Appendix G

Water Resources Studies

Storm Water Hydrology Report for Solar Farm Layout A

Desert Sunlight Solar Farm – Alternative A

Storm Water Hydrology Report: Hydrologic, Hydraulic, Sediment Transport and Scour Analyses

Project Site:
Desert Sunlight Solar Farm
Riverside County, California

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AECOM Project No. 60131167 (114785)

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1 EXECUTIVE SUMMARY

AECOM has conducted hydrologic, hydraulic, sediment transport and scour analyses of storm water for Solar Farm Site – Alternative A of the First Solar, Inc. Desert Sunlight Solar Farm (DSSF or Project). The objectives of this Storm Water Hydrology Report (Report) are:

1. Establish design basis for the DSSF solar farm (Alternative A) improvements and infrastructure from a conservative (100-year) storm water event.
2. Verify that a low impact development measure (decompaction) with an additional measure will mitigate the hydrological impact to the upstream and downstream properties from the DSSF solar farm (Alternative A) improvements and infrastructure for a 10-year storm water event;

The significant results of the modeling determined that:

1. Results of the hydrologic analysis for the DSSF development indicated that implementing decompaction of the areas between the panels will reduce the post development hydraulic conditions to within +/-5% of the pre-development hydraulic conditions. An additional on-site mitigation measure such as basins with rip-rap protection, check dams or strip detention basins can be implemented to retain the remaining excess total off-site storm water volume increase. Please note that the accuracy of the model is approximately +/- 5% and so the differences (i.e. within 5%) calculated by the model are within this range.
2. Results of the hydrologic analysis for the DSSF post-development grading design without the addition of mitigation measures indicated that, in general, storm water off-site peak flow rates and volumes increased 6.7% and 5.5%, respectively for the 10-year storm event. On-site velocities increased 19.4% and flow depths decreased 7.1%, as compared to the pre-development existing conditions for the 10-year storm event.
3. Results of the hydrologic analysis for post-development design that only includes a decompaction mitigation measure indicated that the storm water off-site peak flow and volume increased 2.6% and 2.5%, respectively for the 10-year storm event. On-site velocity increased 19.4%and peak flow depth decreased 7.1%, as compared to the existing conditions for the 10-year storm events. The additional storm water peak volume is reduced by decompaction of soils, which is the most significant measure to mitigate post-development conditions to within +/- 5% of the pre-development conditions.
4. Results of the hydrologic analysis for post-development design that only includes a rip-rap mitigation measure found that for the 10-year storm event, storm water total outflow volume and peak flow depth increased, resulting in decreases in the peak flow and peak velocity, compared to the pre-development existing conditions. The storm water total volume and depth increased 5.5% and 7.1%, for the 10-year storm event. The peak flow and peak velocity decreased 3.0% and 6.5%for the 10-year storm event.
5. The addition of mitigation measures such as basins with rip-rap protection, check dams, or strip detention basins to the DSSF development in addition to decompaction, will address excess post-development hydraulic impacts that are not addressed by decompaction. These additional measures are based on implementing storm water best management practices and have not been rigorously modeled, however they would be designed to retain excess total off-site storm water volume. The intent of an additional mitigation measure is to reduce overall flow depths, velocities and outflow volume by detaining run-on storm water volume. The additional measures would also be successful at reducing potential increases in sediment transport and would be designed to retain the excess total volume capacity which is on the order of 50 ac-ft for the 10-year storm event.
6. Results of the sediment transport analysis for post-development determined that the average degradation for the 100-year and the 10-year storm event within the project site does not change (the difference is 0.0%) for future conditions. The average degradation depth for the 10-year storm would be 0.01 feet (i.e., general scour).

7. Results of the total scour analysis for post-development found that the average on-site scour depth would be 0.8 to 1.3 feet at the base of the PV supports for the 100-year storm, depending on the angle of flow to the supports. Placement of riprap will provide a less significant benefit to mitigate for additional runoff. However, riprap placed at the base of each support structure will help reduce the effects of local scour and lower storm water runoff velocities.
8. Results of the qualitative fluvial geomorphologic analysis indicates existing areas of relatively inactive sediments characterized by desert pavement and more active areas consisting of finer sand and gravel. The changes to the site resulting from Project development will create an area that has consistent compaction, soil type and grading compared to existing conditions. It is anticipated that these changes will create a geologic environment conducive to the formation of shallow channels up to two feet or less in depth (i.e. long-term scour). This long term scour can be mitigated by periodic monitoring to identify changes to the site grading and maintenance activities as/if needed to restore design conditions.
9. Along with the mitigation measures, a Monitoring and Response Plan will be prepared and submitted to the BLM. The Monitoring and Response Plan will indicate the procedures that will be followed to mitigate potential impacts to the site structures, storm water infrastructure or site grading that can occur from local scour, sediment transport and long term degradation (i.e. fluvial geomorphology) during the operation of the DSSF.

2 INTRODUCTION

AECOM has conducted a hydrologic, hydraulic, sediment transport and scour analyses of storm water conditions within and around Solar Farm Site – Alternative A of the Desert Sunlight Solar Farm (DSSF or Project) for First Solar, Inc. The DSSF is a future 550 MW solar photovoltaic (PV) electric generating facility. The Project is located in Riverside County on public lands under the jurisdiction of the Bureau of Land Management (BLM). This report provides a site description which includes an overview of the Project and its environment (climate, geology, land-use/soil-type, drainage areas), and a specific section on fluvial geomorphology. A quantitative hydrologic, hydraulic and sediment transport analysis was conducted using several computer models. In addition, scour evaluation was performed to assess scour potential around the PV support structures.

The objectives of the Report are:

1. Establish design basis for the DSSF solar farm (Alternative A) improvements and infrastructure from a conservative (100-year) storm water event.
2. Verify that a low impact development measure (decompaction) with an additional measure will mitigate the hydrological impact to the upstream and downstream properties from the DSSF solar farm (Alternative A) improvements and infrastructure for a 10-year storm water event;

The 100-year storm was used to focus on the storm water impacts on the development, and 10-year storm was used to evaluate impacts of the development on the storm water and sediment transport characteristics of the site. During a100-year storm event, the magnitude of the run-off is significant resulting in highest potential of structural impact; however, the difference in run-off between pre and post-development is higher during the 10-year storm, which is more probable to occur during the design life of the project. During the 10-year storm event, the percent difference is not overwhelmed by the shear amount of run-off volume associated with 100-year event, which quickly saturates the ground and effect of infiltration capacity diminishes. Therefore, using the 100-year event to evaluate storm water impacts on the development and the 10-year storm event to evaluate post-development stormwater and sediment transport characteristics represents a conservative approach to understanding the potential for stormwater impacts both on the Project and to the upstream and downstream properties.

The storm water analysis was based on the Riverside County Flood Control and Water Conservation District Hydrology Manual, which uses a 100-year storm event under antecedent moisture conditions (AMC) II criteria for the design basis criteria. A 10-year storm event was analyzed in addition to the 100-year storm event in order to evaluate the more probable event that will be experienced in the Project's lifespan.

The Report presents the results of a detailed hydrologic analysis and hydraulic/sediment-transport model of the DSSF for the existing (i.e., pre-development) conditions. It also includes the results of a watershed analysis that encompasses areas immediately upstream and downstream of the DSSF to determine and evaluate the Project's potential on-site and off-site peak flows during design storm events. The detailed analysis calculated off-site peak flow rate, off-site peak flow volume, maximum and average on-site peak flow depth, and on-site peak and average flow velocity. The off-site peak flow rate and volume are determined at the downstream boundary of the model, which is approximately1/4 mile south of the southern boundary

This report includes the results of the initial hydraulic analysis that modeled the pre-development conditions and compared them to the post-development conditions based on the Project's grading design submitted as part of the Project Description on March 19, 2010. The primary concepts relating to storm water characteristics that were incorporated into this DSSF grading design were contour grading. The intent of the contour grading concept is to smooth the existing surface into consistent graded slopes. Existing slopes on-site will be maintained such that the average cut/fill over the entire site is approximately 5-inches. The results of this comparison are discussed in Section 4.

The hydraulic analysis models the post-development conditions based on the Project's grading design that incorporates a decompaction mitigation measure. The intent of the de-compaction concept is to restore the soil infiltration capacity to the pre-development state. De-compaction will be applied to the areas between the rows of PV panels that were compacted during PV support structure and panel installation. The results of this comparison are discussed in Section 5.

Section 5.4 also includes discussions of other mitigation measures that are proposed to be in addition to the decompaction mitigation measure. These additional mitigation measures are recognized to have beneficial effects to the Project storm water characteristics, but are not as effective as the decompaction mitigation measure. Therefore these additional mitigation measures are discussed in qualitative terms.

Section 5.5 discusses the effect of the Project development on the storm water flows in Pinto Wash.

Sediment Transport characteristics comparing the pre-development conditions and post-development conditions based on the Project's grading design is presented in Section 6.

Fluvial geomorphology for the post-development conditions based on the Project's grading design is discussed in Section 7.

Local scour at the base of the PV solar panel supports for the post-development conditions based on the Project's grading design is discussed in Section 8.

3 PROJECT SITE DESCRIPTION

The DSSF is located on a vacant, largely undeveloped and relatively flat tract of land in the Chuckwalla Valley area of the Sonoran Desert in eastern Riverside County, approximately four miles north of the rural community of Tamarisk Park and six miles north of the I-10 freeway and the rural community of Desert Center. The inactive Eagle Mountain Mine and the boundary of Joshua Tree National Park are located approximately 1.5 miles west and 1.4 miles east of the DSSF, respectively. The future DSSF location is shown on Figure 1.

Eagle Mountain Road, Kaiser Road, a paved road, and Eagle Mountain Railroad run from the Eagle Mountain Mine along the southwest portion of the DSSF before continuing south. Because the mine is no longer in operation, the various local roadways are lightly traveled.

Three existing transmission lines pass through the DSSF site. An existing 230-kV transmission line and a 33-kV distribution line, both owned by the Metropolitan Water District of Southern California (MWD), run along Power Line Road and traverse the DSSF.

3.1 Proposed Development

The DSSF, as proposed by First Solar, will be a solar photovoltaic (PV) energy generating facility producing 550 Megawatt AC (MWAC). The solar farm will occupy approximately 4,090 acres and includes the solar arrays, an on-site substation, access roads, a monitoring and maintenance facility, and other support facilities.

The First Solar PV modules, of which there will be a total of approximately 8.4 million on-site, are mounted on module framing assemblies made of steel, each holding 16 modules and measuring approximately eight (8) feet wide by 16 feet long. PV module assemblies are attached at an angle to vertical steel piles that are spaced eight (8) feet center-to-center and are driven into the ground to a depth of four (4) to seven (7) feet below grade. Each steel pile is a single W6x9 "I" beam. Once mounted, the front of each PV module assembly will be approximately 1.5 feet above grade, while the rear will be approximately five (5) to six (6) feet above grade.

The PV modules are electrically connected by wiring harnesses running along the bottom of each assembly to combiner boxes that collect power from several rows of modules. The combiner boxes feed DC power from the modules to the Power Conversion Station (PCS) via underground cables. The inverters in the PCS convert the DC electric input into AC electric output and the isolation transformer steps the current up for on-site transmission of the AC power to the PV combining switchgear (PVCS). The PVCS collects the power for transmission to the Substation.

3.2 Climate

The National Oceanic and Atmospheric Administration (NOAA) Atlas 14, which was used to estimate precipitation frequency for the hydrologic model, defines southwestern California as a semi-arid region. The Riverside County Hydrology Manual describes the inland valley and desert areas as extremely hot and dry during the summer months and moderate during the winter. The mean seasonal precipitation is three inches in the eastern desert regions and 35 to 40 inches in the San Bernardino and San Jacinto Mountains. There are three types of storms within the region: (1) general winter storms, (2) general summer storms and (3) high intensity thunderstorms. General winter storms originate as tropical cyclones (warm Pacific air masses) that occur in the late fall or winter months. High rates of precipitation occur over the interior mountain ranges but precipitation decreases rapidly over the desert areas. General summer storms can result in heavy precipitation and have durations of several days. These typically occur between the months of July and September as a result of tropical air masses from either the Gulf of Mexico or the South Pacific Ocean. Thunderstorms that generate extremely high precipitation rates for short durations can occur at any time of year.

3.3 Geology

Regional and site surficial geology are discussed in the 2007 "Phase 1 Geologic Reconnaissance Report" prepared for the Project by Eberhart/United Consultants (EUC). The site is located within the southwestern portion of the Mojave Desert Geomorphic Province of southern California. The San Andreas Fault defines the southwestern boundary of the Geomorphic Province while the Garlock Fault forms the boundary to the north. The Mojave is a broad interior region of isolated mountain ranges separated by expanses of desert plains. It has an interior enclosed drainage and many playas. The proposed DSSF site is located in the Chuckwalla Valley, which is formed from multiple alluvial fans disseminating from the Eagle Mountains in the west and the Coxcomb Mountains in the east. The Pinto Wash bisects the valley and forms the eastern boundary of the solar farm site.

3.4 Land Use and Soil Type

Available data indicates that land use activities at the DSSF site have remained relatively consistent over the past 30 to 40 years. Several small agricultural plots have been established in the vicinity of the site with the use of irrigation. The site itself has remained as largely undeveloped desert with sparse vegetation.

Field reconnaissance by EUC in 2007 investigated the surficial sediments at the site. Two distinct sediment types were present, one associated with areas of desert pavement and the other with more active wash sediments. EUC collected samples with a hand auger at three locations within the proposed DSSF site. Table 1 below summarizes the sediment characteristics.

Table 1. Surficial Sediment Summary

Sample ID	Location	Depth (ft)	D_{50} (mm)	Description
A	Southwest	-	-	Well graded gravel (desert pavement) grading into well sorted sand with gravel
C	Northwest	0 to 0.5	9.5	Well graded gravel (desert pavement) grading into well sorted sand with gravel
C	Northwest	0.5 to 1.5	0.8	Well sorted sand with gravel
J	South	2.0 to 4.5	1.5	Well graded sand with gravel

3.5 Drainage Areas and Extent of the Modeling

The major drainage in the vicinity of the DSSF is the Pinto Wash. The Pinto Wash is located along the eastern boundary of the DSSF, continues southeast across undeveloped land, and drains into Palen Dry Lake to the east of the DSSF. Figure 2 shows a map of the model extents for both the hydrologic and hydraulic models. The basin delineation and model extents were developed utilizing automatic basin delineation tools available in the U.S. Environmental Protection Agency's (EPA) BASINS software. Elevations from the United States Geological Survey's (USGS) National Elevation Dataset were used for development of the model hydrology, which is discussed further in the following section.

4 HYDROLOGIC ANALYSIS

A two-dimensional (2D) model was constructed to simulate flow patterns and sediment transport within the DSSF. The hydrologic component of the 2D model was developed in HEC-HMS, a product of the Hydrologic Engineering Center (HEC) within the U.S. Army Corps of Engineers. The hydrologic analysis was performed using AMC II conditions utilizing guidelines outlined in the Riverside County Flood Control and Water Conservation District Hydrology Manual. The hydrologic analysis was repeated for the 10-year storm event incorporating various mitigation measures.

4.1 Hydrologic Analysis

The Riverside County Manual refers to the NOAA Atlas 2 for rainfall data. However, NOAA has superseded this source with Atlas 14 in the Project area. The website associated with NOAA Atlas 14 can provide rainfall intensity-duration-frequency (IDF) curves for any location based on latitude and longitude. The approximate coordinates of the DSSF site were entered into the website to develop rainfall totals for the 100- and 10-year storm events. A rainfall distribution was not specified by Riverside County; therefore, the balanced distribution recommended by the San Bernardino County Hydrology Manual (August 1986) was used for the analysis.

The Soil Conservation Service (SCS) curve number methodology was used to estimate flows to the hydraulic model. Curve numbers ranging from 79 in upstream areas to 63 in downstream areas were used for delineated basins. These curve numbers reflect AMC II, or normal moisture, conditions as specified by the Riverside County Manual. An initial abstraction of 0.15 was used. Lag times were calculated using the curve number method.

Hydrologic information was entered into HEC-HMS, which was then used to generate flows to the hydraulic model. Figure 3 presents the rainfall hyetograph at the Project site and Figure 4 shows the estimated total storm water peak flow running onto the entire project site over time during the 100-year and 10-year storm events. A summary of the hydrologic analysis is contained below in Table 2.

Table 2. Hydrologic Analysis Summary

Parameter	Value	Value
Design Storm Frequency	100-year	10-year
Peak Rainfall Depth	0.72 inches in 5 minutes	0.31 inches in 5 minutes
Total Rainfall Depth	3.58 inches	1.96 inches

5 HYDRAULIC ANALYSIS

Flow and sediment transport within the study area were simulated using FLO-2D. FLO-2D is a two-dimensional model designed to simulate unconfined overland flows. The extents of the FLO-2D model are shown in Figure 2 and include Solar Farm Site – Alternative B as well as the Pinto Wash area immediately to the east. The northern and southern boundaries of the model were determined based on the path of water flow as per the USGS National Elevation Dataset. The upstream boundary extends approximately two miles upstream of the DSSF to establish flow patterns and sediment loads flow entering the site. The downstream boundary condition was set over half a mile downstream so that the downstream boundary condition would not affect flows on the Project site. FLO-2D model grid cells were set to dimensions of 200-feet by 200-feet.

Four configurations were analyzed: (1) existing conditions, (2) proposed or future (post-development) conditions, (3) proposed or future conditions with soil decompaction and (4) proposed or future conditions with rip-rap. Future conditions were modeled without stormwater mitigation measures and with the inclusion of a storm water mitigation measure in the form of either soil decompaction or rip-rap.

5.1 Inputs and Assumptions

Light Detection and Ranging (LIDAR) topographic survey data was collected within the DSSF. The LIDAR data was combined with USGS elevation data to populate the 2D model grid with elevations. These elevations represent the existing conditions of the site. For this analysis the same topographic data was used for both existing and proposed or future (post-development) conditions. Using the LIDAR data for both existing and future conditions will show the hydraulic changes at the project site as a result of grading and compaction by changing only the Manning’s roughness and infiltration parameters. The grading plan would not greatly affect the model elevations that are averaged within the 200 foot by 200 foot grid elements created in FLO-2D.

The FLO-2D model uses the Green-Ampt method to simulate ground infiltration. The parameters for the Green-Ampt method were calibrated using information from the hydrologic HEC-HMS model. HEC-HMS uses the Curve Number infiltration method. The volume of flow that should runoff the site was estimated in HEC-HMS. The hydraulic conductivity in FLO-2D was adjusted so that the correct volume of flow was generated in the FLO-2D model. A curve number of 63 (i.e. barren land) was used for the majority of the existing conditions. The areas classified as “barren land” represent areas containing existing wash. The areas of desert pavement that occur within the project site were assumed to have similar infiltration capacity as the dirt roads introduced for the future conditions (i.e. curve number 72). Earth Systems Southwest (ESSW) provided an estimate that suggests approximately 20-30 percent of the total project area is covered in moderate to strong desert pavement. Delineation of the desert pavement areas were done by EUC (EUC, 2007). AECOM reviewed EUC’s delineation against recent aerial images to confirm accuracy. This delineation is shown in Appendix E the mapped desert pavement area is approximately 30 percent of the project site. The infiltration capacity of desert pavement was assigned in the area shown. It should be noted that approximately 6 (six) percent of the project area is covered in weak desert pavement. This area will not be disturbed by the proposed development; the area will not be graded but will be mowed to remove vegetation. The properties of desert pavement are discussed further in Section 7.1, Fluvial Geomorphologic Assessment Methodology. A curve number of 72 (i.e. dirt roads) was used for future conditions to account for compaction and loss of vegetation within the DSSF site. Outside the project site the existing conditions assignment of 63 representing barren land was retained.

A Manning’s “n” value of 0.043 was used for existing conditions and was based on guidelines established by the USGS for developing Manning’s roughness coefficients in floodplains (USGS Water-supply Paper 2339). For the post-development conditions, the Manning’s “n” is reduced to 0.034, reflecting both the reduction in roughness due to smoothing the grade and removing existing vegetation and takes into account the increase in roughness due to the presence of the piles supporting the solar panels. See Appendix B for a detailed review of the Manning’s value assignments.

5.2 Results: Future Conditions

The results presented in this section show the future hydraulic conditions without stormwater mitigation measures. The FLO-2D model was simulated for a 48-hour period for the 100- and 10-year design storm events. Plots of peak storm water depth and velocity for both future and existing conditions were produced with the FLO-2D model results. To be conservative in terms of peak velocities, sediment transport was not taken into account during these simulations. In reality, when sediment transport (scour) takes place flow depth will increase and the peak velocities will therefore decrease. Sediment transport models were developed separately, the results of the sediment transport analysis can be found in Section 6. Sediment transport models were developed separately, the results of the sediment transport analysis can be found in Section 6. Figure 5 through Figure 16 present the results of the 2D model without the sediment transport module activated. The results included on these figures include the peak flow depth and peak velocity at each 200-foot by 200-foot cell for both existing and future conditions, as well as plots for the change in these values between the existing and future conditions.

As shown on Figure 6 the 100-year future conditions model indicates that the storm water peak flow depth would be less than 2.1 feet in the center of the DSSF and towards the east due to the Pinto Wash. In general, the modeling results demonstrate that there would be very little change (less than one tenth (1/10) foot of difference) in flow depth as a result of Project-related changes to the site. Figure 7 presents the difference in the storm water peak flow depth at each modeling cell for the post-development future condition as compared to the existing conditions.

The modeling results also demonstrate that there would be a slight increase in storm water peak flow velocities as a result of the changes to the Project site. Figure 10 presents the difference in the storm water peak velocity at each modeling cell for the future conditions compared to the existing conditions. This shows an increase in velocity of up to eight-tenths of a foot per second at certain locations within the DSSF.

The increase in velocity, combined with the increased runoff due to compaction, will have some impact on the downstream peak flows and volumes from the study area. A summary of the hydraulic analysis for the 100-year storm is contained in Table 3 below. In this table, “on-site location” essentially indicates the changes within the Project site and “off-site location” indicates the impacts to the areas immediately downstream of the DSSF site.

Table 3. Hydraulic Analysis Summary: 100-year

Parameter	Location	Existing Conditions	Future Conditions	Change
Peak Outflow	Off-site	24,811 cfs	26,253 cfs	1,442 cfs (5.8%)
Total Outflow Volume	Off-site	7,154 acre-ft	7,319 acre-ft	165 acre-ft (2.3%)
Maximum Peak Flow Depth	On-site	2.2 ft	2.1 ft	-0.1 ft (-4.5%)
Average Peak Flow Depth	On-site	0.8 ft	0.7 ft	-0.1 ft (-12.5%)
Peak Velocity	On-site	4.6 ft/s	5.4 ft/s	0.8 ft/s (17.4%)
Average Velocity	On-site	2.0 ft/s	2.2 ft/s	0.2 ft/s (10.0%)

The hydraulic model results of the 10-year storm can be found in Table 4, below. Figure 12 shows the grid element maximum flow depths and Figure 13 shows the change in flow depth from existing to proposed. The change in peak flow depth decreased one-tenth of a foot from existing to proposed conditions and the average flow depth remained the same. Maximum velocities at each grid element are shown in Figure 15 and the change in velocity is shown in Figure 16. Peak flow velocity and average velocities will increase as a result of development for the 10-year storm.

Table 4. Hydraulic Analysis Summary: 10-year

Parameter	Location	Existing Conditions	Future Conditions	Change
Peak Outflow	Off-site	5,376 cfs	5,738 cfs	362 cfs (6.7%)
Total Outflow Volume	Off-site	2,030 acre-ft	2,142 acre-ft	112 acre-ft (5.5%)
Maximum Peak Flow Depth	On-site	1.4 ft	1.3 ft	-0.1 ft (-7.1%)
Average Peak Flow Depth	On-site	0.4 ft	0.4 ft	0.0 ft (0.0%)
Peak Velocity	On-site	3.1 ft/s	3.7 ft/s	0.6 ft/s (19.4%)
Average Velocity	On-site	1.2 ft/s	1.3 ft/s	0.1 ft/s (8.3%)

Table 3 and Table 4 do not reflect storm water mitigation measures that will be incorporated into the final design of the DSSF. See Section 5.3 for the model results with incorporated LID design mechanisms.

5.3 Results: Future Conditions with Storm Water Mitigation Measures

The results presented in this section show the future hydraulic conditions with decompaction or rip-rap as stormwater mitigation. The FLO-2D model was simulated for a 48-hour period for the 100- and 10-year design storm events. Infiltration rates were adjusted to represent decompaction of the soil between the rows of the arrays. Plots of peak storm water depth and velocity for both future and existing conditions were produced with the FLO-2D model results. To be conservative in terms of peak velocities, sediment transport was not taken into account during these simulations. Sediment transport models were developed separately, the results of the sediment transport analysis can be found in Section 6.

The goal of the design is to minimize the change of hydraulics and sediment transport. Since the results of the future conditions modeling analysis (presented in Section 5.2) has not achieved this goal, additional storm water mitigation measures were modeled to determine the effect of each measure on the changes to post development hydraulic conditions. Low Impact Development types of storm water and erosion control measures including decompaction of the soil after array installation or placement of rip-rap were identified and modeled in order to reduce post-development hydraulic parameters.

5.3.1 Results: Future Conditions with Decompaction Mitigation Measure

The second mitigation measure modeled involves decompacting the soil after the arrays have been installed. Soil decompaction would be implemented between the rows of tables within each of the arrays. The decompaction operation will restore the infiltration to the pre-development original state. The intent of the decompaction mitigation measure is to increase the post-development soil infiltration that results in a lower total storm water outflow volume.

For the project areas located on existing desert pavement, the decompaction measure is not anticipated to restore the pre-development conditions. Project areas that are currently covered with desert pavement already have a low infiltration capacity. Although the decompaction measure is intended to increase post-development soil infiltration, the decompaction measure is not anticipated to significantly change the infiltration capacity as compared to pre-development conditions for desert pavement areas.

The values presented in Table 5 are the results from simulating decompaction of 37.3% of the total project site. This percentage was calculated based on the current array configuration and site layout that allows for approximately 9.4 feet of the area between rows to be decompacted with an allowance to minimize damage to the panels . Figure 17 shows the maximum peak flow depths, Figure 18 shows the change in maximum peak flow depth, Figure 19 shows the maximum peak velocity and Figure 20 shows the change in peak velocity. The change in total outflow volume was reduced from 165 to 76 acre-feet or a 1.1% increase from existing conditions when decompaction was considered.

