model Limitations and Assumptions

The HEC-RAS model and results discussed in this Report are not exact simulations of the processes occurring in nature, rather they are approximate representations of natural processes based on sets of equations simplifying the natural system and water cycle and making use of measured and collected data. The accuracy of these models is limited to the quality of the input data (e.g., accuracy of the pipe inverts), the specific point-in-time when the data collection (e.g., as watersheds develop imperviousness may increase), and the empirical mathematical relationships describing the water cycle. The data collected and used in these models reflects a level of effort commensurate with the purpose of the study. Calibration of hydrology, hydraulics, or water quality was not a part of this scope of work and therefore model accuracy is limited to literature values used for model inputs.

The 2D HEC-RAS modeling accounts for portions of the total precipitation that is infiltrated and that becomes runoff within the hydrologic modeling routine. Precipitation that is infiltrated is essentially subtracted from the precipitation hyetograph when the model calculates the quantity of water that develops as runoff. The hydraulic modeling does not model infiltration that may occur as runoff moves from grid cell to grid cell. Therefore, closed depressions and other topographic impediments with insufficient or no outlet show maximum inundation elevations that do not take infiltration into account and remain in the model as ponded water. Areas of maximum inundation should be viewed as areas of concern and not as actual observed depths.

The model results presented above reflect simulations of proposed conceptual site plan, and during final engineering the actual construction configuration and alignment of the piles may further increase or decrease the scour from that shown.



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