Table 5. Hydraulic Analysis Summary: 100-year with Decompaction

Parameter	**Location**	**Existing Conditions**	**Future Conditions with Decompaction Measure**	**Change**
Peak Outflow	Off-site	24,811 cfs	26,070 cfs	1,259 cfs 5.1%
Total Outflow Volume	Off-site	7154 acre-ft	7,230 acre-ft	76 acre-ft 1.1%
Maximum Peak Flow Depth	On-site	2.2 ft	2.1 ft	-0.1 ft -4.5%
Average Peak Flow Depth	On-site	0.8 ft	0.7 ft	-0.1 ft -12.5%
Peak Velocity	On-site	4.6 ft/s	5.3 ft/s	0.7 ft/s 15.2%
Average Velocity	On-site	2.0 ft/s	2.2 ft/s	0.2 ft/s 10.0%

The 10-year decompaction simulation resulted in a change in total outflow volume of 50 acre-feet or a 2.5% increase from existing conditions. Figure 21 shows the maximum peak flow depths, Figure 22 shows the change in maximum peak flow depth, Figure 23 shows the maximum peak velocity and Figure 24 shows the change in peak velocity.

Table 6. Hydraulic Analysis Summary: 10-year with Decompaction

Parameter	Location	Existing Conditions	Future Conditions with Decompaction Measure	Change
Peak Outflow	Off-site	5,376 cfs	5,517 cfs	141 cfs 2.6%
Total Outflow Volume	Off-site	2,030 acre-ft	2,080 acre-ft	50 acre-ft 2.5%
Maximum Peak Flow Depth	On-site	1.4 ft	1.3 ft	-0.1 ft -7.1%
Average Peak Flow Depth	On-site	0.4 ft	0.4 ft	0.0 ft 0.0%
Peak Velocity	On-site	3.1 ft/s	3.7 ft/s	0.6 ft/s 19.4%
Average Velocity	On-site	1.2 ft/s	1.3 ft/s	0.1 ft/s 8.3%

The results presented in Table 5 and Table 6 do not include sediment transport functions.

5.3.2 Results: Future Conditions with Rip-Rap Mitigation Measure

The addition of rip rap to the final graded surface was identified as the first mitigation measure to reduce the hydraulic effects of proposed development at the DSSF site. Placing riprap on the final graded surface at the project site increases the Manning's roughness values for the post-development condition as well as protects the array supports from localized scour (See Section 8). This measure will counteract the reduction in the Manning's roughness from pre- to post-development conditions that occurs from vegetation removal during DSSF grading activities. The intent of this measure is to return the post-development roughness to the value of the existing conditions. The model assumes a 6-inch rip-rap, 108 ft across placed in every 200 ft cell (i.e. 54% of the project area), following the graded contours.

It is reasonable to assume that the placement of rip-rap would increase the Manning's roughness value across the DSSF site. If the overall site post-development roughness is increased by 0.005 (to a total value of 0.39) for the 100-year event, the change in flow depth and velocity from pre to post-development would be significantly decreased compared to post-development results without mitigation measures. Assuming that the placement of rip-rap increases the post-development roughness value to 0.39, the changes in peak outflow and total outflow volume from the pre to post-development conditions for the 100-year storm event would limit the change to less than 5%. Figure 25 shows the maximum flow depths for each grid element and Figure 26 shows the difference from existing conditions. The maximum peak flow depth increased by one-tenth of a foot and the average peak flow depth remained the same as existing conditions. Peak velocity and average velocity remained the same for existing and proposed conditions. Figure 27 shows the maximum velocities at each grid element and Figure 28 shows the change in velocity. Table 7, below, summarizes the hydraulic analysis for the 100-year storm using rip rap.

Table 7. Hydraulic Analysis Summary: 100-year with Rip Rap

Parameter	Location	Existing Conditions	Future Conditions with Rip-Rap Mitigation	Change
Peak Outflow	Off-site	24,811 cfs	24,954 cfs	143 cfs (0.6%)
Total Outflow Volume	Off-site	7154 acre-ft	7,317 acre-ft	163 acre-ft (2.3%)
Maximum Peak Flow Depth	On-site	2.2 ft	2.3 ft	0.1 ft (4.5%)
Average Peak Flow Depth	On-site	0.8 ft	0.8 ft	0.0 ft (0.0%)
Peak Velocity	On-site	4.6 ft/s	4.6 ft/s	0.0 ft/s (0.0%)
Average Velocity	On-site	2.0 ft/s	2.0 ft/s	0.0 ft/s (0.0%)

For the 10-year storm event, several iterations of models found that the Manning's roughness value would need to be increased by 0.023 to decrease the effect of development. These iterations resulted in a roughness value for the post-development conditions that was higher than the existing condition value. Figure 29 through Figure 32 show the results of the storm water modeling with a roughness value of 0.57. Table 8 shows the hydraulic results for the 10-year storm. Even by increasing the site roughness value to a value greater than existing conditions did not limit the change to less than 5% for pre to post-development conditions for the 10-year storm event.

Table 8. Hydraulic Analysis Summary: 10-year with Rip Rap

Parameter	Location	Existing Conditions	Future Conditions with Rip-Rap Mitigation	Change
Peak Outflow	Off-site	5,376 cfs	5,216 cfs	-160 cfs -3.0%
Total Outflow Volume	Off-site	2,030 acre-ft	2,142 acre-ft	112 acre-ft 5.5%
Maximum Peak Flow Depth	On-site	1.4 ft	1.5 ft	0.1 ft 7.1%
Average Peak Flow Depth	On-site	0.4 ft	0.4 ft	0.0 ft 0.0%
Peak Velocity	On-site	3.1 ft/s	2.9 ft/s	-0.2 ft/s -6.5%
Average Velocity	On-site	1.2 ft/s	1.2 ft/s	0.0 ft/s 0.0%

The results presented in Table 7 and Table 8 do not include sediment transport functions. In order to achieve a roughness of 0.039 for the 100-year future conditions approximately 54% of the project site would need to be covered in six (6) inch diameter rip-rap. The roughness value of 0.057 for the 10-year storm event cannot be obtained with six (6) inch rip-rap (See Manning's roughness calculations in Appendix B). Introducing rip rap will decrease the depths and velocities at the project site but rip rap as the only mitigation measure implemented, by itself does not provide the storage that would be required to decrease the outflow volume and outflow discharge. Additional mitigation measures can be implemented to further reduce the impact of the storm water outflows.

5.3.3 Discussion of Results: Future Conditions with Mitigation Measures

Decompaction of soils is the most significant measure to mitigate post-development conditions to within 5% of the pre-development conditions, by reducing added runoff. Decompacting the soil provides additional infiltration capacity which reduces runoff volume, peak flow rate, flow velocities and sediment transport. Placement of riprap provides a less significant benefit to mitigate post-development conditions to within 5% of the pre-development conditions. Increasing surface roughness (e.g. use of riprap) slows down the velocities, decreases sediment transport and increases flow depth.

Neither the rip-rap nor the decompaction measures alone will mitigate the post development conditions to within 5% of the pre-development hydraulic conditions. A combination of these two mitigation measures and/or addition of further mitigation measures should be considered to achieve a change from pre to post development conditions of less than 5%.

5.3.4 Discussion of Additional Mitigation Measures

An additional mitigation measure such as retention basins can be implemented to address specific post-development hydraulic characteristics that remain after implementation of the decompaction measure. These retention basins could be located along the upstream western boundary of the project site to intercept run on storm water flows. The intent of this measure is to reduce overall flow depths, velocities and outflow volume by retaining run-on storm water volume. They will also reduce sediment transport within the project site. Due to the size of the grid elements in FLO-2D (200 foot by 200 foot) an accurate representation of the basins cannot be distinguished in the model. However, it can be assumed that the basins can be designed to retain the excess total storm water volume. Once the basins are designed, their retention capacity volume can be subtracted from the total outflow volume of any of the simulations. Retentions basins would be designed to retain the excess total volume capacity which for the current modeling results is on the order of 50 ac-ft for the 10-year storm event.

An additional mitigation measure such as check dams can be implemented to address specific post-development hydraulic characteristics that remain after implementation of the decompaction measure. Check dams could be located near the downstream southern boundary of the project site to intercept run off storm water flows. The intent of this measure is to reduce outflow volume by retaining run-off storm water volume. Check dams would have an effect on the storm water upstream of each dam because the storm water would back up behind each dam. Check dams would also reduce flow velocities and sediment transport leaving the project site. Check dams would change the Manning's roughness ("n") values used in the model at their immediate vicinity. It can be assumed that the check dams can be designed to retain the excess total storm water volume. Once the check dams are designed, their detention capacity volume can be subtracted from the total outflow volume of any of the simulations. Check dams would be designed to retain the excess total volume capacity which for the current modeling results is on the order of 50 ac-ft for the 10-year storm event.

An additional mitigation measure such as strip detention basins can be implemented to address specific post-development hydraulic characteristics that remain after the implementation of the decompaction measure. The strip detention basins would be approximately 6-inches deep and 70 feet wide. The strip detention basins would be designed to follow the contours and so the lengths would be dependent on the locations of the basins on the site. These detention basins could be located near the downstream southern boundary of the project site to intercept run off storm water flows. The intent of this measure is to reduce outflow volume by detaining run-off storm water volume, similar to the check dam measures. Strip detention basins would not have an effect on the storm water upstream of each basin but would reduce flow velocities and sediment transport leaving the project site. Strip basins would not appreciably change the Manning's roughness ("n") values used in the model for the project. The strip detention basins would not be as effective a measure as the check dams. Check dams can be designed to hold more volume than the strip detention basins when placed on flatter slopes and also check dams will act as a bigger obstacle than strip detention basins attenuating storm water flow. It can be assumed that the strip detention basins can be designed to retain the excess total storm water volume and would have a retention volume capacity equivalent to that for the check dams. Strip detention basins would be designed to retain the excess total volume capacity which for the current modeling results is on the order of 50 ac-ft for the 10-year storm event. Once the strip detention basins are designed, their detention capacity volume can be subtracted from the total outflow volume of any of the simulations.

5.3.5 Discussion of Effect on the Pinto Wash

As shown on the pre-development and post-development figures, the development will not significantly affect the storm water flow in the Pinto Wash. For the most part, the storm water flow in the Pinto Wash will encroach onto the DSSF for 10-year and 100-year storm events. The figures show that the flow on the DSSF does not enter the Pinto Wash along the DSSF boundary (or within the boundaries of the model), rather the storm water outflow from the site will enter the Pinto Wash in an area several miles downstream of the DSSF. The volume of storm water in the Pinto Wash is on the order of 4,072 ac-ft for the 100-year storm event and 1,545 ac-ft for the 10-year storm event. The DSSF does not increase Pinto Wash flows at the downstream end of the project, however, an additional 76 ac-ft for the 100-year event from the DSSF would eventually make its way into Pinto Wash at which point the increase is expected to be less than 1%. Velocities and depths within the pinto wash will not change as a result of development. The DSSF development would not have a significant impact to a storm water flow in the Pinto Wash.

6 SEDIMENT TRANSPORT ANALYSIS

This section describes sediment transport for the project as predicted by FLO-2D. The sediment transport analysis is conservative because degradation depths presented do not reflect sediment deposition which may occur within the same model cell. The model does not account for local scour at the supports for the solar panels. Local scour is evaluated later in this report; see Section 8 LOCAL SCOUR ANALYSIS

6.1 Methodology

The existing and proposed model configurations discussed in the Hydraulics Section were modified to account for sediment transport. FLO-2D has the capability of simulating sediment transport and offers several different methodologies. The Zeller and Fullerton methodology was selected for sediment transport analysis of the DSSF since this methodology is appropriate for alluvial floodplain conditions (FLO-2D User's Manual, 2007). Sediment profile information was obtained from the geotechnical study (EUC, 2007).

6.2 Results

The existing and future conditions with decompaction were modeled under AMC II conditions to determine the loss in depth of the sediment (degradation or scour) during the 100- and 10-year storm events. Maps presenting the results of the 100-year and 10-year peak degradation are shown in Figure 34 and Figure 37 respectively. Graphs showing change in sediment transport depth can be found in Figure 35 and Figure 38. Table 9 presents average degradtion depths for the 100-year storm event within the DSSF for the simulations. The modeling results determined that the average degradation for the 100-year storm event within the project site does not change (the difference is 0.0%) for the future conditions with decompaction.

Table 9. Sediment Transport Summary: 100-year storm

Simulation	Average Degradation Depth	Change
Existing Conditions	0.04 ft	NA
Future Conditions with Decompaction	0.04 ft	0.00 ft (0.0%)

The 10-year simulation results are presented below in Table 10. The modeling results determined that the average degradation for the 10-year storm event within the project site does not change (the difference is 0.0%) for future conditions.

Table 10. Sediment Transport Summary: 10-year storm

Parameter	Average Degradation Depth	Change
Existing Conditions	0.01 ft	NA
Future Conditions with Decompaction	0.01 ft	0.00 ft (0.0%)

Sediment transport, based on the sediment particle size, showed that the proposed installation did not have any impact on degradation; the average degradation depth is 0.04 feet for the 100-year storm and 0.01 feet for the 10-year storm over most of the DSSF for both pre- and post-development conditions. The results show that the average degradation within the project site remains the same for existing conditions and all development options.

Although the modeling results indicate that the average degradation depth is not significant for both pre- and post-development conditions, sediment transport may occur as a result of either a large storm event or a series of smaller storm events. This issue can be mitigated by periodic monitoring and maintenance of the site. For example, monitoring conducted after storm events would indicate sediment depth at that time and maintenance activities would be conducted as/if needed to add/remove material to restore design conditions. A Monitoring and Response Plan will be incorporated into the final design of the DSSF to ensure that the storm water infrastructure is in good working order on an ongoing basis during Project operation.

7 FLUVIAL GEOMORPHOLOGIC ASSESSMENT

7.1 Methodology

AECOM reviewed existing data including geologic literature, site reports, aerial mapping and topographical survey to qualitatively determine the fluvial geomorphology of the DSSF. Aerial photographs from the years 1978, 1996 and 2002 were analyzed to determine changes in land use and stream channel configurations.

As noted earlier, the DSSF is located in the Chuckwalla Valley, which is bounded by a series of alluvial fans that slope gently to moderately toward the southwest and southeast. The Pinto Wash runs through the center of the valley. The DSSF facilities are to be located to the west of the Pinto Wash. Vegetation at the site generally consists of sage and other scrub-type brush that is typical for the arid regions of southern California (EUC, 2007).

The geomorphology of alluvial fans is described by John Field and Philip Pearthree in their article "Geomorphologic Flood-Hazard Assessment of Alluvial Fans and Piedmonts" published in the Journal of Geoscience Education, Vol. 45, 1997:

> "Alluvial fans are generally cone-shaped depositional landforms with distributary drainage patterns that emanate from a discrete source and increase in width downslope. Older, inactive, alluvial fans commonly are isolated from active depositional processes and dendritic drainage patterns are developed on them."
>
> "Surfaces that are subject to flooding are undissected, display well preserved bar-and-swale topography, and lack desert pavement and varnish. In contrast, surfaces that have not been flooded for hundreds of thousands of years are moderately to deeply dissected, have well developed desert pavements and abundant shattered cobbles on the surface; their soils include substantial accumulations of clay and calcium carbonate (caliche)."
>
> "Several criteria can be used to distinguish between a permanent and temporary trench. Fanhead trenches dissecting inactive surfaces with well developed soils, desert pavement, and rock varnish are permanent features, since it is the incision of the trench itself that is largely responsible for the isolation of the adjacent old surfaces. A trench dissecting a young surface, on the other hand, is potentially only a transient feature. The depth of incision alone should not be used to determine whether a trench is permanent. Trenches as deep as 8 m can be filled and/or cut during a single debris flow event. …Regardless of the absolute depth of the incision, a fanhead trench is not a permanent feature if floodwaters can overtop or backfill the channel under the prevailing hydrologic conditions."

Review of recent aerial imagery and site photographs indicates that there are two significant geologic environments occurring at the DSSF. The first geologic environment is characterized as older alluvial sediments with developed desert pavement. This environment occurs in the northwest portion of the site in the vicinity of Power Line Road. It also occurs in the southwest corner of the site adjacent to Kaiser Road (Co Route R2). Based on LIDAR topographic survey data, alluvial stream channel depths near Power Line Road approach four feet at the northwest end of the project while the channels near Kaiser Road are generally two (feet or less.

The second significant geologic setting at the DSSF site consists of an area of active younger sediments with no evidence of desert pavement. Topography in these areas tends to be very consistent with channels depths generally less than one foot deep.

The EUC "Phase 1 Geologic Reconnaissance Report" corroborates the two significant conditions encountered at the site. EUC describes the established alluvial sediments as follows:

> "Older alluvial fan deposits consisting of Pleistocene nonmarine sediments extend outward into the valley from both the Eagle Mountains on the west and the Coxcomb Mountains on the east. Desert pavement type deposits (manganese and iron oxidized coatings on cobbles and sand) blanket the top three (3) to six (6) inches of the older alluvial fan material."

EUC describes the area near Power Line Road and Kaiser Road as the “Northwest fan – includes sediments derived from the Eagle Mountain Quartz Monzonite, Pleistocene volcanic rocks, and Pre-Cretaceous metamorphosed sediments.” In contrast, they describe the younger active sediments as “of Holocene age. These soils consist of fine to coarse sand, interbedded with clay, silt and gravel.”

Lateral migration of stream channels is typically evaluated based on the analysis of historical aerial photographs. AECOM reviewed aerial photographs from the years 1978, 1996 and 2002 at the proposed site. Based on the data available, stream channels at the site have been relatively stable over the period evaluated. It is more difficult to determine the stability of smaller channels located in the more active portions of the site due to their scale. Based on knowledge of similar environments, it would be expected that alluvial stream channels in the older alluvial regions remain relatively stable. It is anticipated that the shallow channels that exist within the younger sediment would exhibit frequent channel avulsion and lateral migration during flood flows.

7.2 Results

Changes to the vertical profile of the stream channels are difficult to quantify without detailed survey data of Project site topography over time. However, existing conditions at the site indicate channel depths of two to four feet in the older alluvial sediments and less than two feet in the younger sediments.

The grading design of the DSSF includes grading of the entire site with varying levels of compaction depending on proposed land use (primary road, secondary road, etc.). Existing slopes on the site vary from zero to two percent in the active alluvial areas to two to four percent in the regions of less active older alluvial sediments. Planned slopes will be zero to two percent across the entire site.

The proposed changes to the site will have an impact on future geomorphic conditions. Instead of relatively inactive areas characterized by desert pavement in combination with more active areas, the geologic conditions at the site will change to a more consistent geological condition. Changes to existing site grades will also have an impact on flood flows. It is anticipated that these changes will create a geologic environment conducive to rapidly migrating shallow channels, approximately two feet deep or less. Channel formation from fluvial geomorphology occurs as a result of multiple storm events over time. This long term scour or channel formation can be mitigated by periodic monitoring to identify changes to the site grading, followed by maintenance measures to address these changes as/if needed. Development of a Monitoring and Response plan would address monitoring of the drainage control devices after storm events and development of appropriate maintenance responses so that the drainage control devices are operational for subsequent storm events. Flatter slopes may also contribute to areas of sediment deposition during storm events.

If further evaluation of existing and post-development conditions at the site is needed, a detailed quantitative fluvial geomorphologic assessment will be conducted. The quantitative evaluation would include a detailed analysis of stream migration based on historical aerial images, additional historical information including interviews with local inhabitants, and site reconnaissance to determine channel characteristic, extent of desert pavement and soil properties.

Photo 1. Stream channel in older alluvial sediments (desert pavement)

Photo 2. View of desert pavement material

8 LOCAL SCOUR ANALYSIS

The total predicted scour depth is the sum of the following components: general scour, long term scour and local scour. General scour is discussed in the Sediment Transport Analysis Section 6 of this report. Long term scour depth is estimated in the previous Fluvial Geomorphologic Assessment Section 7. It is assumed that the long term scour can be mitigated by periodic monitoring to identify changes to the site grading and followed by maintenance measures to address these changes as/if needed. Therefore, the total scour depth presented in this section is assumed to be the local scour and general scour that the site structures could experience. The local scour is discussed herein for the future conditions 100-year storm event. Local scour is measured at an instantaneous point in time as a result of turbulent flow at the pylons. Sediment is suspended at the base of these structures within the turbulent flow. As the sediment moves away from the turbulent zone the flow can no longer support the sediment load and it is deposited a short distance downstream. Local scour occurs at the base of a structure as a result of the change in direction and velocity of storm water as the water flows around the structure. The effect of the local scour is limited to the area immediately adjacent to the base of the PV solar panel support structures.

8.1 Methodology

For the purpose of this study, local scour was analyzed at the base of the PV solar panel support structures. Scour depths were calculated using a local pier scour equation from the Federal Highway Administration's Hydraulic Engineering Circular No. 18 (HEC-18), "Evaluating Scour at Bridges" (4th Edition).

Scour depths were calculated for each element in the 2D model within the DSSF. Velocity and depth outputs from the model were used to determine scour at each element. The dimensions of a model element are 200-feet by 200-feet and velocities and depths predicted by the model are averaged across the element area. Therefore, the velocities may not be conservative because high concentrations at portions of the element are lost and larger scour depths than predicted may occur.

The local scour equation and the various parameters and assumptions are as follows:

$$\frac{y_s}{a} = 2.0K_1K_2K_3K_4\left(\frac{y}{a}\right)^{0.35}Fr^{0.43}$$ **(Equation 1)**

Where:

y_s = Local scour depth (ft);
K_1 = Correction factor for pier nose shape;
K_2 = Correction factor for angle of attack of flow;
K_3 = Correction factor for bed condition;
K_4 = Correction factor for armoring;
a = Pier width (ft);
y = Flow depth (ft);
Fr = Froude number:

$$Fr = \frac{V}{\sqrt{gy}}$$ **(Equation 2)**

Where:

V = Average velocity (ft/s);
g = Acceleration due to gravity (ft/s^2).

8.2 Approach

Two (2) different scour depth analyses were performed to encompass the best and worst case scour depths by varying the pile geometry. The only parameters of the scour equation that change in each case are the pier width (a) and the correction factor for angle of attack (K_2). All other values (velocity, depth, etc.) remain the same for a given element within the modeled domain. A plane bed was assumed for the bed condition, resulting in a K_3 factor of 1.1. The grain size analyses collected during the EUC Phase 1 Geologic Reconnaissance Report all contained a median particle diameter of less than two (2) millimeters, resulting in a K_4 factor of 1.0.

8.3 Inputs and Assumptions

The proposed pile configuration consists of steel wide flange I-beams (W6X9). The shape correction factor was assumed to be square for both cases, resulting in a K_1 factor of 1.1. The worst case analysis assumed the pier width was the largest flange dimension (5.9 inches) and the angle of attack was assumed to be 90 degrees. A 90 degree angle of attack produces the largest K_2 value (1.3). The equation for determining K_2 is shown below (HEC-18):

$$K_2 = \left(Cos\theta + \frac{L}{a} Sin\theta \right)^{0.65}$$

(Equation 3)

Where:

L = Length of pile (ft);
Θ = Angle of attack of flow (degrees).

The worst case angle of attack assumptions mentioned above produce the most conservative scour depth results. The best case scour analysis assumed the pier width was the smallest flange dimension (3.94 inches) and the angle of attack was assumed to be zero degrees. A zero degree angle of attack produces the smallest K_2 value (1.0). The best case angle of attack assumptions produce less conservative scour depths and are not presented herein. A visual representation of the 100-year worst case scenario is shown on Figure 39.

8.4 Results

The maximum local scour depth (i.e. when the flow is aligned with the widest part of the support structure) for the DSSF using the worst case assumptions described above for the 100-year storm was 2.1 feet. The maximum total scour within the project site was 2.9 feet. This was the combination of local scour and general scour within the same model cell. This scour depth occurred for both the future conditions and future conditions including the decompaction mitigation measure. The areas of maximum scour potential are along the northwest portion of the site. The average scour depth was found to be 1.2 feet. Table 11 shows the frequency of occurrence for the more-erosive scour depths within the project site. Figure 39 shows the distribution of maximum local scour depths using worst case assumptions within the Project area for the future conditions 100-year storm.

Formation of local areas of scour can occur as a result of a large storm event or a series of smaller storm events. Local scour can be mitigated by periodic monitoring and maintenance of the site. A Monitoring and Response Plan will be utilized during operations of the DSSF to ensure that PV supports remain in stable operational condition and are not compromised by local scour impacts.

Table 11. Local Scour Summary: 100-year Worst Case Frequency of Occurrence within the Project Site for Decompaction

Depth of Scour	Local Scour	Total Scour
0.0 to 0.5 feet	0.2%	0.2%
0.5 to 1.0 foot	20.6%	20.0%
1.0 to 1.5 feet	63.3%	57.5%
1.5 to 2.0 feet	15.9%	20.6%
2.0 to 2.5 feet	0.1%	1.5%
2.5 to 3.0 feet	0.0%	0.2%
Average Scour Depth	1.2 ft	1.3 ft
Maximum Scour Depth	2.1 ft	2.9 ft

The less erosive-case (i.e. when flow direction is aligned with the narrow side of the support structure) maximum scour depth was 1.2 feet and total scour was 2.2 feet. Frequency of occurrence can be found in Table 12 for the less-erosive case.

Table 12. Local Scour Summary: 100-year Best Case Frequency of Occurrence within the Project Site for Decompaction

Depth of Scour	Local Scour	Total Scour
0.0 to 0.5 feet	11.3%	10.8%
0.5 to 1.0 foot	86.1%	79.6%
1.0 to 1.5 feet	2.5%	8.9%
1.5 to 2.0 feet	0.0%	0.7%
2.0 to 2.5 feet	0.0%	0.1%
Average Scour Depth	0.7 ft	0.8 ft
Maximum Scour Depth	1.2 ft	2.2 ft

9 CONCLUSIONS

The results of the storm water modeling are:

1 Results of the hydrologic analysis for the DSSF development indicated that implementing decompaction of the areas between the panels will reduce the post development hydraulic conditions to within +/-5% of the pre-development hydraulic conditions. An additional on-site mitigation measure such as basins with rip-rap protection, check dams or strip detention basins can be implemented to retain the remaining excess total off-site storm water volume increase. Please note that the accuracy of the model is approximately +/- 5% and so the differences (i.e. within 5%) calculated by the model are within this range.

2 Results of the hydrologic analysis for the post-development DSSF grading design without the addition of a mitigation measure indicated that, in general, storm water off-site peak flow rates and volumes increased 5.8% and 2.3%, respectively for the 100-year storm event and 6.7% and 5.5respectively for the 10-year storm event. On-site velocities increased 17.4% for the 100-year and 19.4% for the 10-yearand on-site flow depths decreased 4.5% for the 100-year and 7.1% for the 10-year, as compared to the pre-development existing conditions

3 Results of the hydrologic analysis for post-development design that only includes a decompaction mitigation measure indicated that the storm water off-site peak flow rate and volume increased 5.1% and 1.1%, respectively for the 100-year storm event and 2.6% and 2.5%, respectively for the 10-year storm event. On-site velocity increased 15.2% for the 100-year and 19.4% for the 10-year, and on-site peak depth decreased 4.5% for the 100-year and 7.1% for the 10-year storm event, as compared to the existing conditions.

4 Results of the hydrologic analysis for post-development design that only includes a rip-rap mitigation measure indicated that the storm water off-site peak flow rates and volume, and on-site depth slightly increased 0.6%, 2.3%, and 4.5%, respectively for the 100-year storm event. On-site peak velocity did not change, as compared to the pre-development existing conditions for the 100-year storm event. However, for the 10-year storm event, storm water total off-site outflow volume and on-site peak flow depth increased, 5.5% and 7.1%, for the 10-year storm event. Off-site peak flow rate and on-site peak velocity decreased 3.0% and 6.5% for the 10-year storm event, compared to the existing conditions.

5 The addition of mitigation measures such as basins with rip-rap protection, check dams, or strip detention basins to the DSSF development in addition to decompaction, will address excess post-development hydraulic impacts that are not addressed by decompaction. These additional measures are based on implementing storm water best management practices and have not been rigorously modeled, however they would be designed to retain excess total off-site storm water volume. The intent of an additional mitigation measure is to reduce overall flow depths, velocities and outflow volume by detaining run-on storm water volume. The additional measures would also be successful at reducing potential increases in sediment transport and would be designed to retain the excess total volume capacity which is on the order of 50 ac-ft for the 10-year storm event. Results of the sediment transport analysis for post-development determined that the average degradation for the 100-year and the 10-year storm event within the project site does not change (the difference is 0.0%) for future conditions. The average degradation depth for the 100-year storm would be 0.04 feet, and 0.01 feet for the 10-year storm (i.e., general scour).

6 Results of the total scour analysis for post-development found that the average on-site scour depth would be 0.8 to 1.3 feet at the base of the PV supports for the 100-year storm, depending on the angle of flow to the supports. Placement of riprap will provide a less significant benefit to mitigate for additional runoff. However, riprap placed at the base of each support structure will help reduce the effects of local scour and lower storm water runoff velocities.

7 Results of the qualitative fluvial geomorphologic analysis indicates existing areas of relatively inactive sediments characterized by desert pavement and more active areas consisting of finer sand and gravel. The changes to the site resulting from Project development will create an area that has

consistent compaction, soil type and grading compared to existing conditions. It is anticipated that these changes will create a geologic environment conducive to the formation of shallow channels up to two feet or less in depth (i.e. long-term scour). This long term scour can be mitigated by periodic monitoring to identify changes to the site grading and maintenance activities as/if needed to restore design conditions.

The results of the modeling indicate that the DSSF development would have a small impact on off-site peak flow rate and a negligible increase in maximum degradation depth comparing pre-development conditions to post-development conditions. These impacts are relatively small. However, the implementation of storm water mitigation measures will minimize impacts of the DSSF development on sedimentation and erosion characteristics in downstream areas with the result that post-development downstream conditions are essentially the same as pre-development existing conditions.

Along with the mitigation measures, a Monitoring and Response Plan will be prepared and submitted to the BLM. The Monitoring and Response Plan will indicate the procedures that will be followed to mitigate potential impacts to the site structures, storm water infrastructure or site grading that can occur from local scour, sediment transport and long term degradation (i.e. fluvial geomorphology) during the operation of the DSSF. This plan will address monitoring of the mitigation measures after storm events and development of appropriate maintenance responses so that the mitigation measures are in good working order and continue to be effective for subsequent storm events. Because the differences are so small (i.e. within +/- 5%) and there are a number of unknowns associated with real life conditions (i.e. compared to computer simulation), it is recommended that after each significant event (e.g. a 1-year storm or larger) hydrologic, hydraulic and sediment transport characteristics to be monitored. If acute or chronic problems are detected then modifications can be made as necessary.

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12. U.S. Department of Transportation, Federal Highway Administration, “Hydraulic Engineering Circular No. 18, Evaluating Scour at Bridges,” Fourth Edition. Publication No. FHWA NHI 01-001, May 2001.
13. Hill, Craig S., Earth Systems Southwest, “Estimate of Desert Pavement Coverage,” February 19, 2010

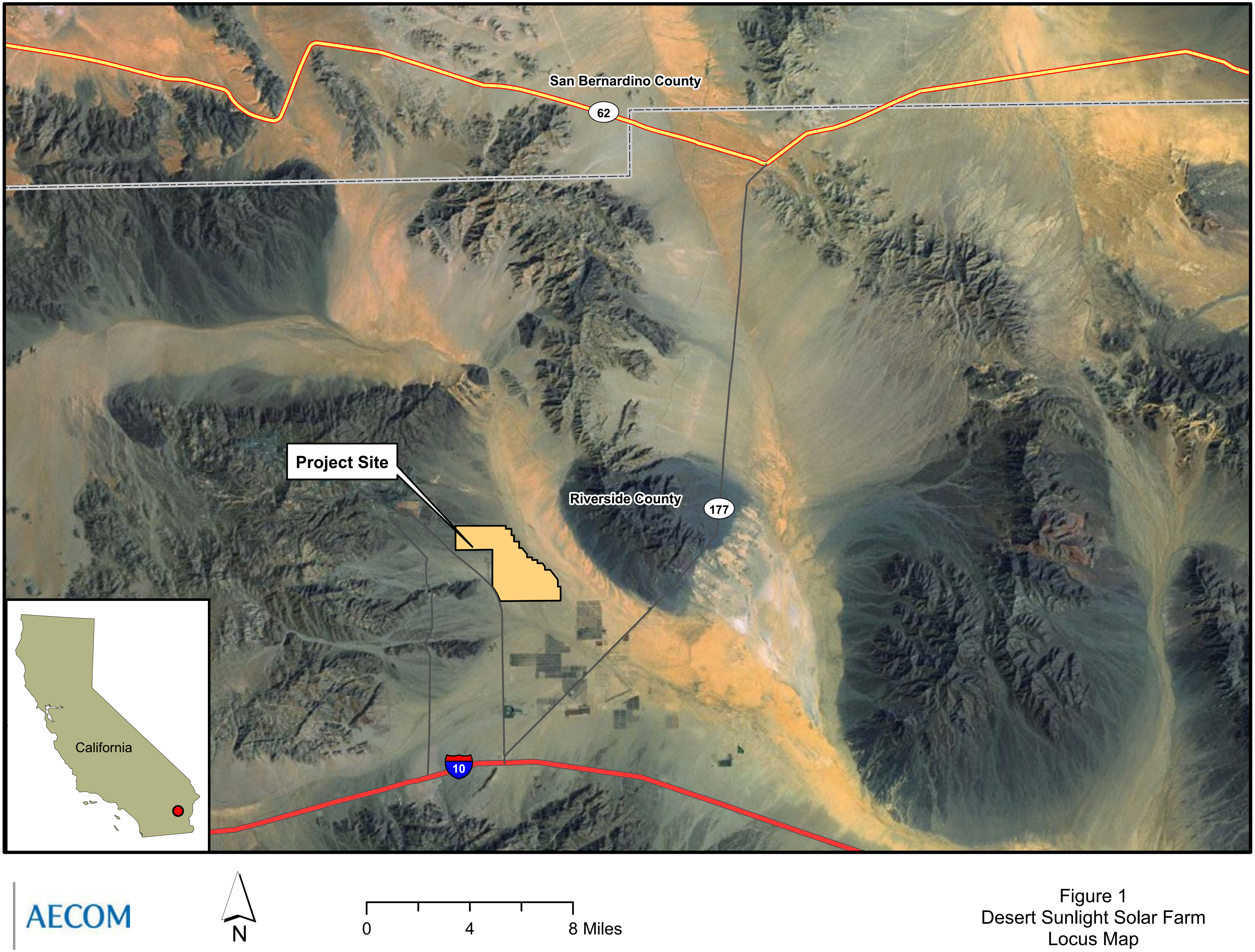

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Figure 1
Desert Sunlight Solar Farm
Locus Map

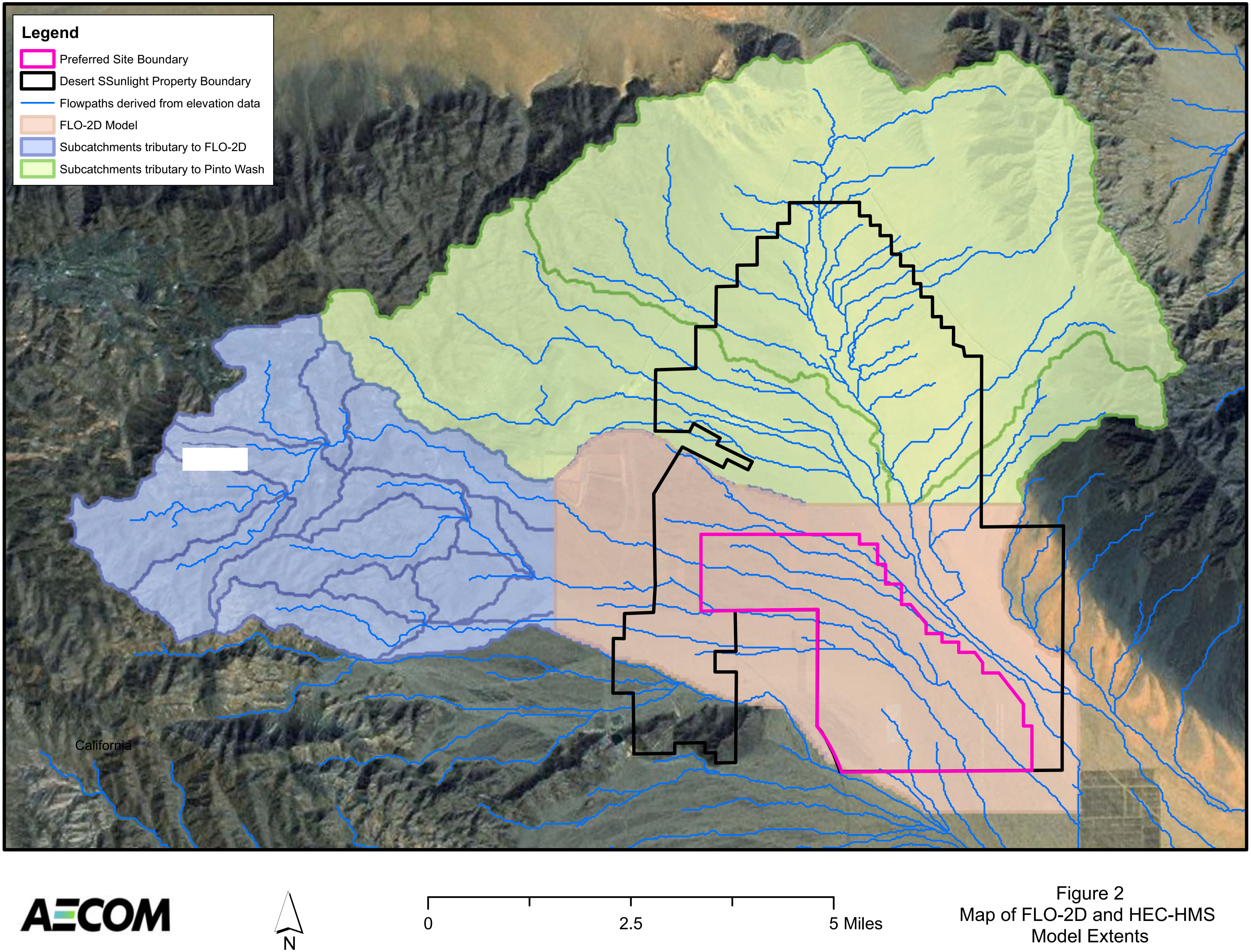

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Figure 2
Map of FLO-2D and HEC-HMS
Model Extents

Figure 3. Hyetograph of 100-year and 10-year Storm Events

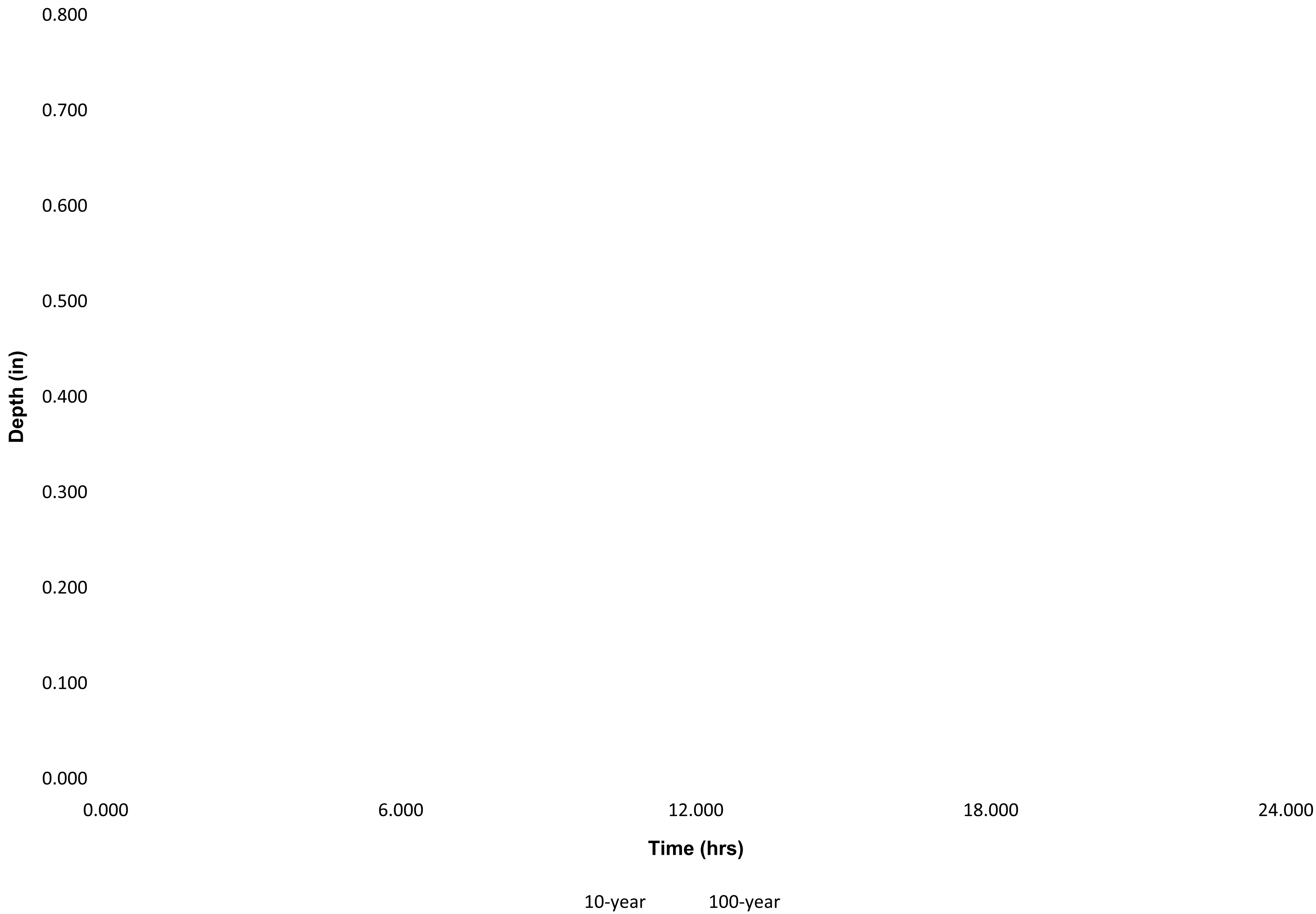
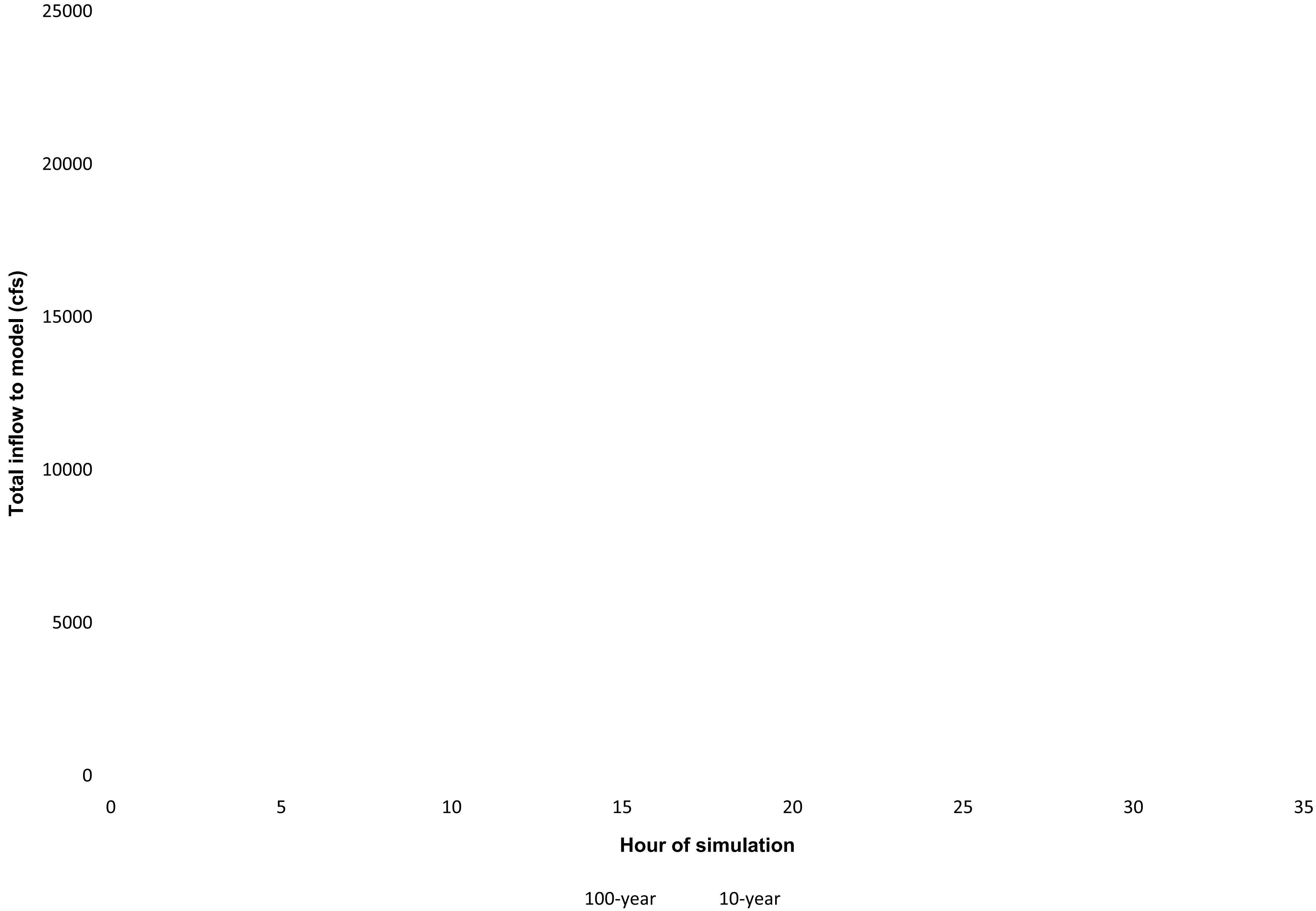
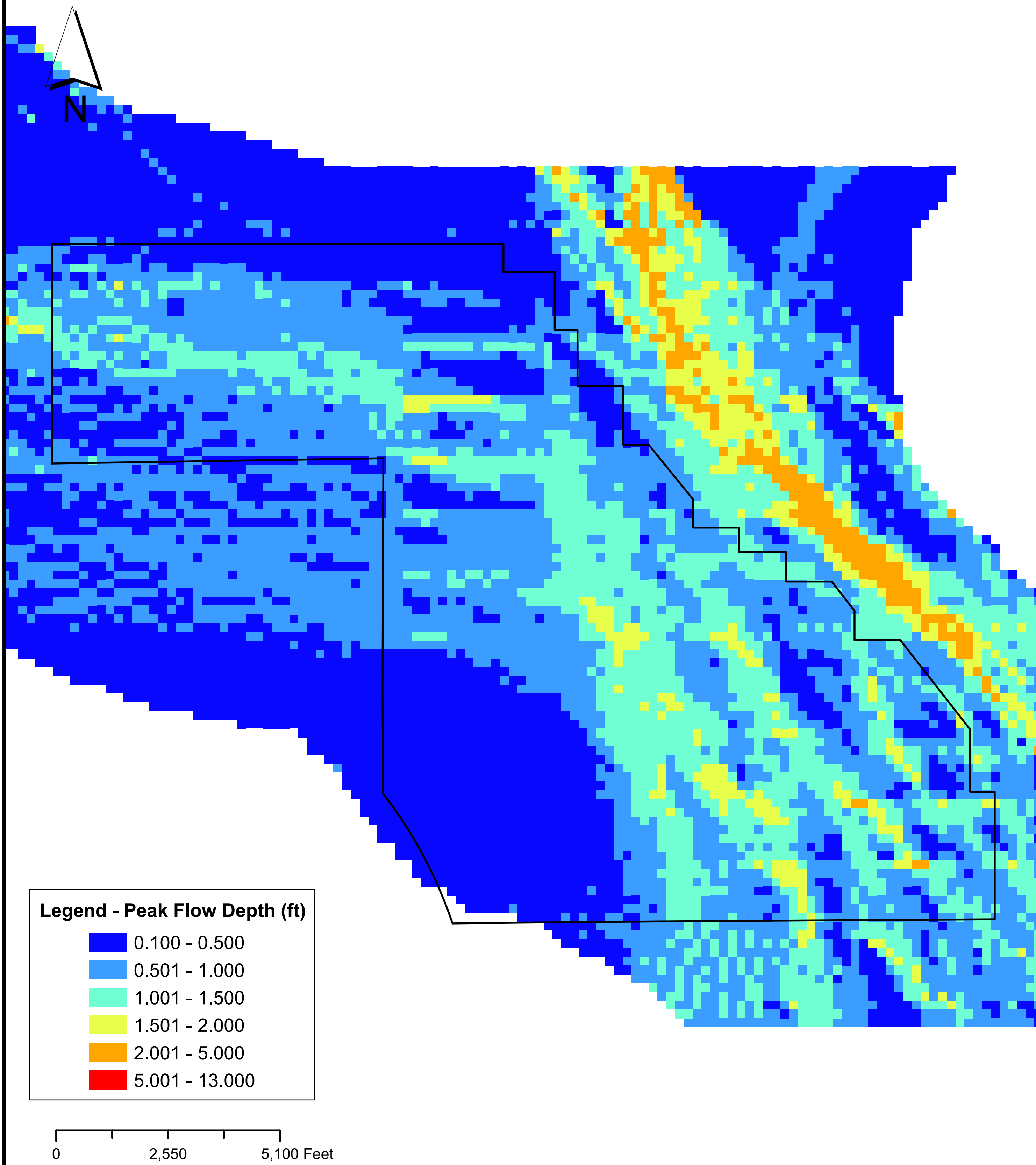


Figure 4. Estimated Inflow to Model





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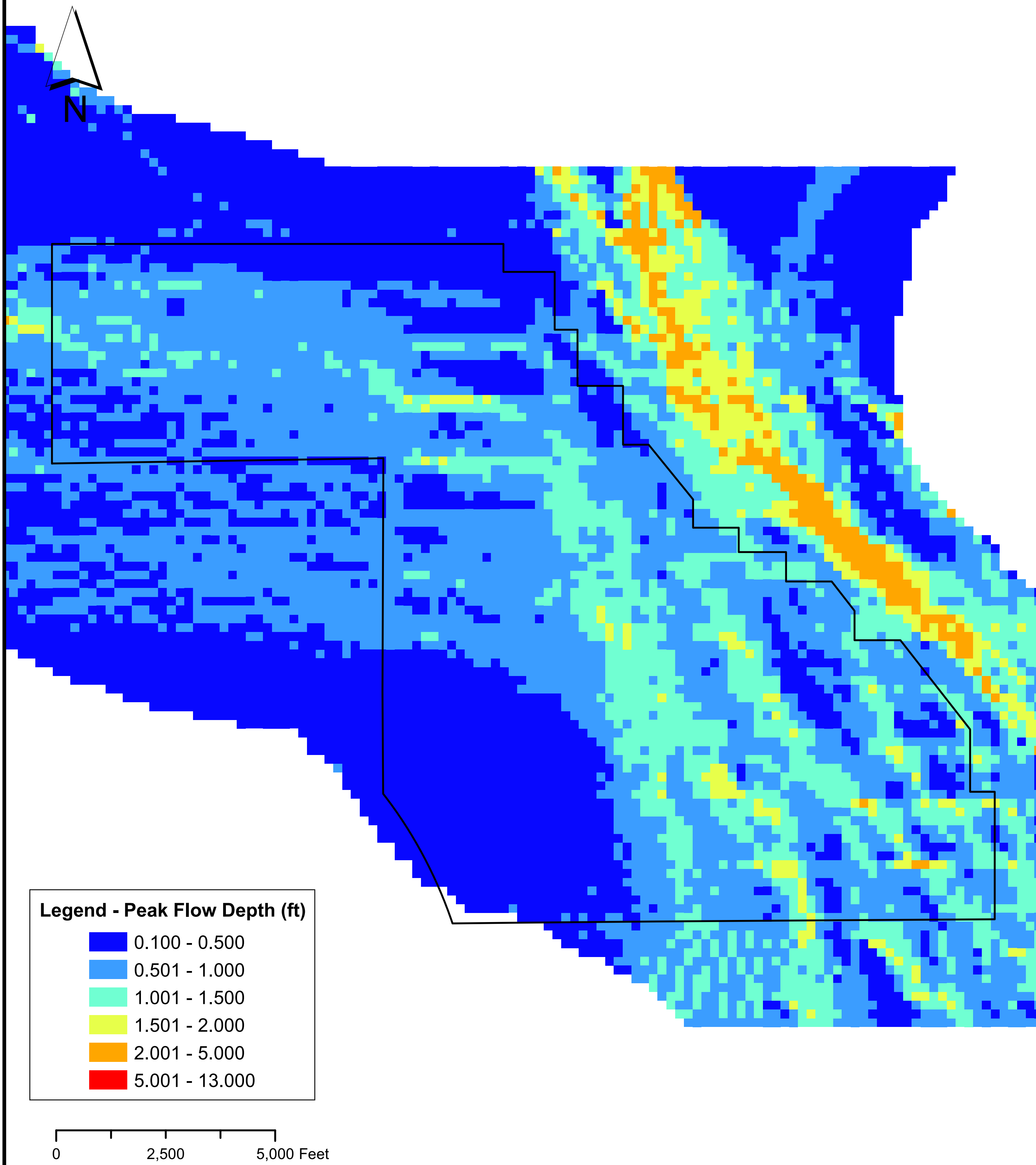
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Existing Conditions Peak Flow Depth

GIS FILE:	SCALE: AS NOTED	DATE: 03/05/2010	Figure 5



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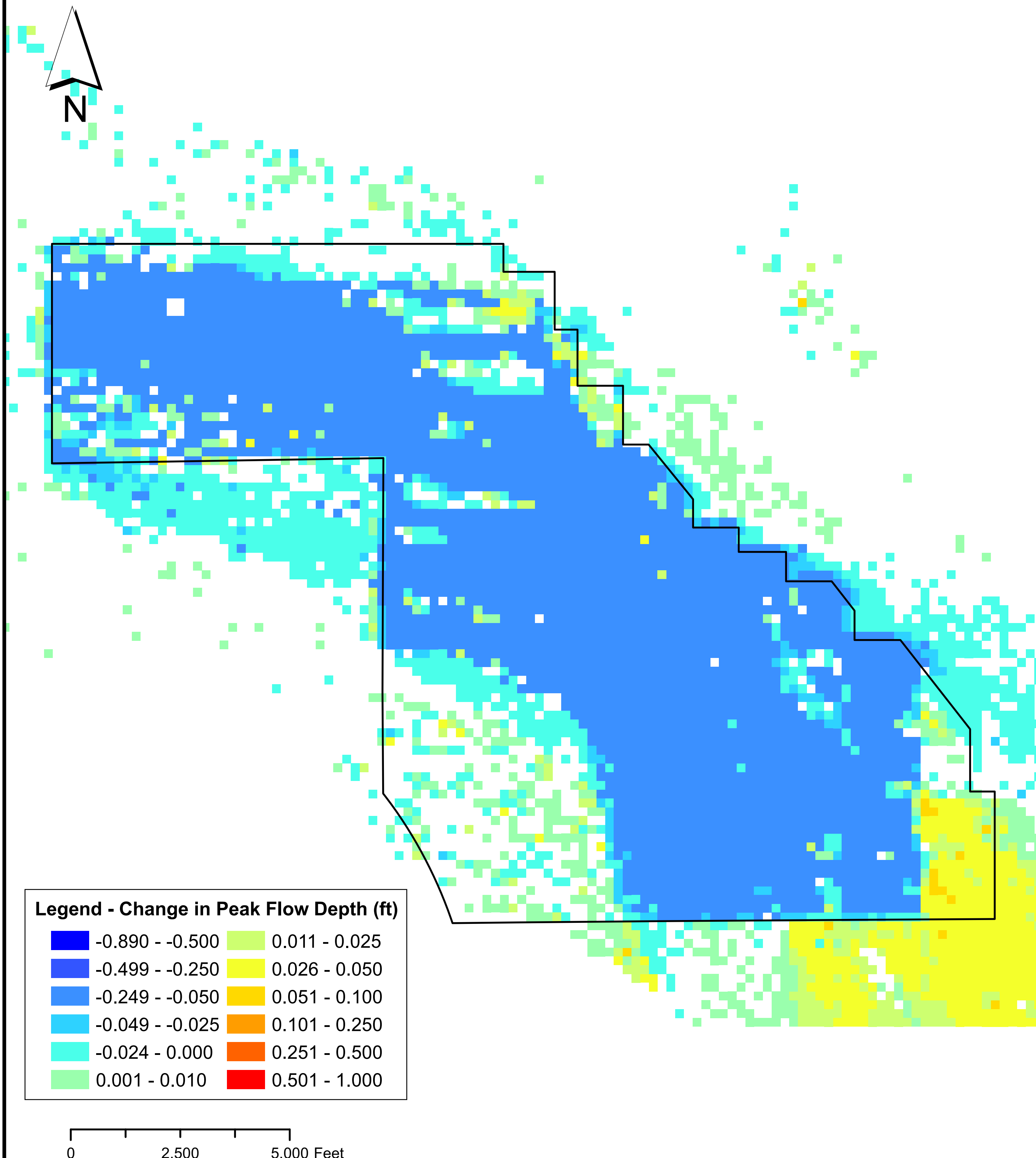
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Future Conditions Peak Flow Depth

GIS FILE:	SCALE: AS NOTED	DATE: 02/17/2010	Figure 6



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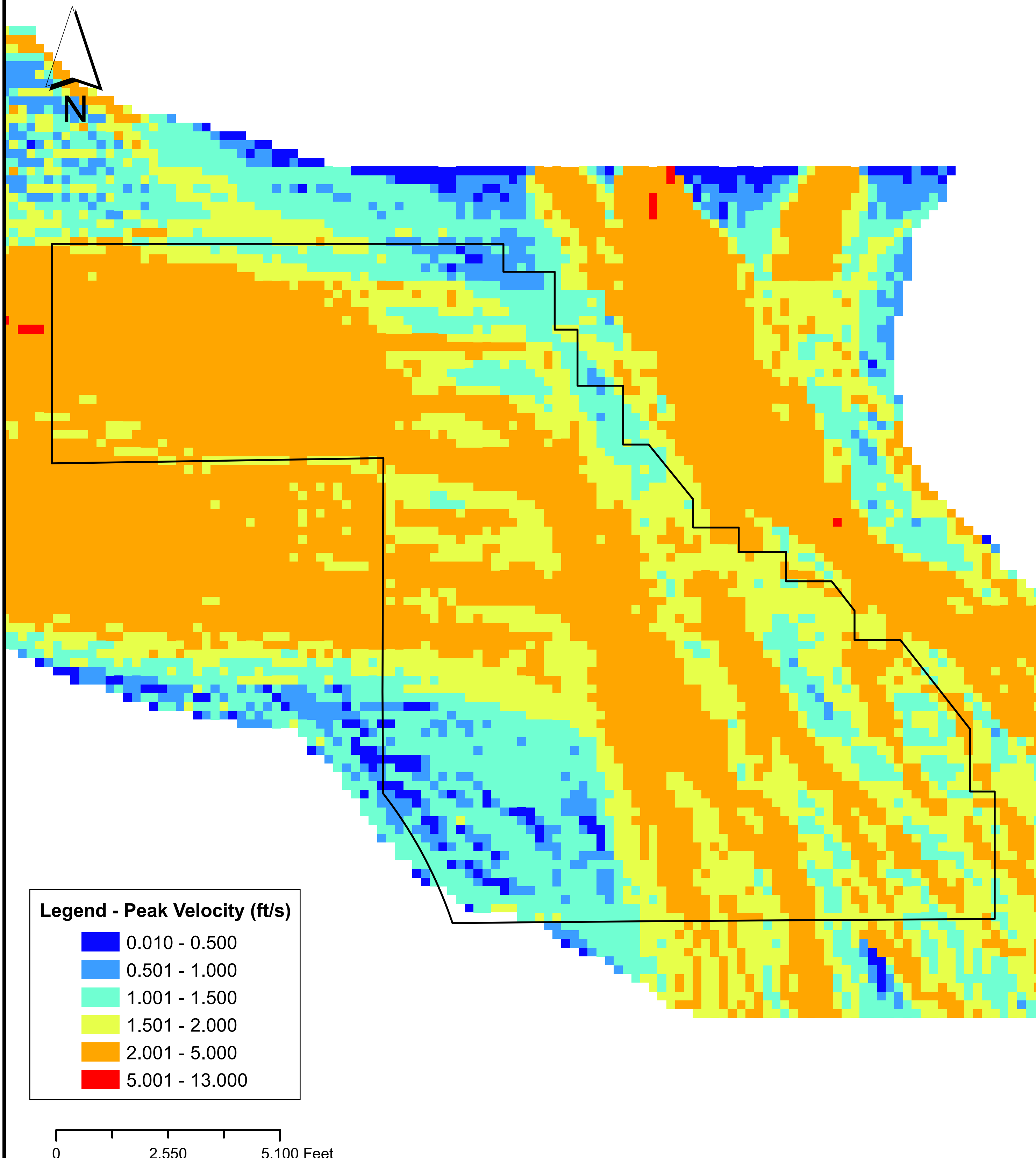
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Change in Peak Flow Depth (Future - Existing)

GIS FILE:	SCALE: AS NOTED	DATE: 03/09/2010	Figure 7



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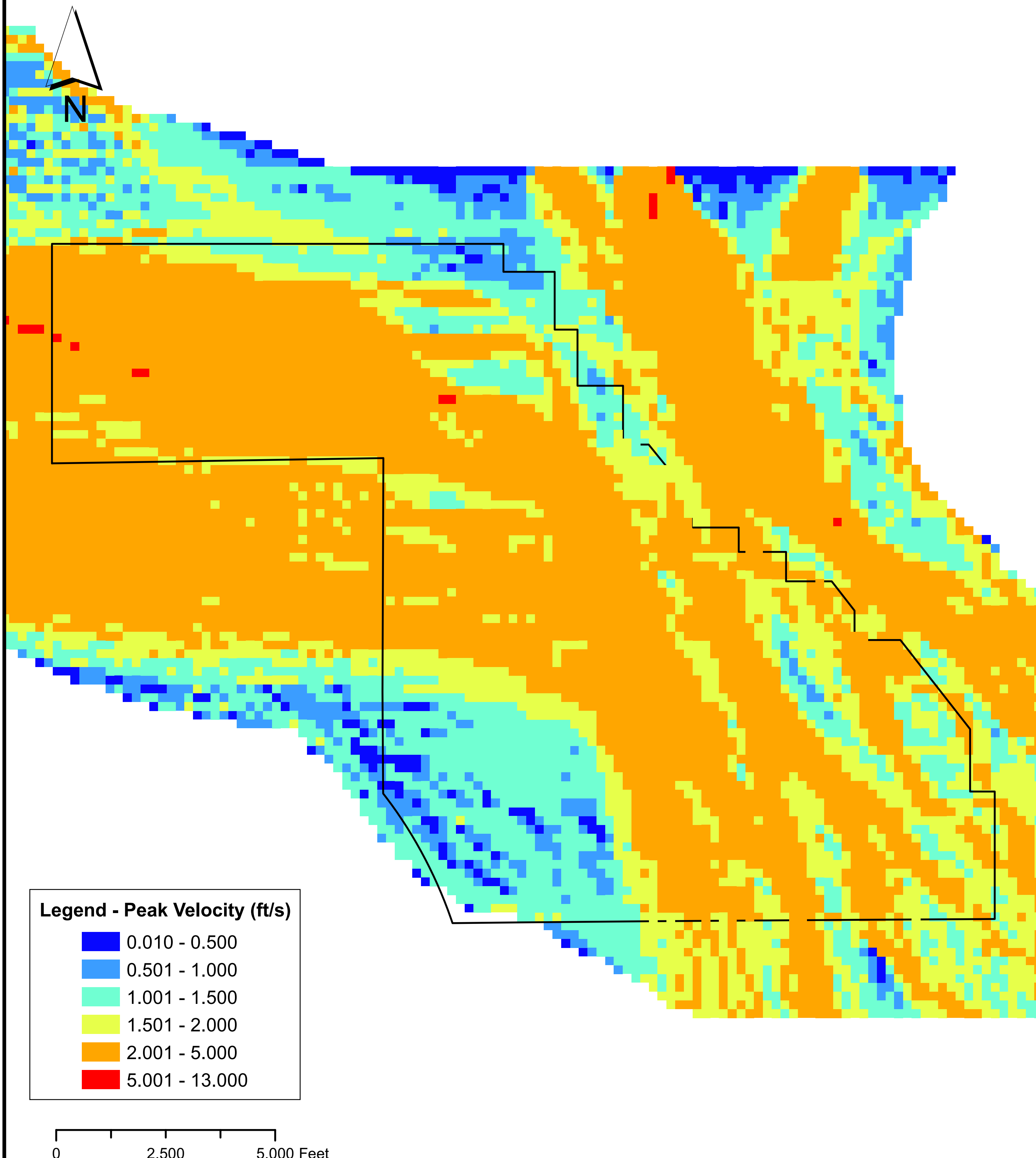
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Existing Conditions Peak Velocity

GIS FILE:	SCALE: AS NOTED	DATE: 03/05/2010	Figure 8



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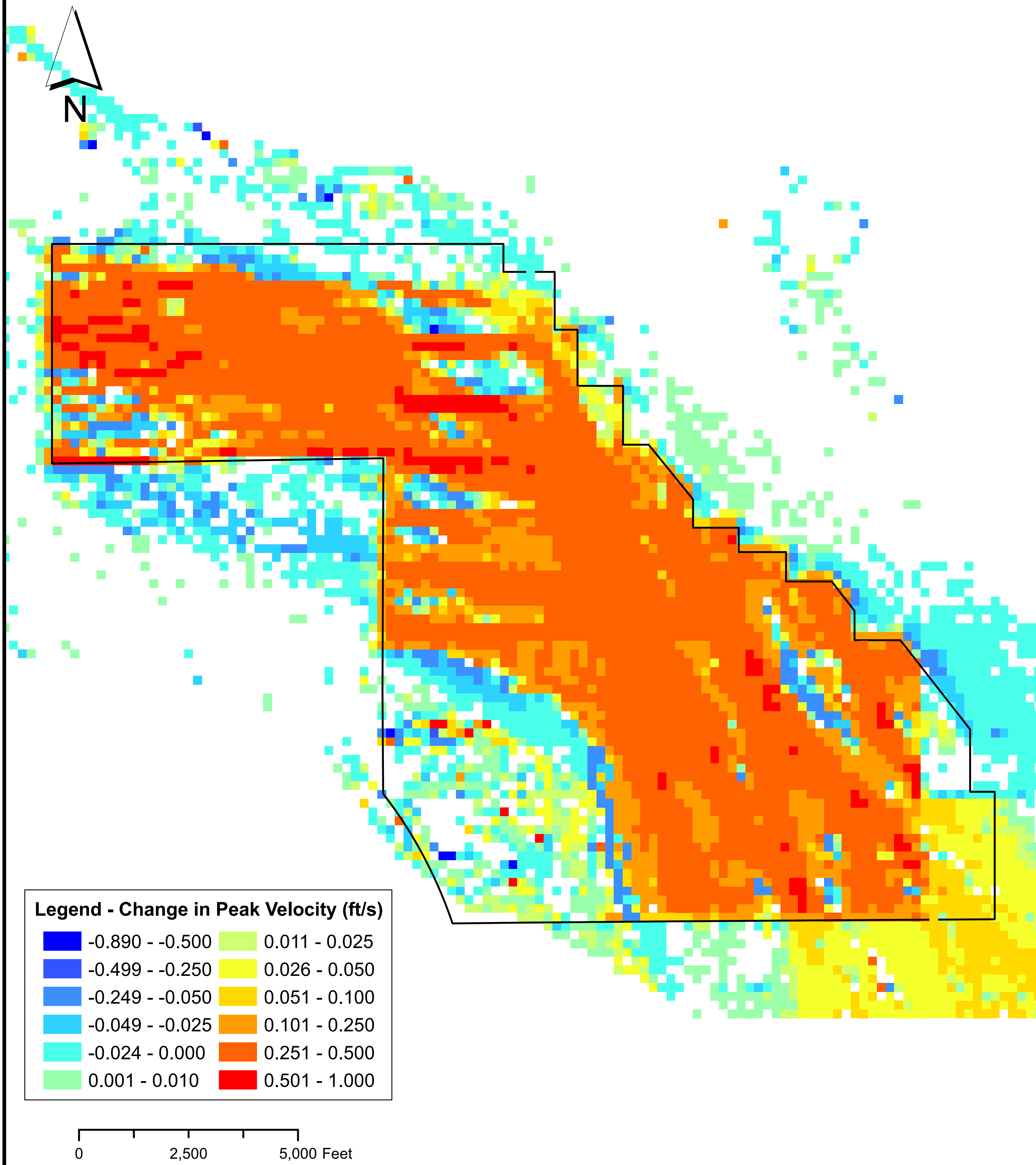
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Future Conditions Peak Velocity

GIS FILE:	SCALE: AS NOTED	DATE: 02/17/2010	Figure 9



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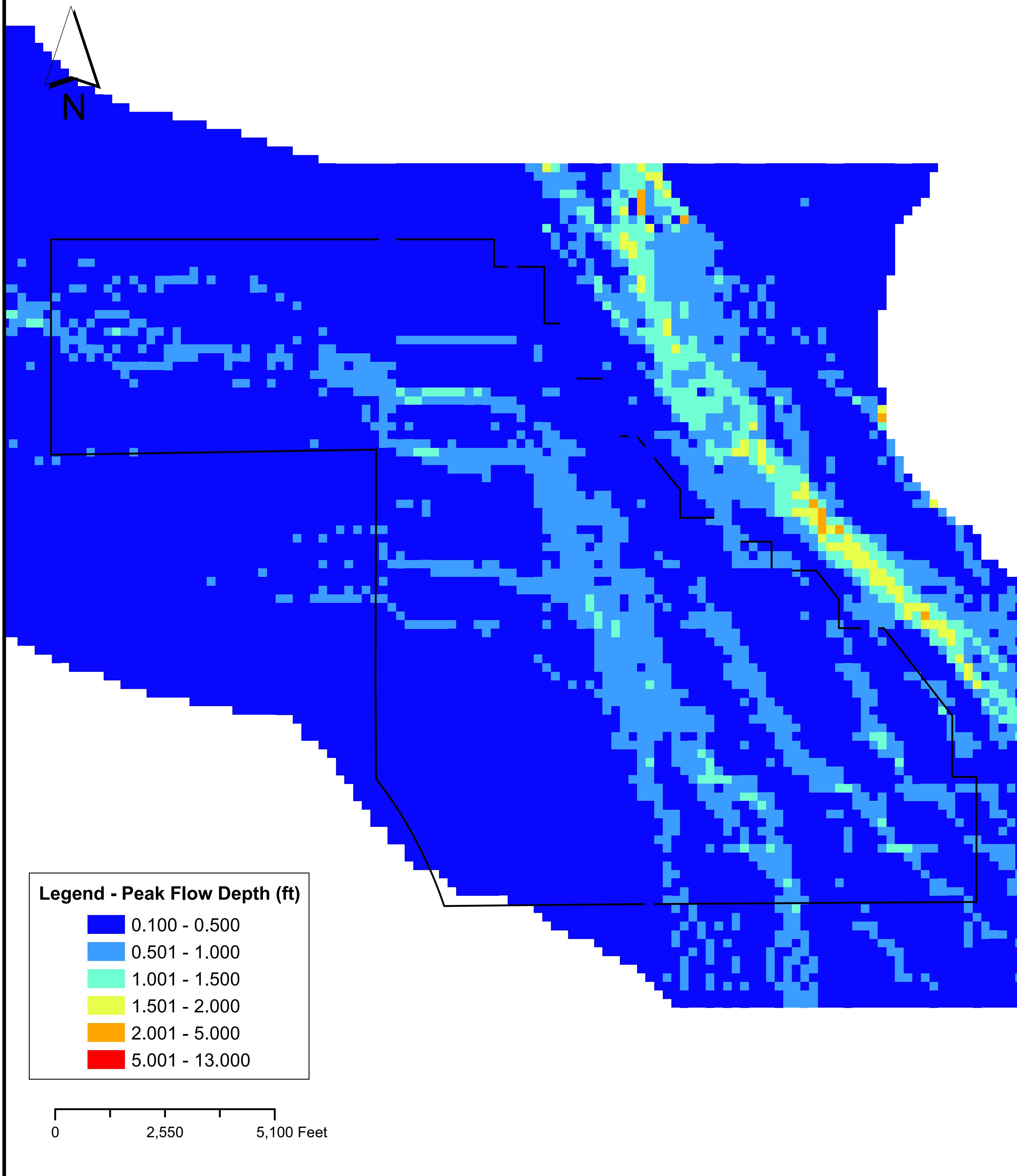
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Change in Peak Velocity (Future - Existing)

GIS FILE:	SCALE: AS NOTED	DATE: 03/09/2010	Figure 10



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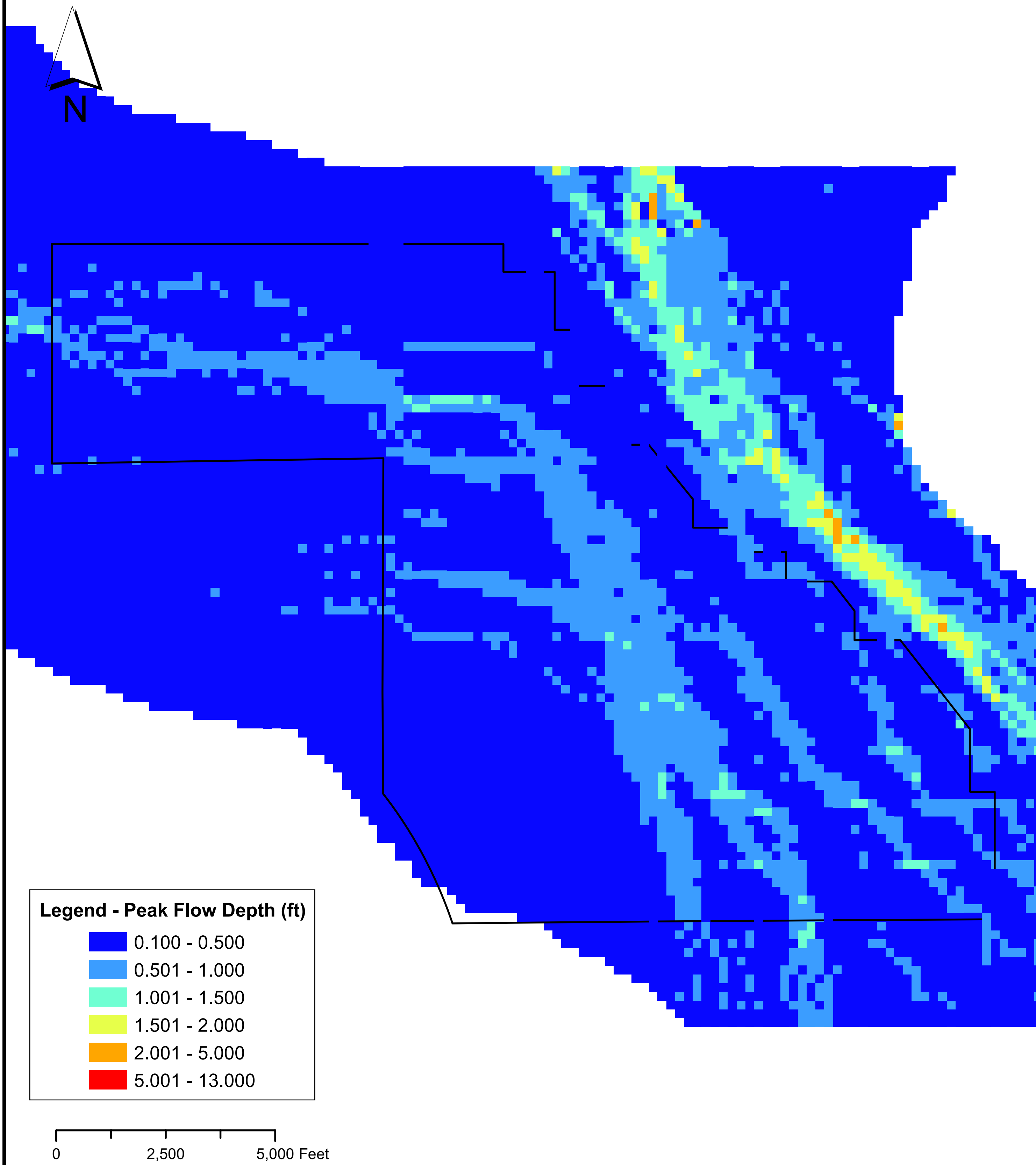
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
10 Year - Existing Conditions Peak Flow Depth

GIS FILE:	SCALE: AS NOTED	DATE: 03/05/2010	Figure 11



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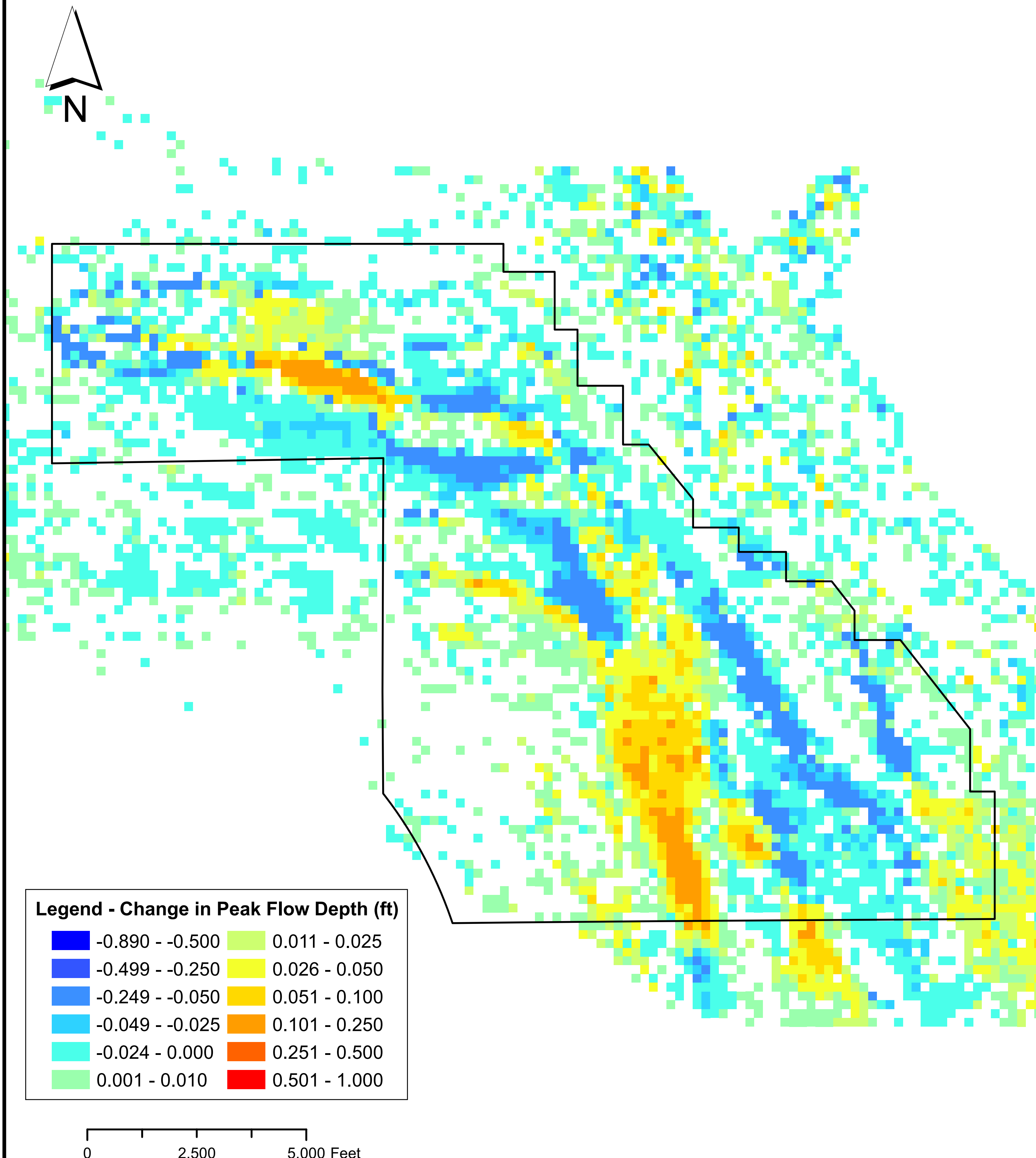
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
10 Year - Future Conditions Peak Flow Depth

GIS FILE:	SCALE: AS NOTED	DATE: 02/17/2010	Figure 12



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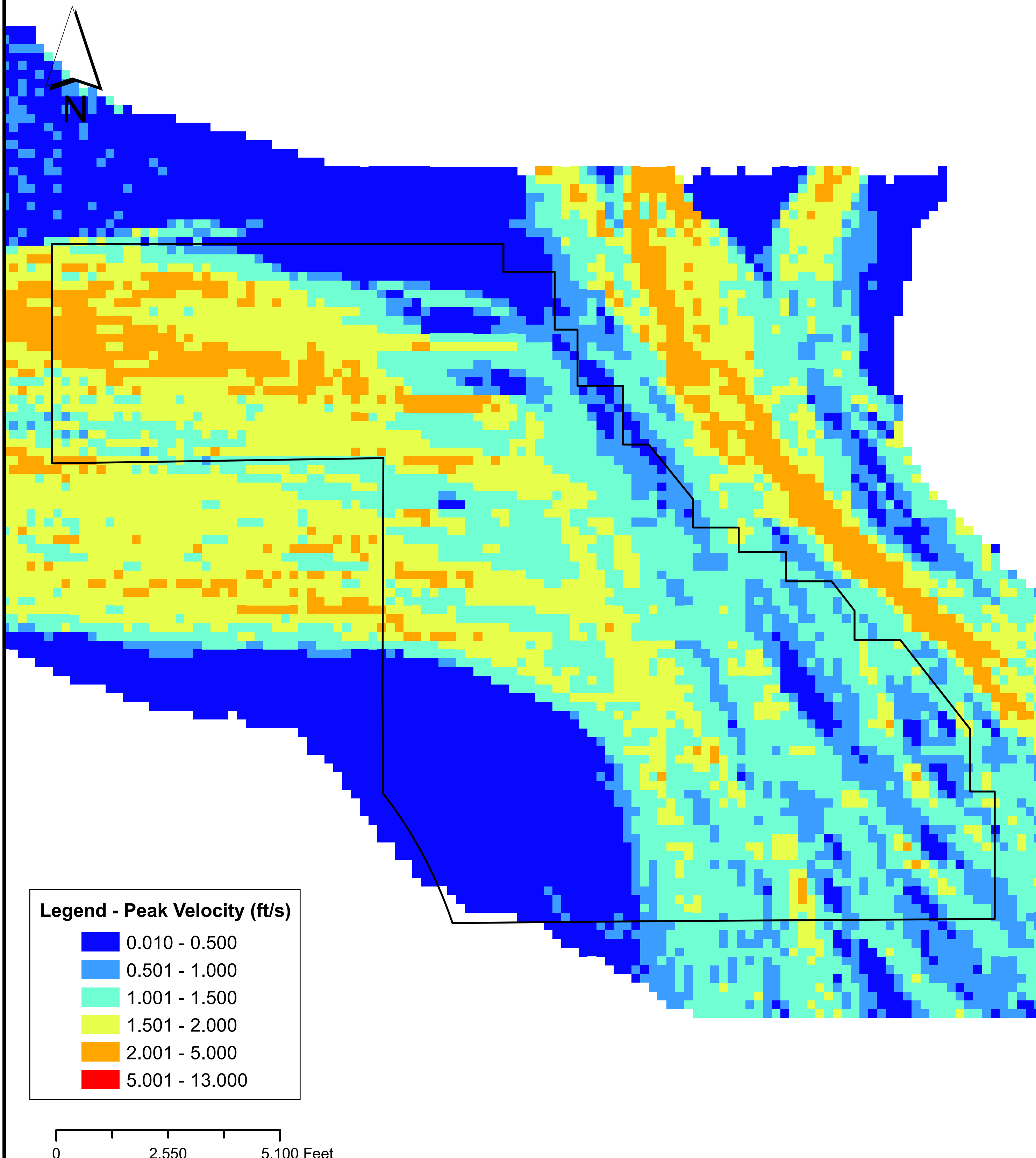
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
10 Year - Change in Peak Flow Depth (Future - Existing)

GIS FILE:	SCALE: AS NOTED	DATE: 03/09/2010	Figure 13



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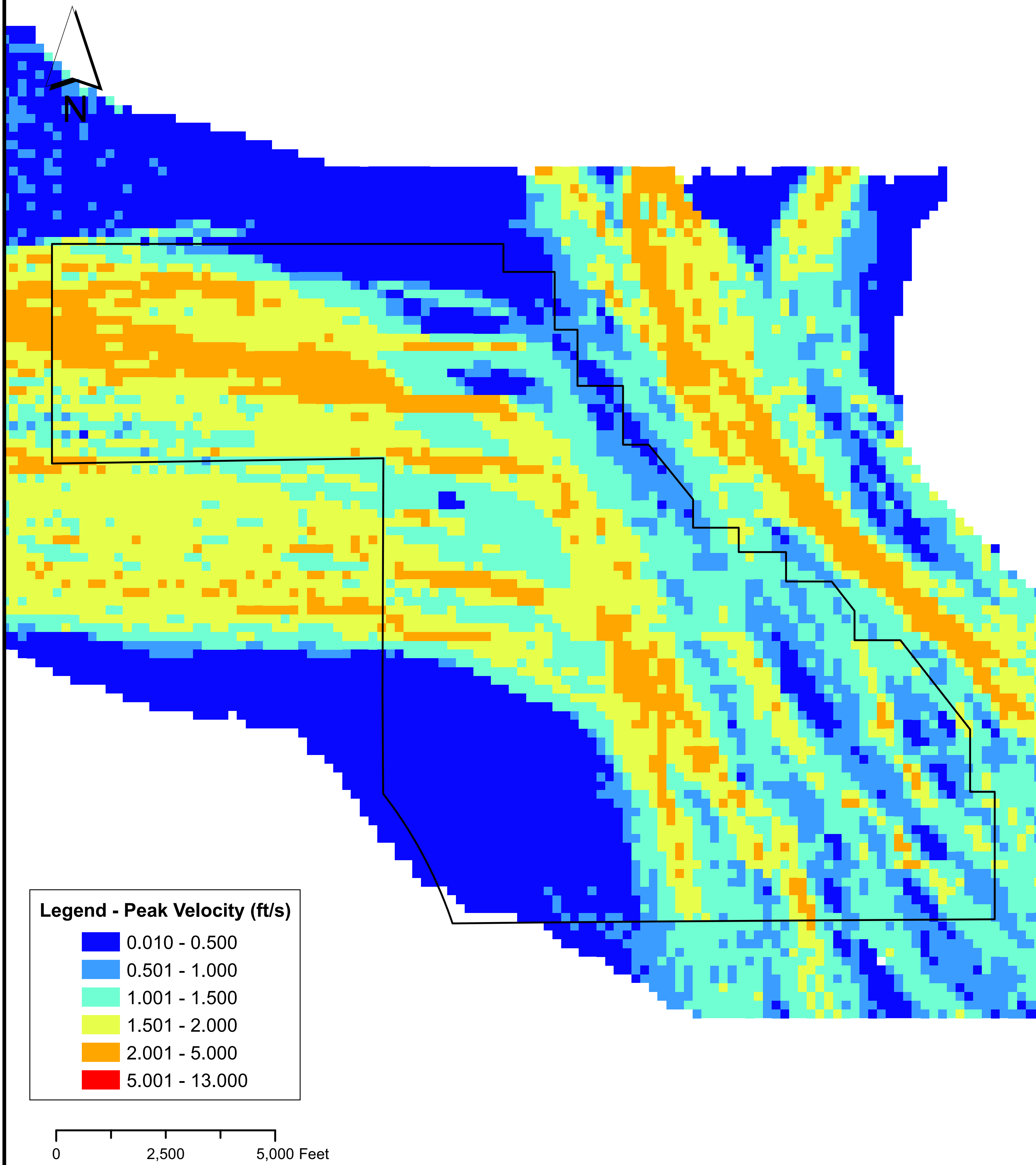
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
10 Year - Existing Conditions Peak Velocity

GIS FILE:	SCALE: AS NOTED	DATE: 03/05/2010	Figure 14



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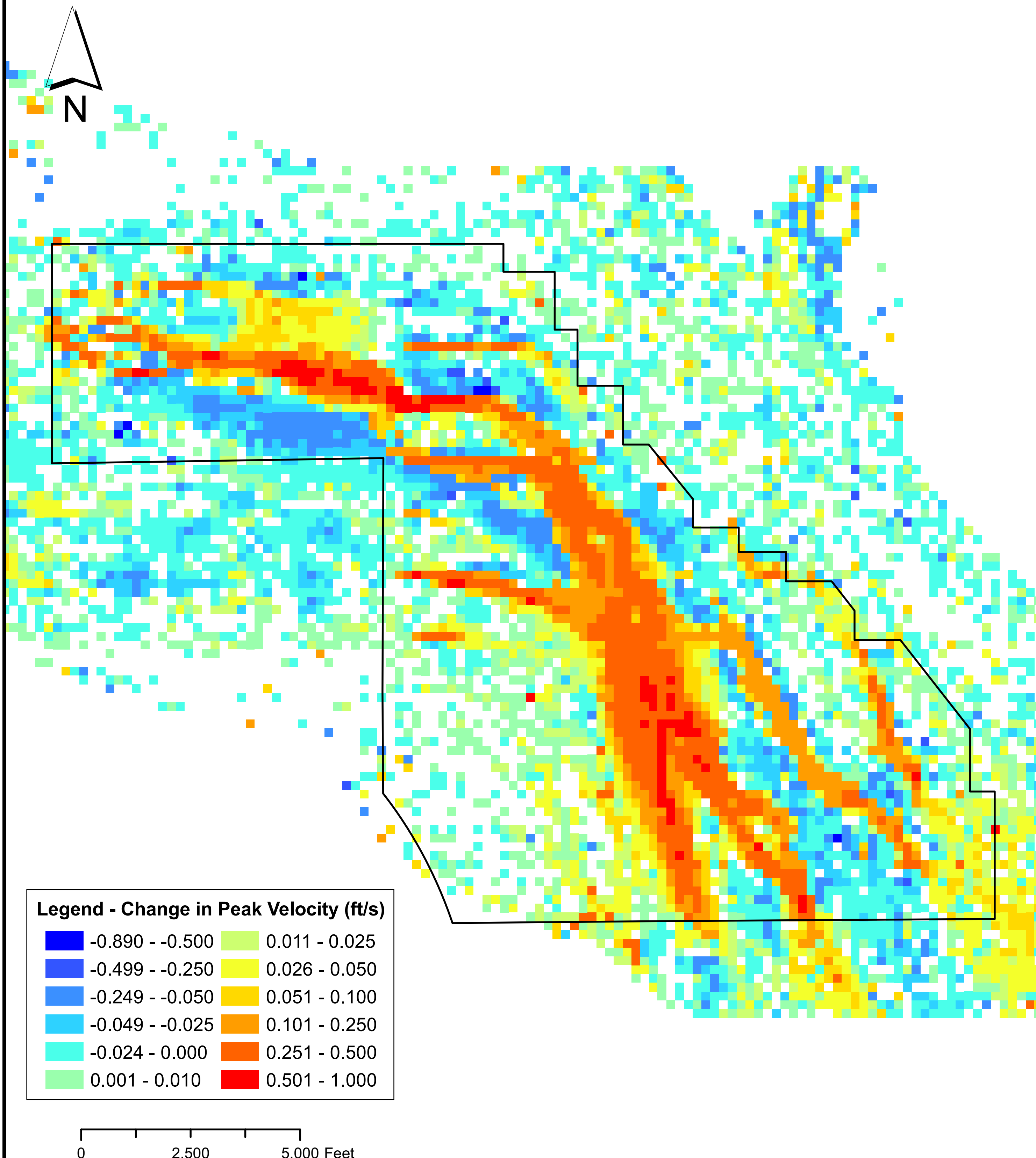
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
10 Year - Future Conditions Peak Velocity

GIS FILE:	SCALE: AS NOTED	DATE: 02/17/2010	Figure 15



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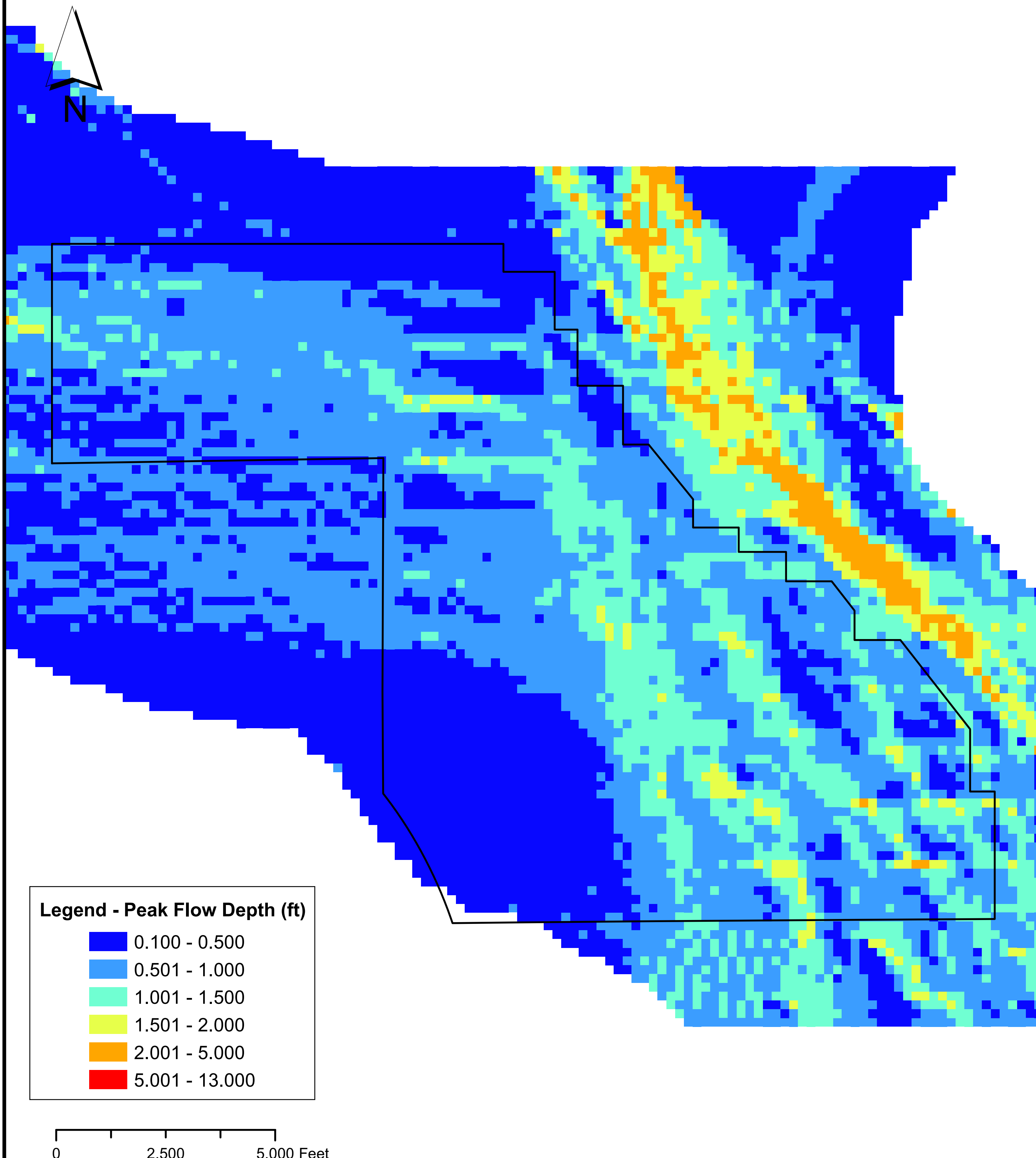
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
10 Year - Change in Peak Velocity (Future - Existing)

GIS FILE:	SCALE: AS NOTED	DATE: 03/09/2010	Figure 16



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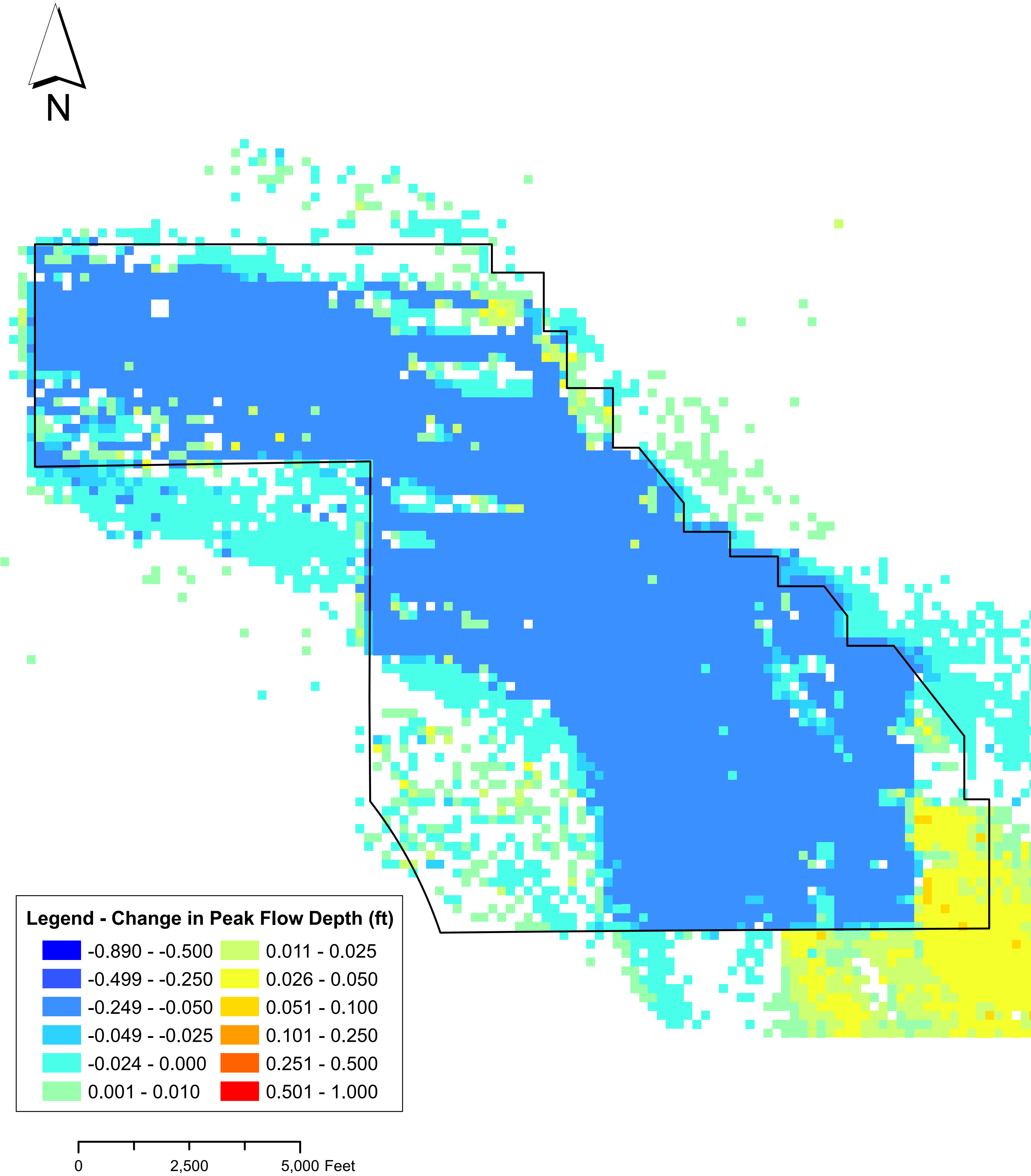
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Future Conditions Decompact Peak Flow Depth

GIS FILE:	SCALE: AS NOTED	DATE: 02/17/2010	Figure 17



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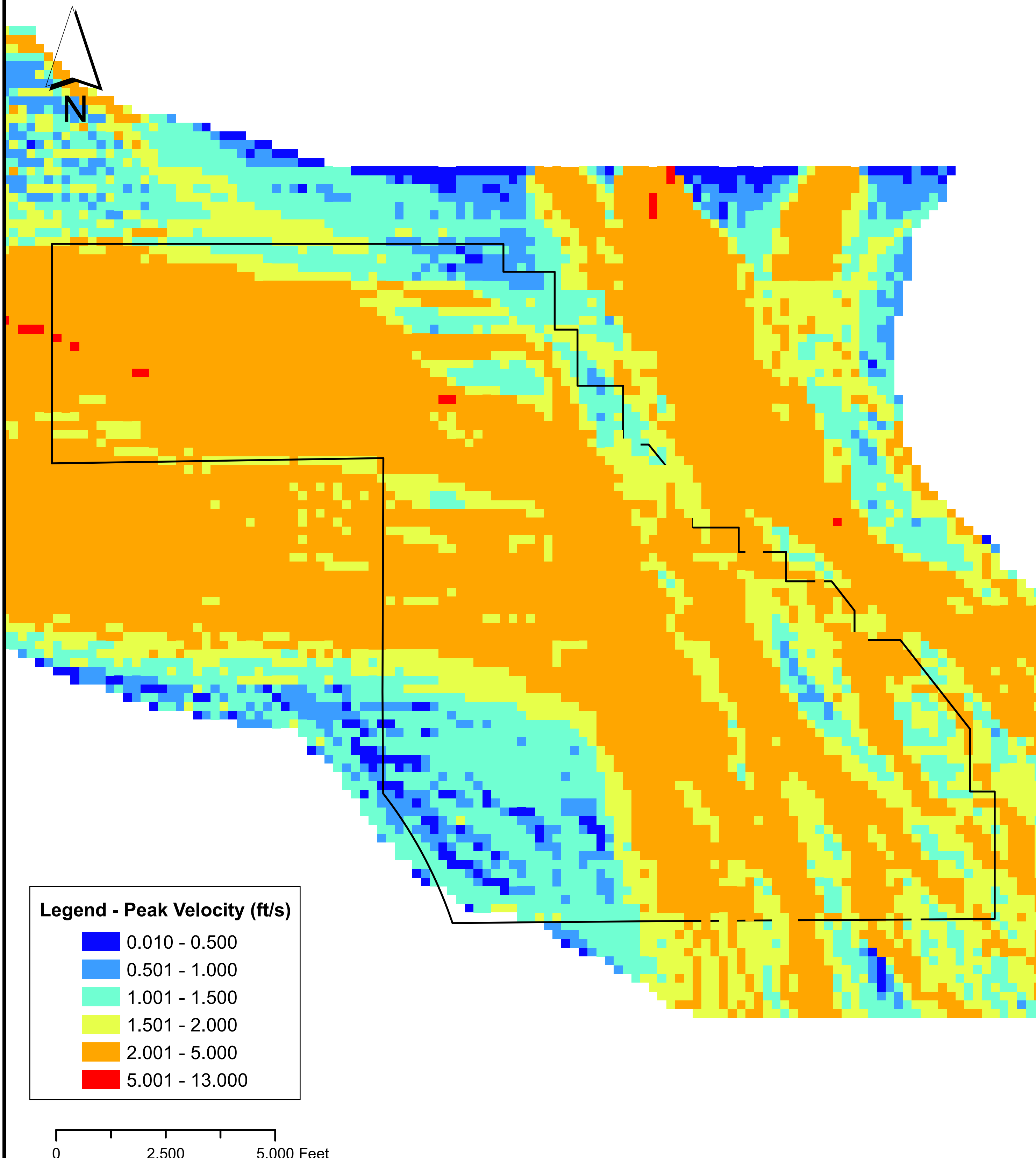
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Change Peak Flow Depth (Future Decomp - Existing)

GIS FILE:	SCALE: AS NOTED	DATE: 03/09/2010	Figure 18



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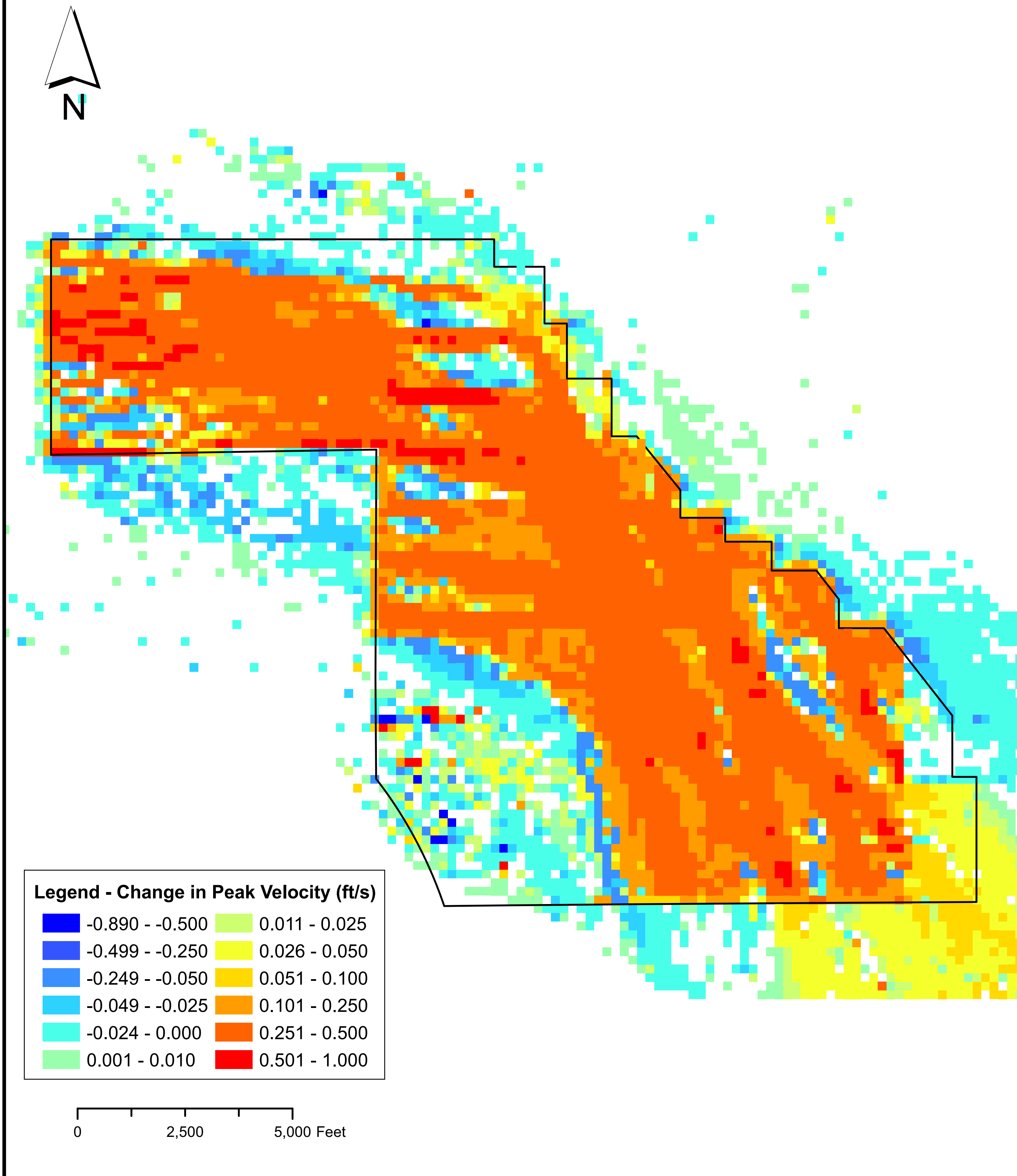
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Future Conditions Decompact Peak Velocity

GIS FILE:	SCALE: AS NOTED	DATE: 02/17/2010	Figure 19



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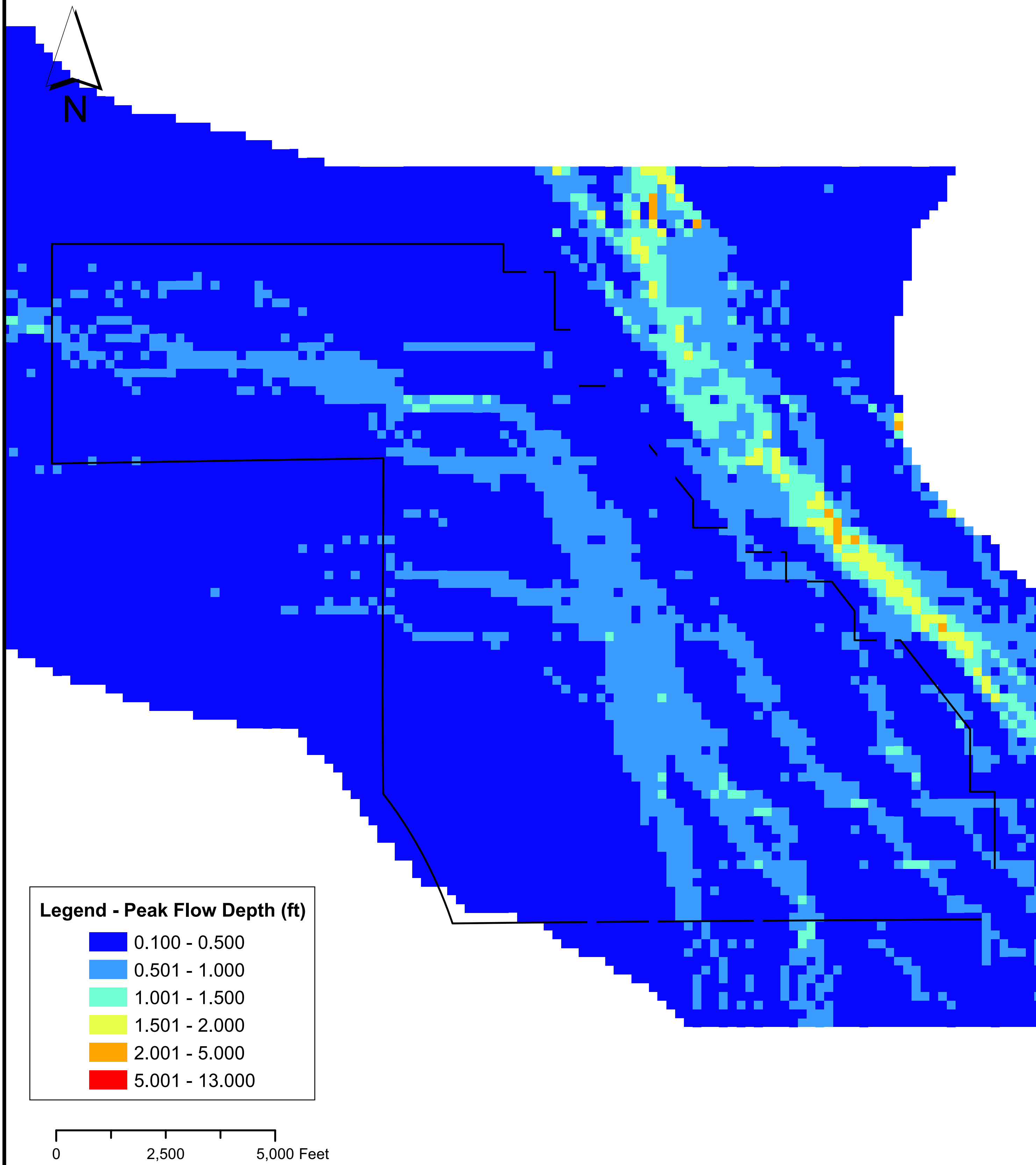
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Change Peak Velocity (Future Decomp - Existing)

GIS FILE:	SCALE: AS NOTED	DATE: 03/09/2010	Figure 20



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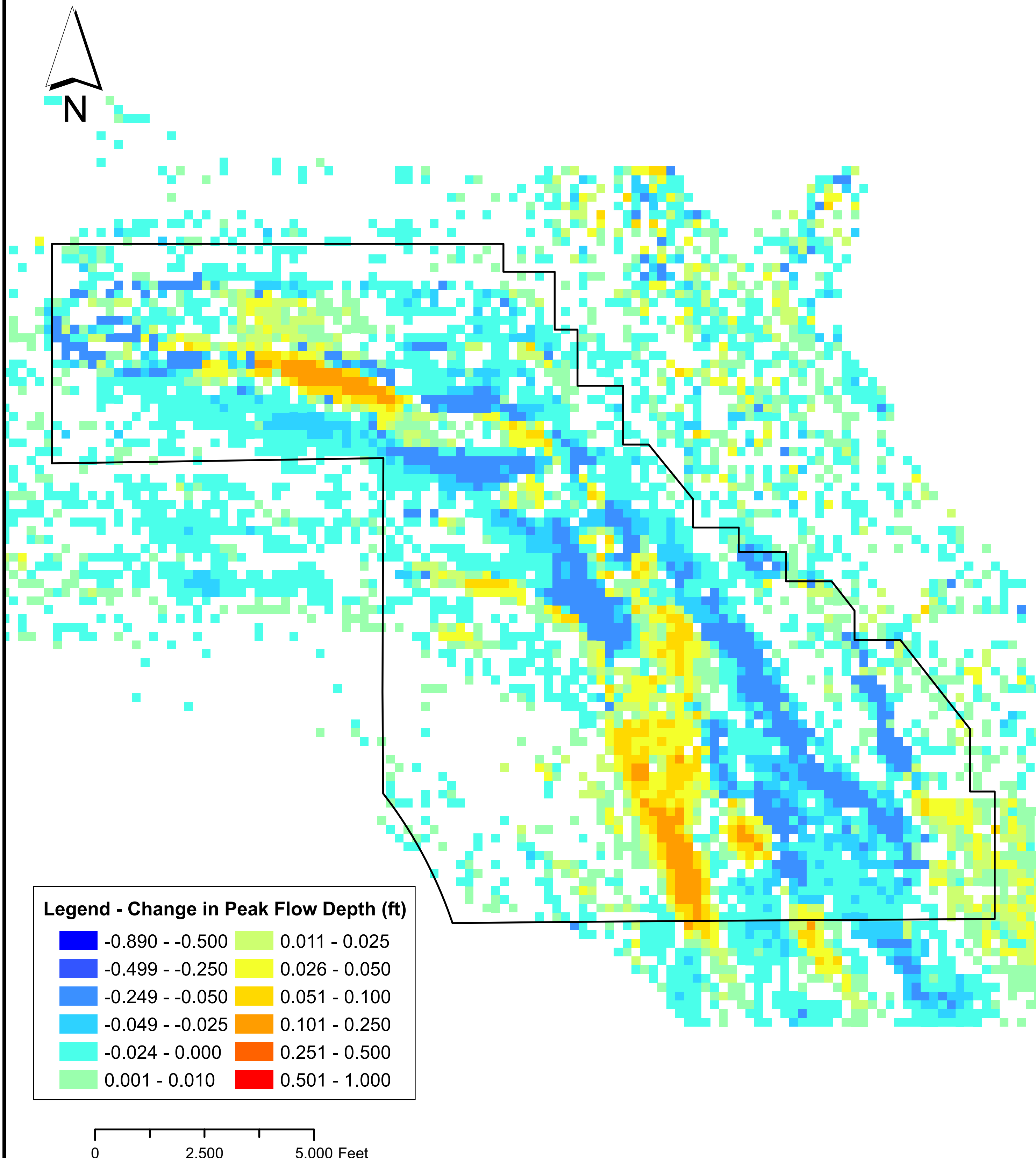
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
10 Year - Future Conditions Decompact Peak Flow Depth

GIS FILE:

SCALE: AS NOTED

DATE: 02/17/2010

Figure 21



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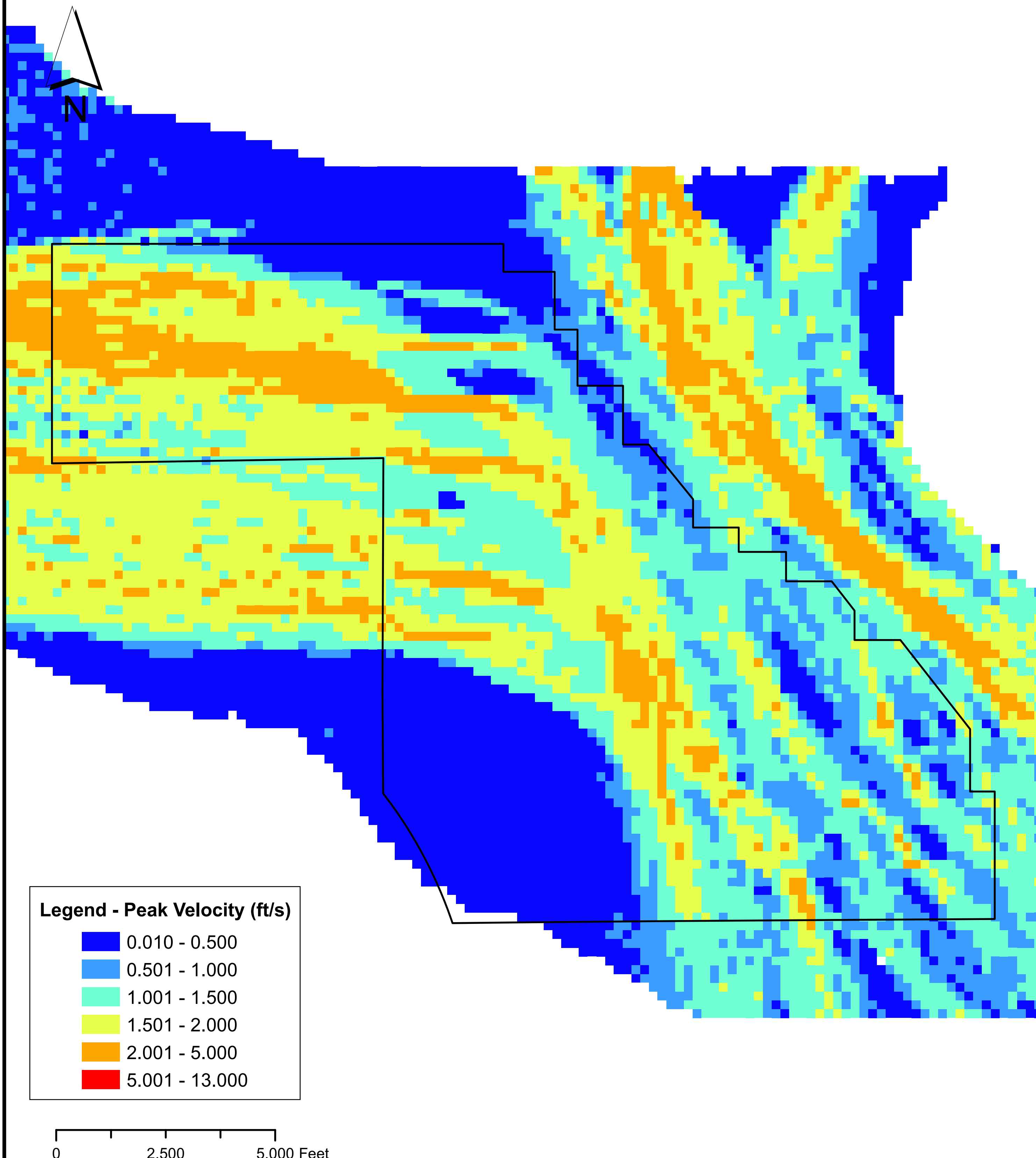
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
10 Year - Change Peak Flow Depth (Future Decomp - Existing)

GIS FILE:	SCALE: AS NOTED	DATE: 03/09/2010	Figure 22



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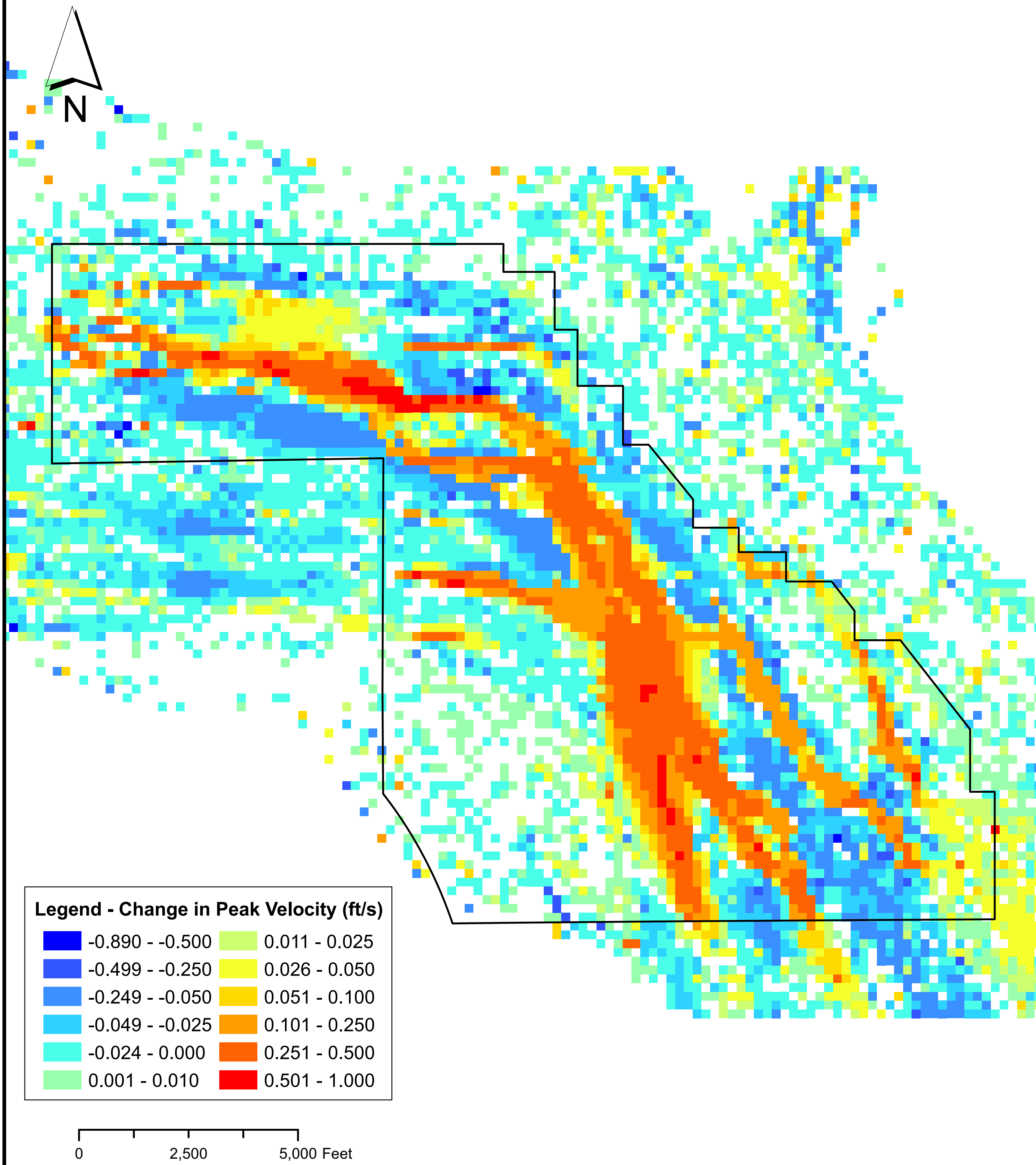
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
10 Year - Future Conditions Decompact Peak Velocity

GIS FILE:	SCALE: AS NOTED	DATE: 02/17/2010	Figure 23



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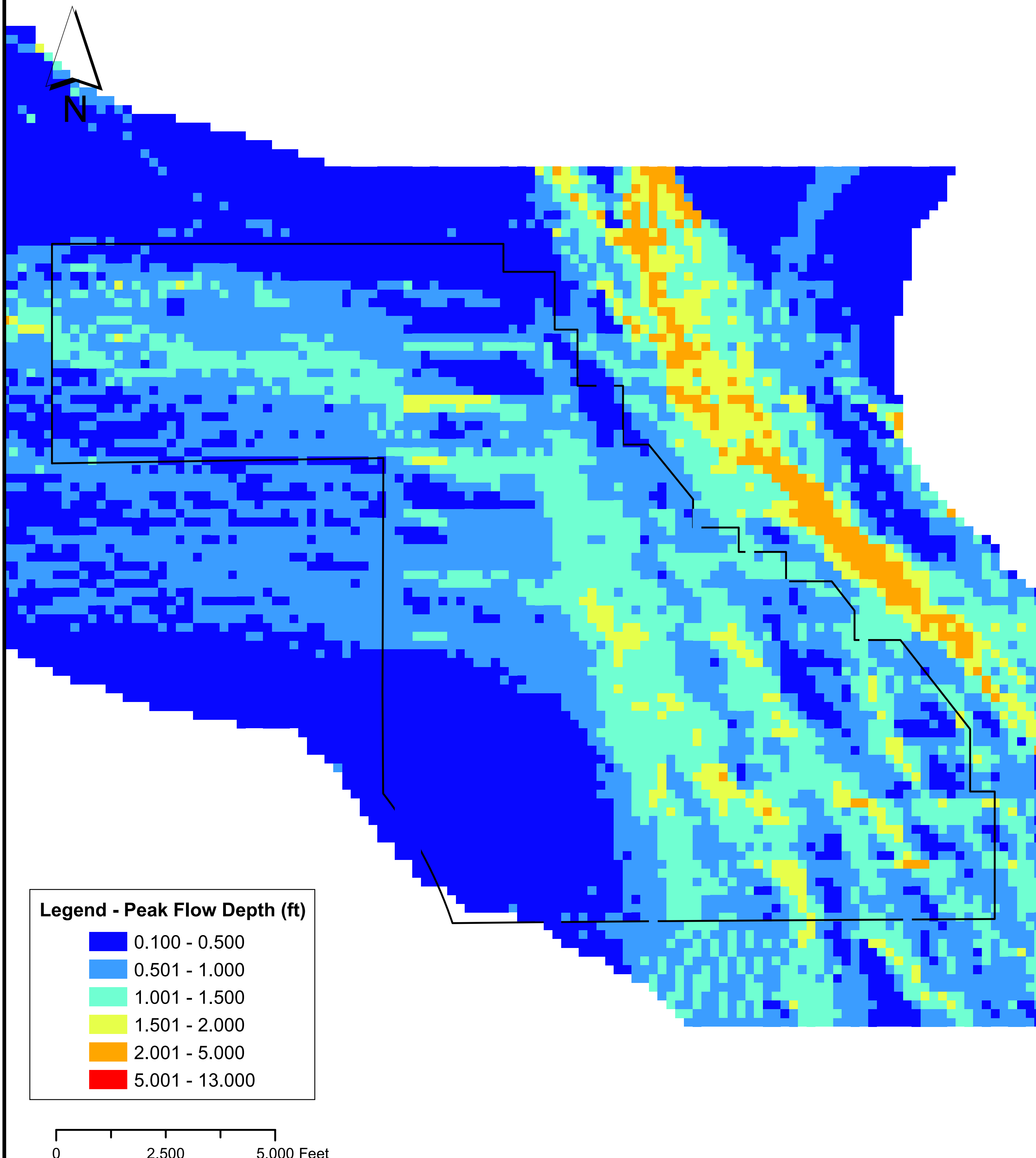
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
10 Year - Change Peak Velocity (Future Decomp - Existing)

GIS FILE:	SCALE: AS NOTED	DATE: 03/09/2010	Figure 24



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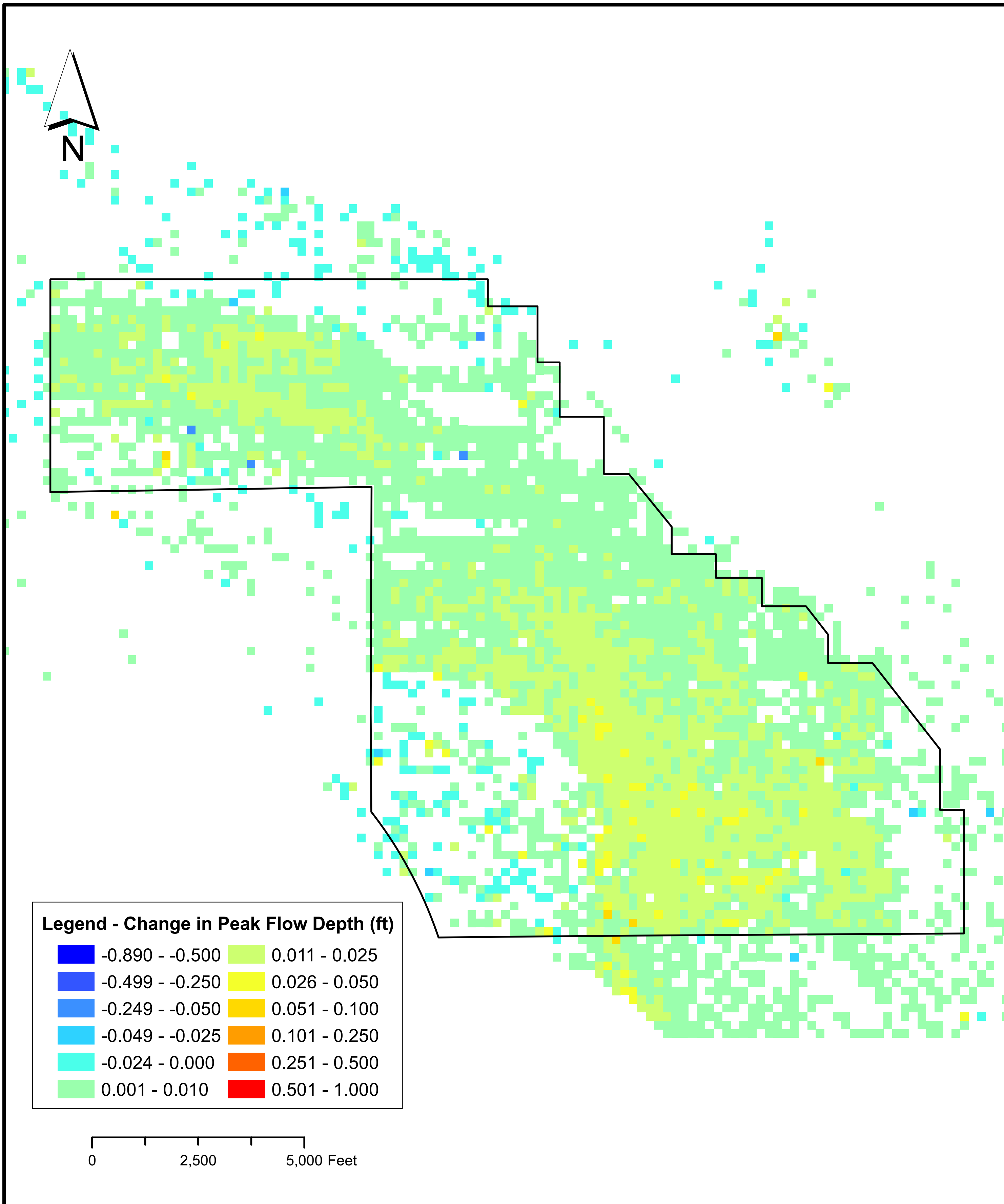
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Future Conditions with Rip-Rap Flow Depth

GIS FILE:	SCALE: AS NOTED	DATE: 02/17/2010	Figure 25



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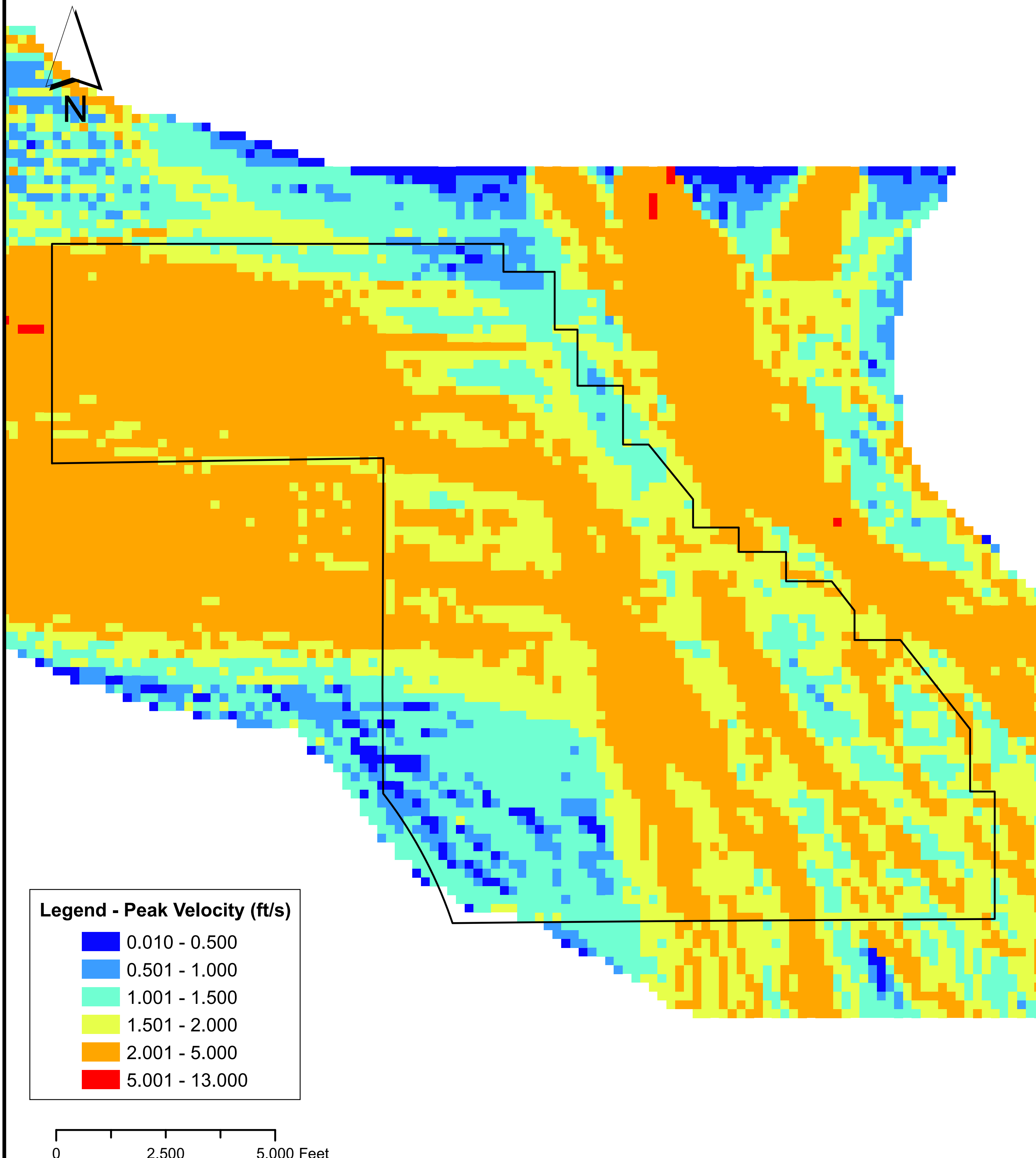
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Change in Peak Flow Depth (Future Rip-Rap - Existing)

GIS FILE:	SCALE: AS NOTED	DATE: 03/09/2010	Figure 26



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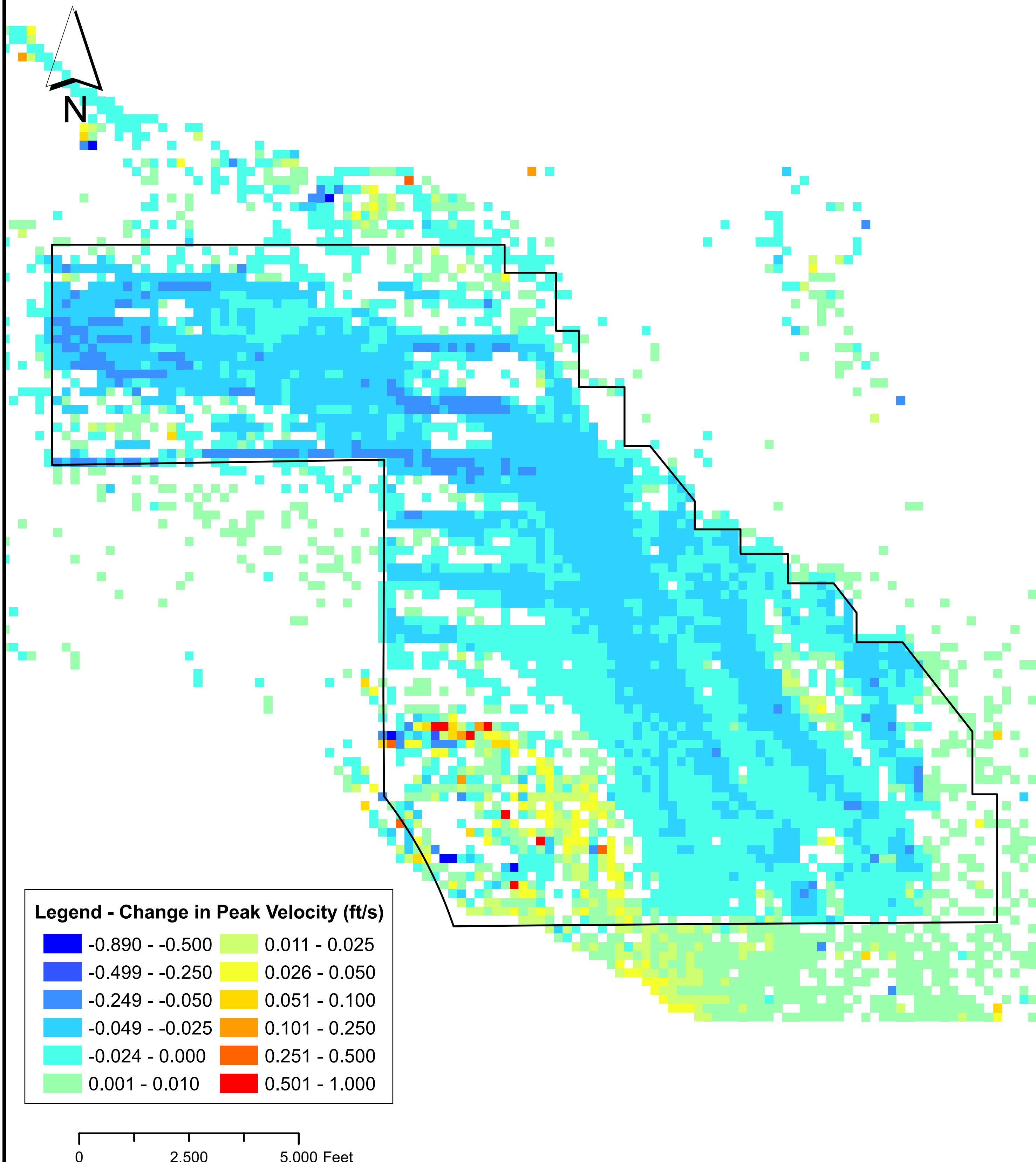
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Future Conditions Rip-Rap Peak Velocity

GIS FILE:	SCALE: AS NOTED	DATE: 02/17/2010	Figure 27



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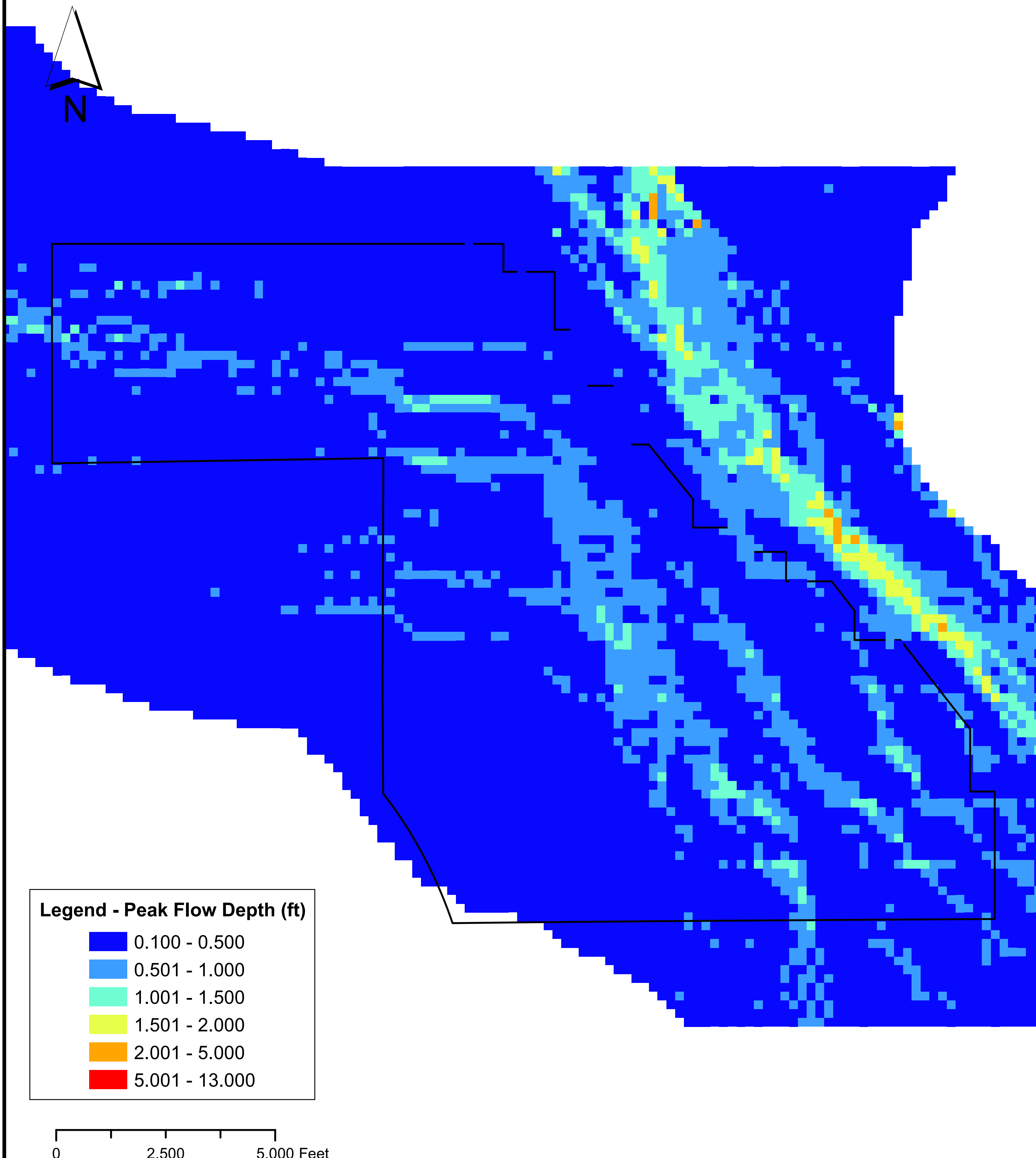
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Change in Peak Velocity (Future Rip-Rap - Existing)

GIS FILE:	SCALE: AS NOTED	DATE: 03/09/2010	Figure 28



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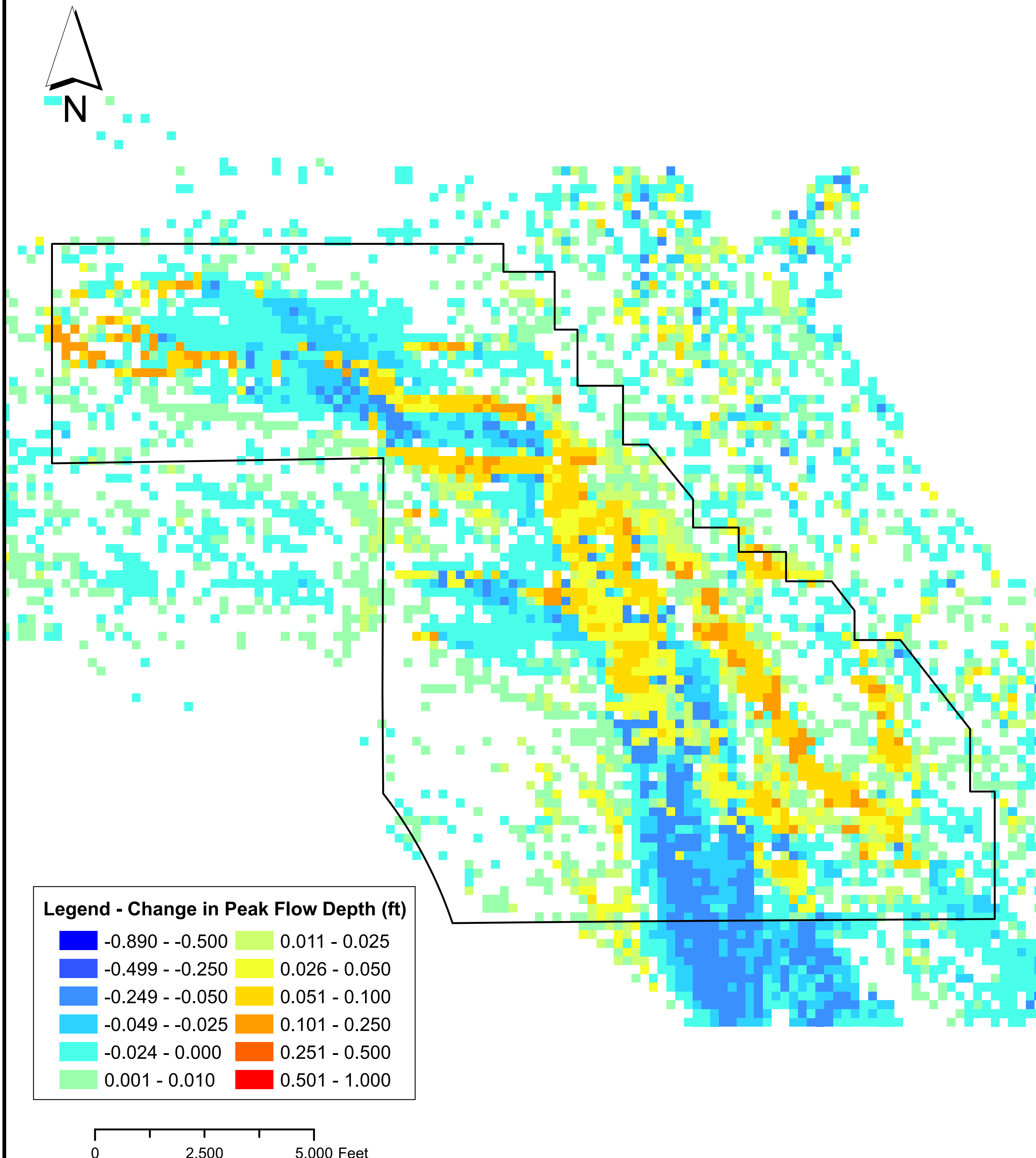
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
10 Year - Future Rip-Rap Conditions Peak Flow Depth

GIS FILE:	SCALE: AS NOTED	DATE: 02/17/2010	Figure 29



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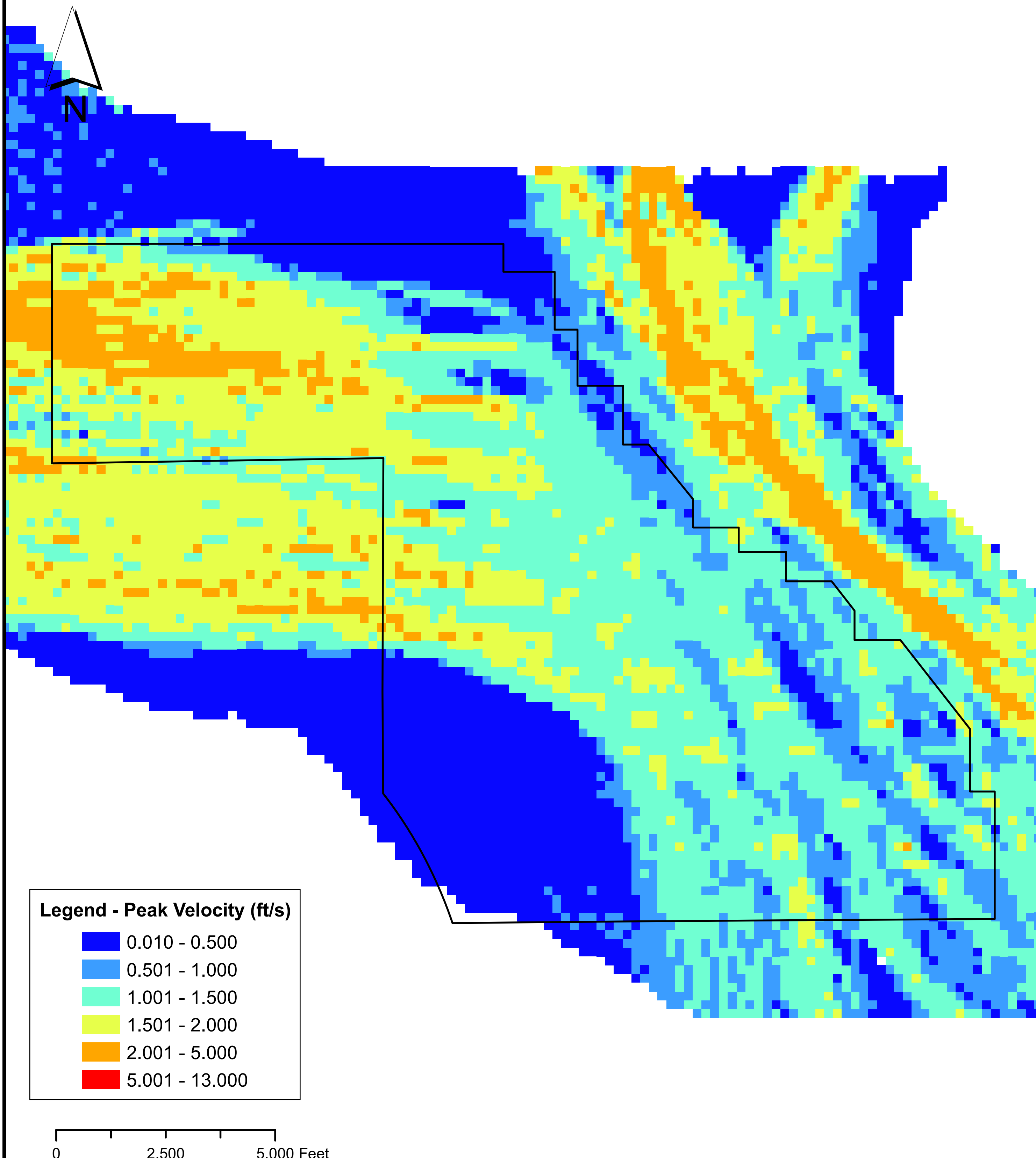
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
10 Year - Change in Peak Flow Depth (Future Rip-Rap - Existing)

GIS FILE:	SCALE: AS NOTED	DATE: 03/09/2010	Figure 30



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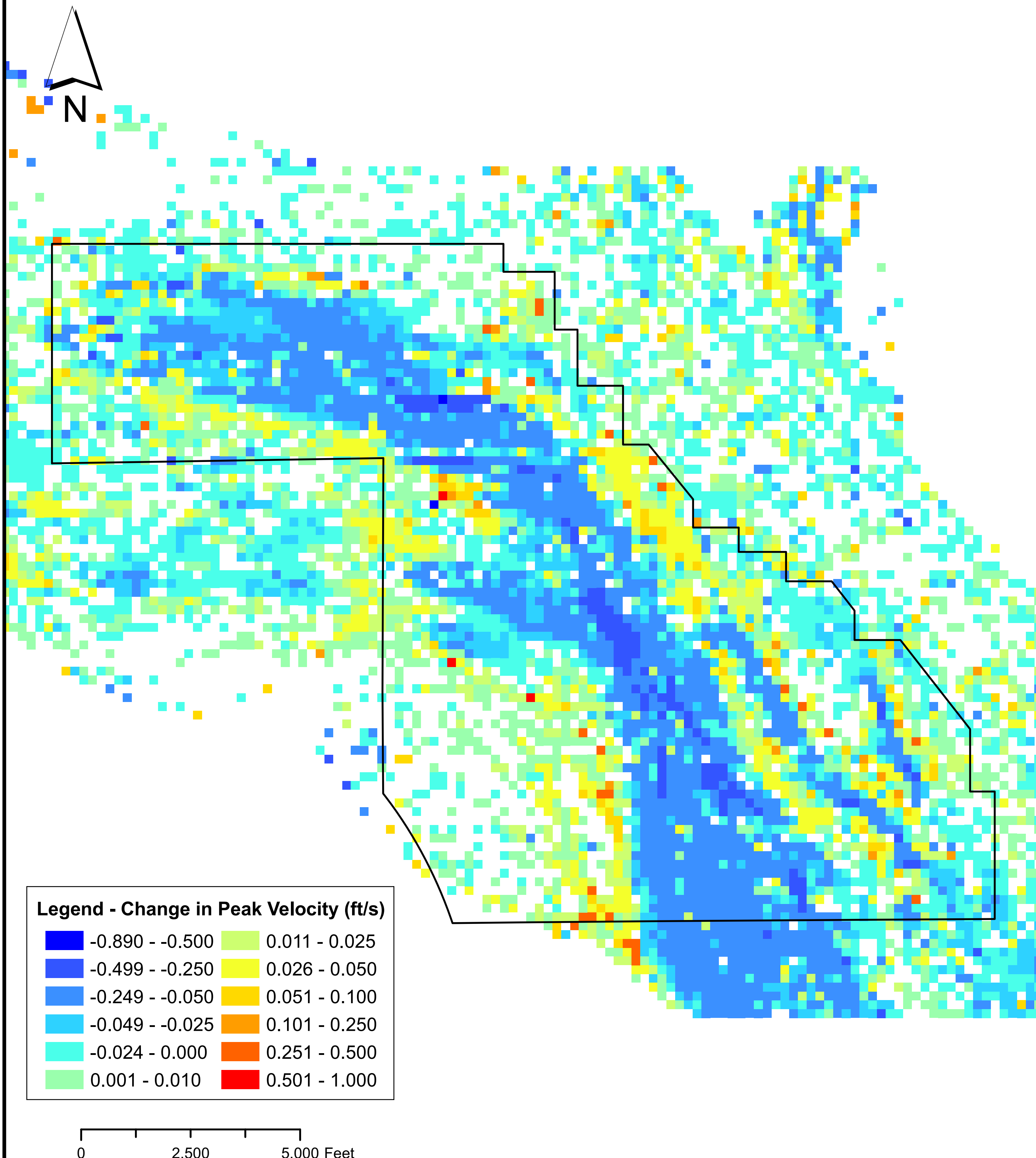
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
10 Year - Future Rip-Rap Conditions Peak Velocity

GIS FILE:	SCALE: AS NOTED	DATE: 02/17/2010	Figure 31



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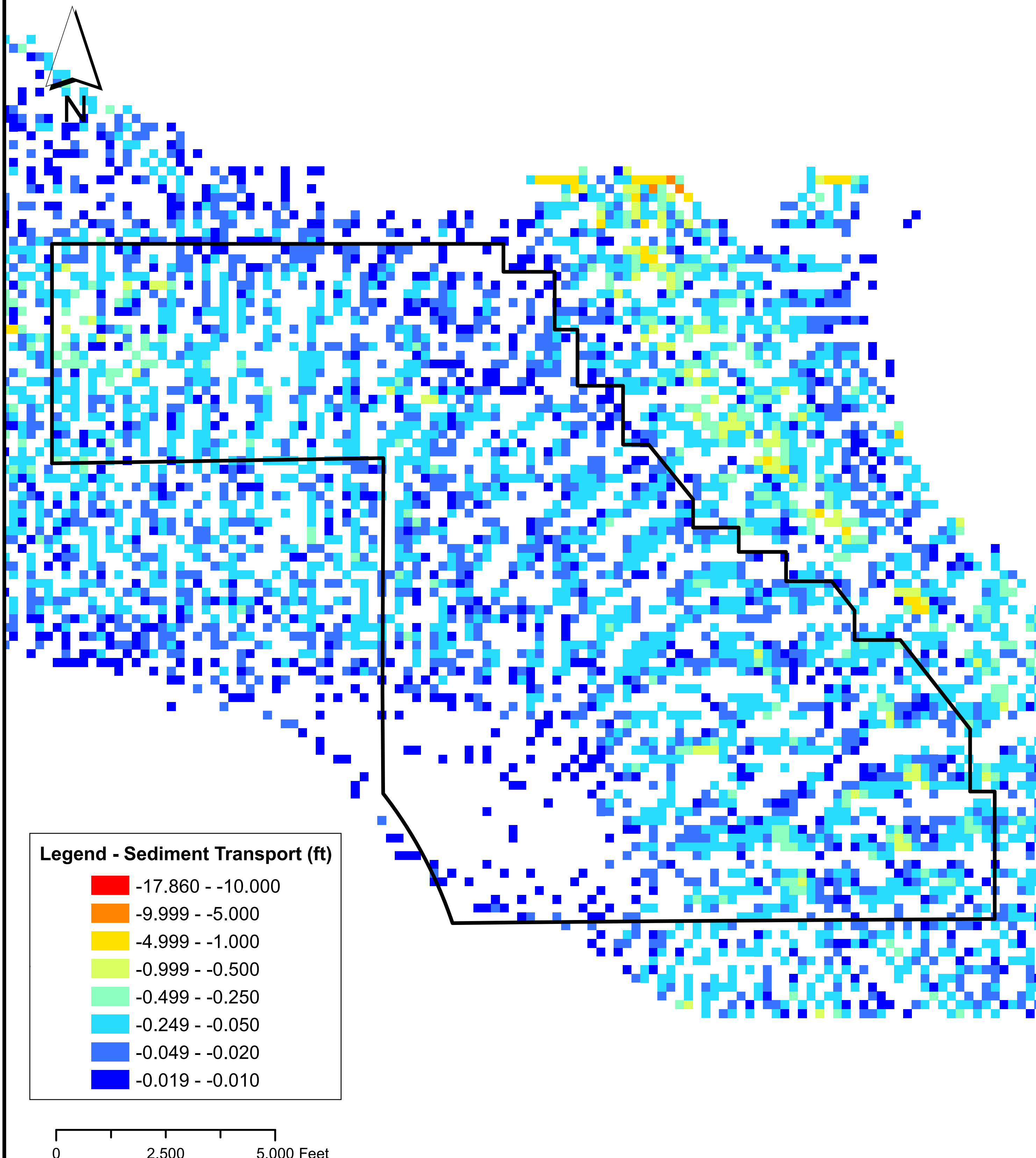
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
10 Year - Change in Peak Velocity (Future Rip-Rap - Existing)

GIS FILE:	SCALE: AS NOTED	DATE: 03/09/2010	Figure 32



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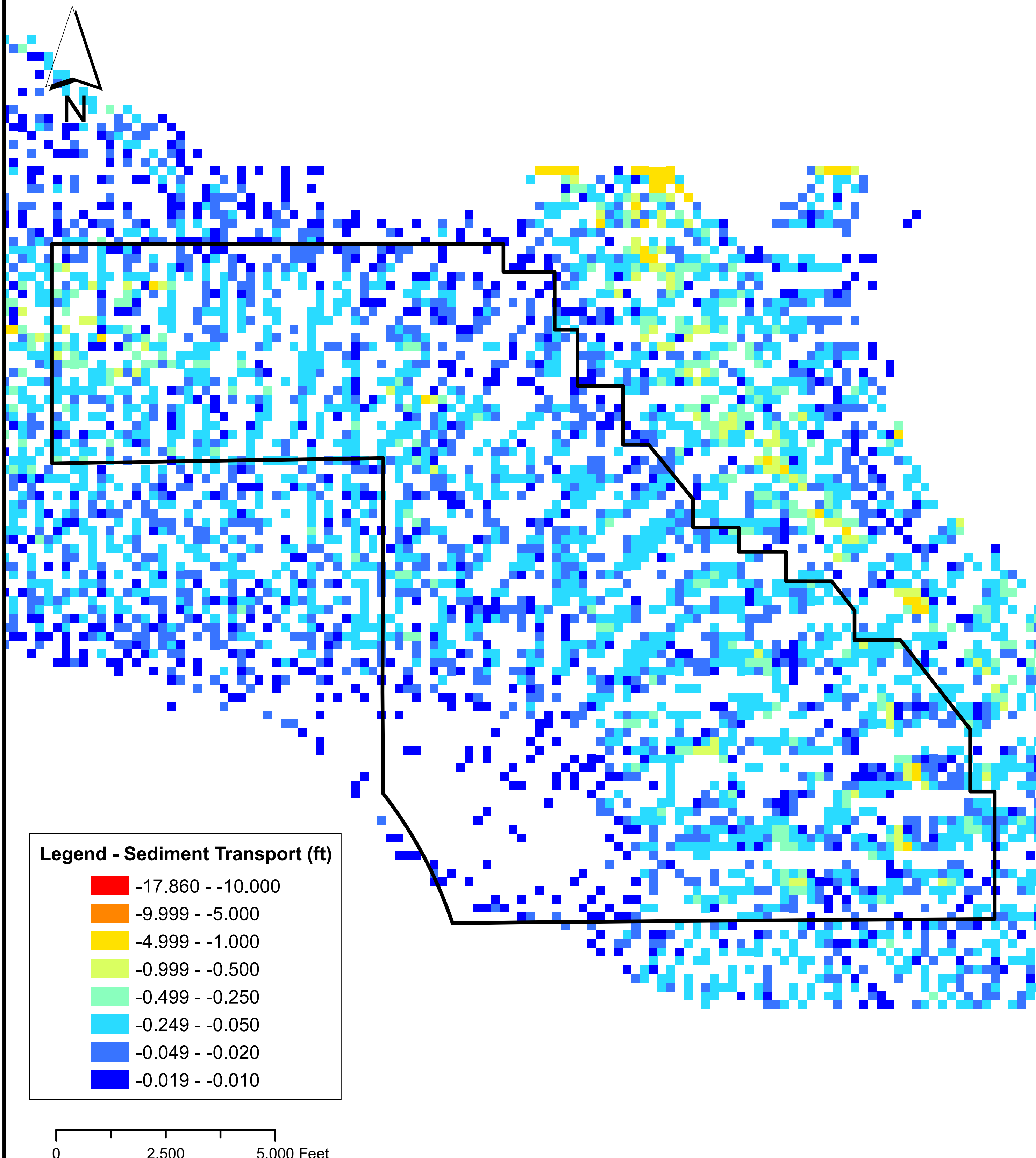
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Existing Conditions Maximum Sediment Transport

GIS FILE:	SCALE: AS NOTED	DATE: 03/11/2010	Figure 33



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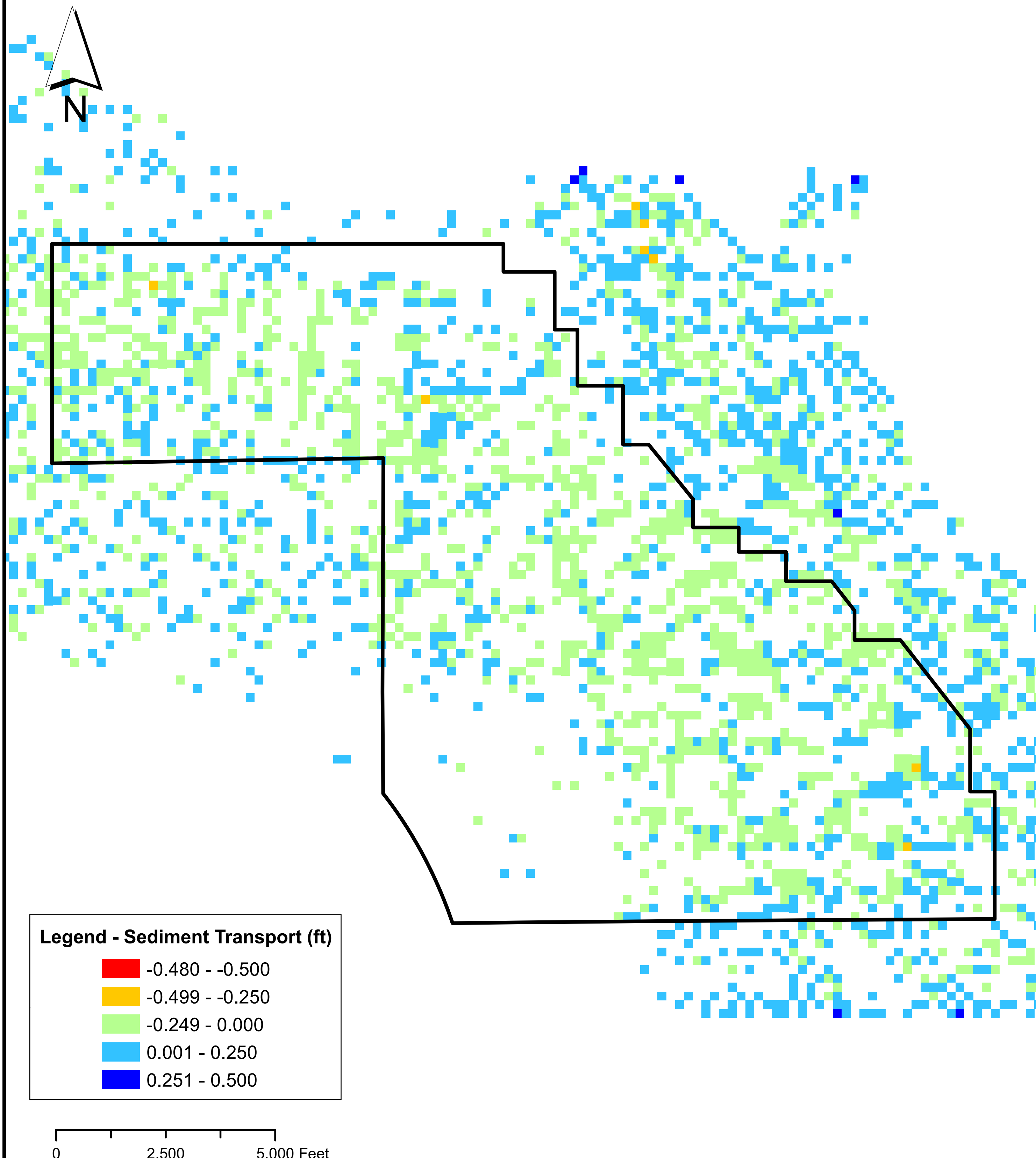
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Future Conditions Maximum Sediment Transport

GIS FILE:	SCALE: AS NOTED	DATE: 02/17/2010	Figure 34



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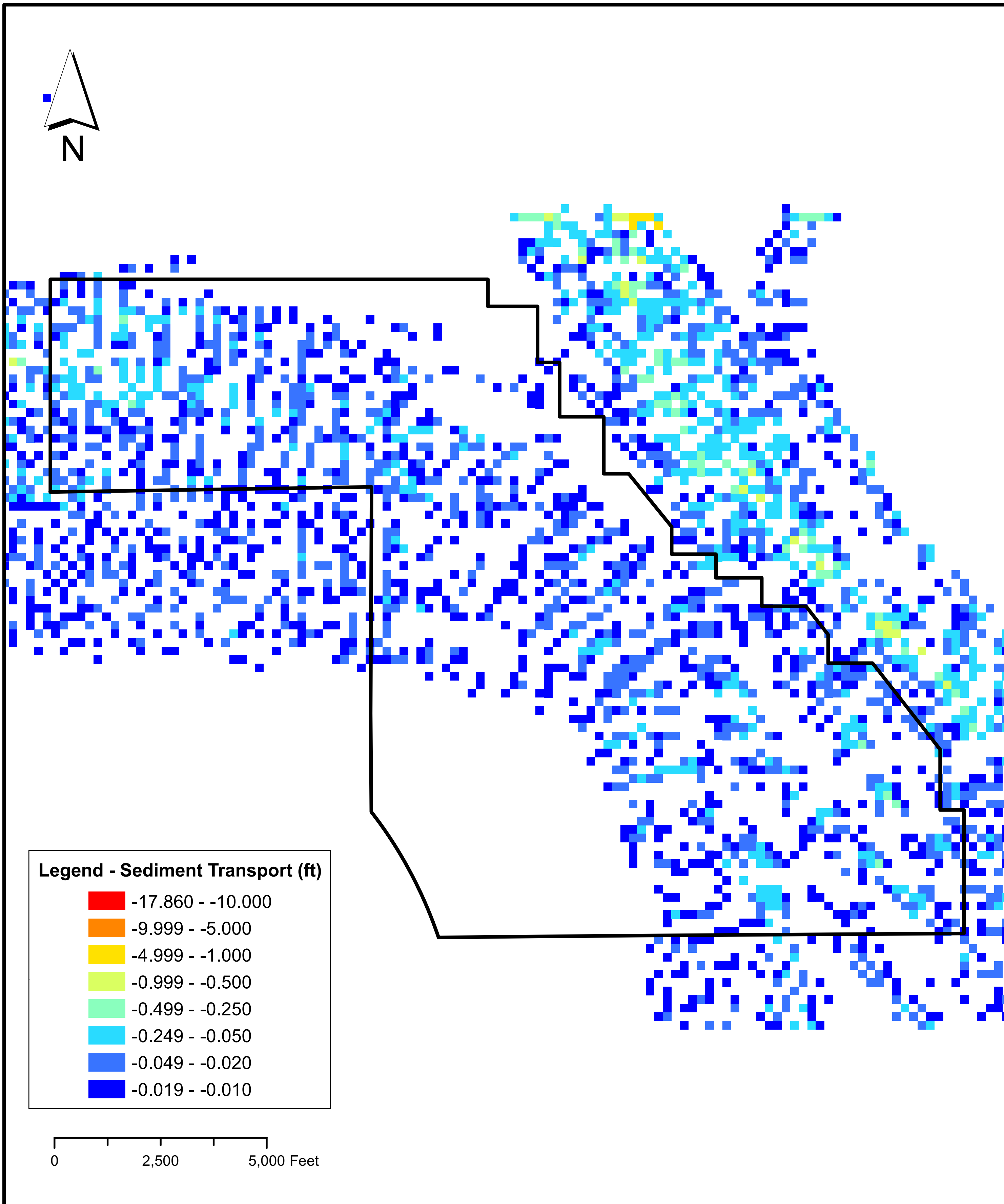
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Max Sediment Transport Change (Future - Existing)

GIS FILE: | SCALE: AS NOTED | DATE: 03/11/2010 | Figure 35



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DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Existing Conditions Maximum Sediment Transport

GIS FILE:

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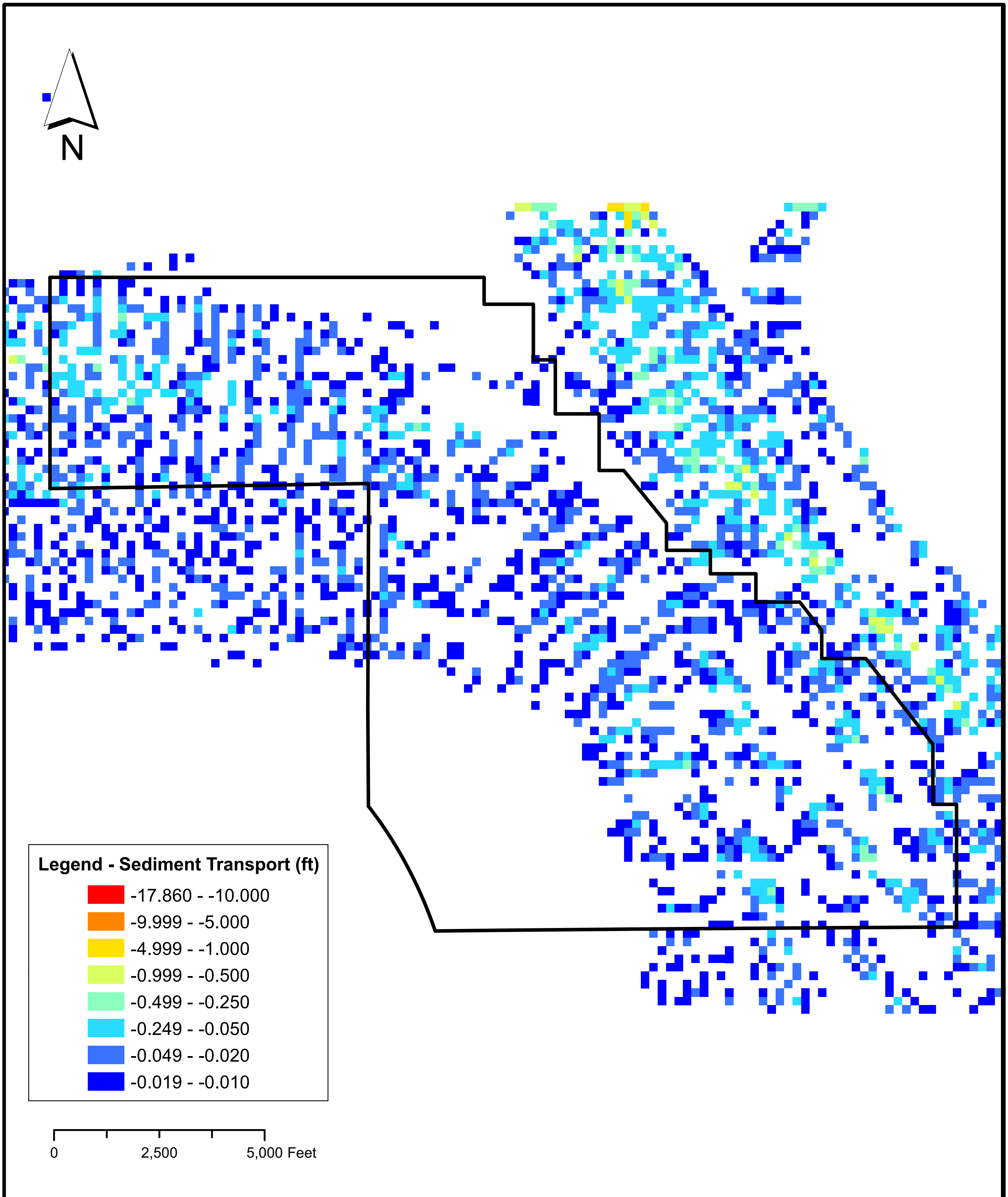
DATE: 03/11/2010

Figure 36



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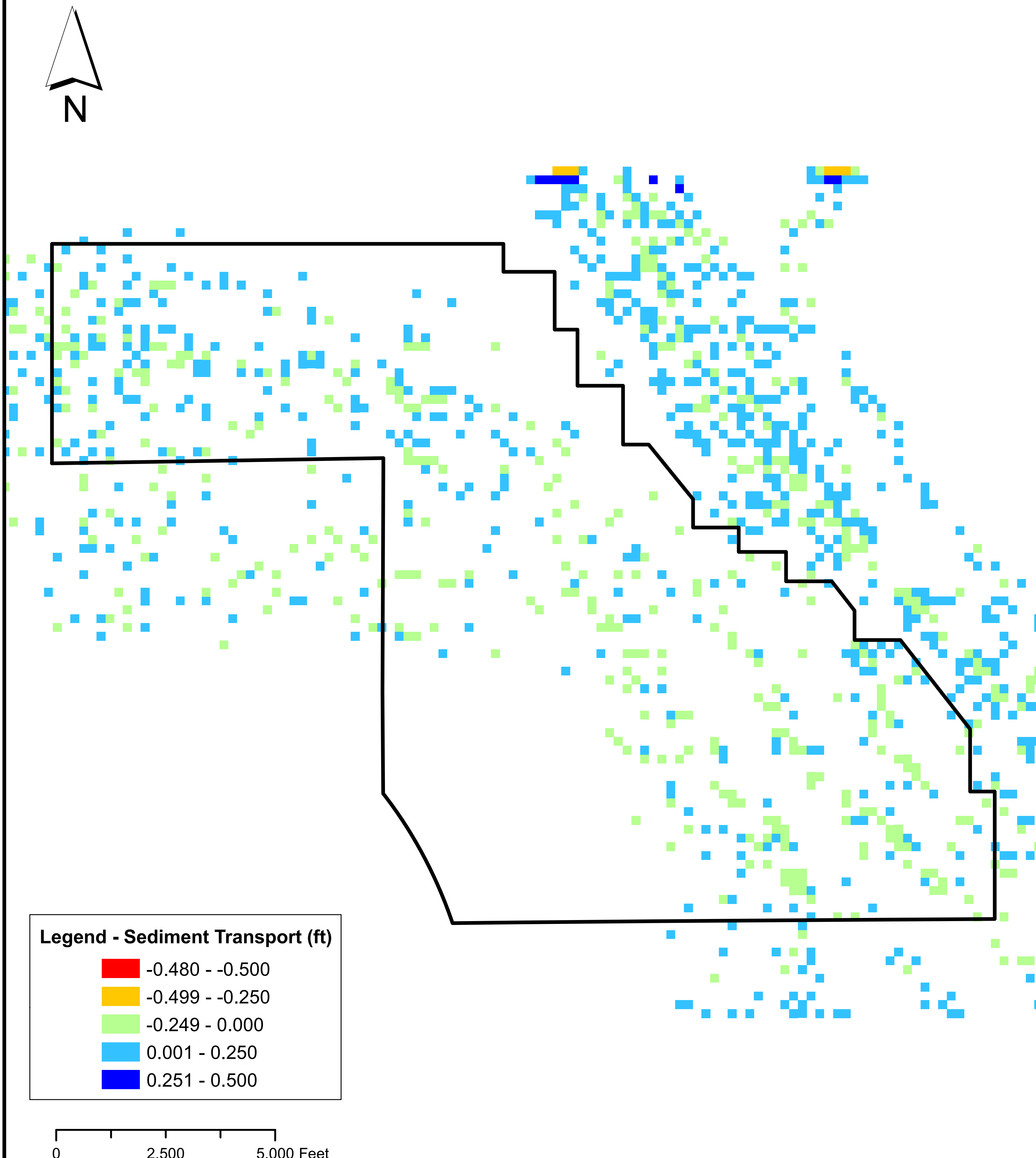
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
10 Year - Future Conditions Maximum Sediment Transport

GIS FILE:	SCALE: AS NOTED	DATE: 02/17/2010	Figure 37



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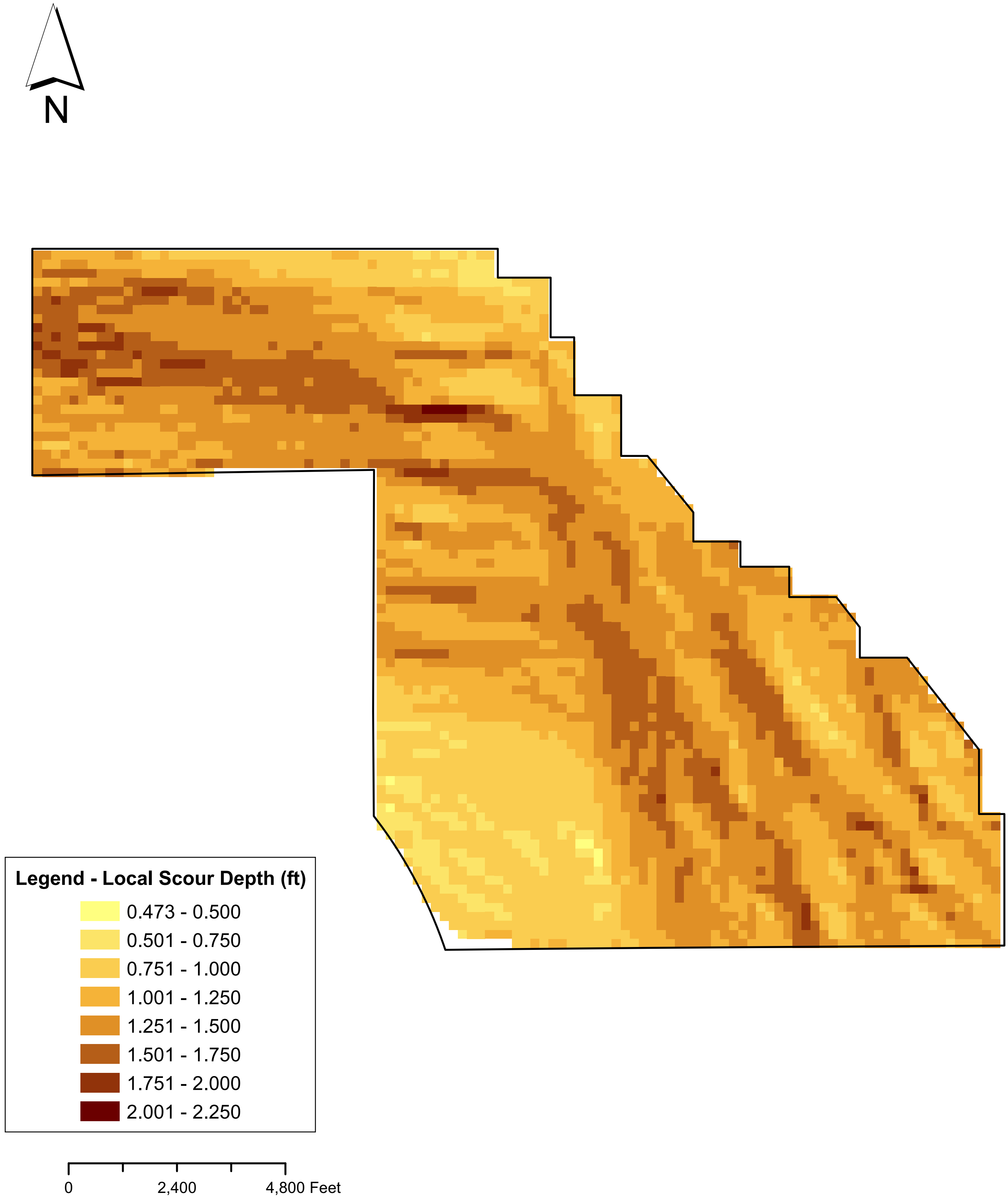
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
10 Year - Max Sediment Transport Change (Future - Existing)

GIS FILE:	SCALE: AS NOTED	DATE: 03/11/2010	Figure 38



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DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Local Scour Depth (Worst Case Scour)

GIS FILE:	SCALE: AS NOTED	DATE: 02/17/2010	Figure 39

Appendix A: Hydrologic Analysis Supporting Data

U.S. Department of Agriculture, Natural Resources Conservation Service, Urban Hydrology for Small Watersheds, Technical Release 55 (TR-55) dated June 1986 was used to estimate runoff/infiltration characteristics. Following is the table from TR-55 that contains the curve numbers used in this analysis.

Table 2-2a Runoff curve numbers for urban areas [1]

Cover description: Cover type and hydrologic condition	Average percent impervious area [2]	Curve numbers for hydrologic soil group: A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

1 Average runoff condition, and $I_a = 0.2S$.
2 The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.
3 CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.
4 Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.
5 Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4 based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

Appendix B: Hydraulic Analysis Supporting Data

Manning's n value was used to describe surface roughness. The roughness was calculated as shown below:

Estimate for existing conditions n

Coarse Sand Floodplain	0.03
Minor irregularities	0.003
Small-Medium Vegetation	0.01
Total	**0.043**

Estimate for future conditions n

Coarse Sand Floodplain	0.03
Add Poles/Obstructions	0.004
Total	**0.034**

The addition of poles and other obstructions was assumed to be negligible to minor; occupying between 5% and 15% of the cross-sectional area.

Estimate for six (6)-inch rip-rap n

Cobble	0.039
Add Poles/Obstructions	0.004
Total	**0.043**

In order to achieve a roughness of 0.039 for the 100-year future conditions approximately 54% of the project site would need to be covered in six (6) inch diameter rip-rap. The roughness value of 0.057 for the 10-year storm event cannot be obtained with six (6) inch rip-rap

The methodology is based on the following USGS methodology:

Arcement, Jr., G.J., Schneider, V.R., "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains," United States Geological Survey Water-Supply Paper 2339.

Appendix C: Sediment Transport Analysis Supporting Data

Zeller and Fullerton equation was used in sediment transport modeling within the Flo2D modeling software framework. Flo2D model user manual states the following:

"Zeller-Fullerton Equation. Zeller-Fullerton is a multiple regression sediment transport equation for a range of channel bed and alluvial floodplain conditions. This empirical equation is a computer generated solution of the Meyer-Peter, Muller bed-load equation combined with Einstein's suspended load to generate a bed material load (Zeller and Fullerton, 1983). The bed material discharge qs is calculated in cfs per unit width as follows:

qs = 0.0064 n1.77 V4.32 G0.45 d-0.30 D50-0.61

where n is Manning's roughness coefficient, V is the mean velocity, G is the gradation coefficient, d is the hydraulic depth and D50 is the median sediment diameter. All units in this equation are in the ft-lb-sec system except D50, which is in millimeters. For a range of bed material from 0.1 mm to 5.0 mm and a gradation coefficient from 1.0 to 4.0, Julien (1995) reported that this equation should be accurate with 10% of the combined Meyer-Peter Muller and Einstein equations. The Zeller-Fullerton equation assumes that all sediment sizes are available for transport (no armoring). The original Einstein method is assumed to work best when the bedload constitutes a significant portion of the total load (Yang, 1996)."

Also the Flo2D model user manual recommends the following:

"Summary. Yang (1996) made several recommendations for the application of total load sediment transport formulas in the absence of measured data. These recommendations have been expanded to all the equations in the FLO-2D and are slightly edited:

- *Use Zeller and Fullerton equation when the bedload is a significant portion of the total load.*
- *Use Toffaleti's method for large sand-bed rivers.*
- *Use Yang's equation for sand and gravel transport in natural rivers.*
- *Use Ackers-White or Engelund-Hansen equations for subcritical flow in lower sediment transport regime.*
- *Use Lausen's formula for shallow rivers with silt and fine sand.*
- *Use MPM-Woo's relationship for steep slope, arroyo sand bed channels and alluvial fans. "*

Appendix D: Fluvial Geomorphology Analysis Supporting Data

The following historical aerial photos were used in studying fluvial geomorphology of the Project site.

Desert Solar Farm Site
Riverside County
Desert Center, CA 92239

Inquiry Number: 2624402.1
October 30, 2009

The EDR Aerial Photo Decade Package

EDR® Environmental Data Resources Inc

440 Wheelers Farms Road
Milford, CT 06461
800.352.0050
www.edrnet.com

EDR Aerial Photo Decade Package

Environmental Data Resources, Inc. (EDR) Aerial Photo Decade Package is a screening tool designed to assist environmental professionals in evaluating potential liability on a target property resulting from past activities. EDRs professional researchers provide digitally reproduced historical aerial photographs, and when available, provide one photo per decade.

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Thank you for your business.
Please contact EDR at 1-800-352-0050
with any questions or comments.

Date EDR Searched Historical Sources:
Aerial Photography October 30, 2009

Target Property:
Riverside County
Desert Center, CA 92239

Year	***Scale***	***Details***	***Source***
1978	Aerial Photograph. Scale: 1"=1000'	Flight Year: 1978	Nasa
1978	Aerial Photograph. Scale: 1"=1000'	Flight Year: 1978	Nasa
1978	Aerial Photograph. Scale: 1"=1000'	Flight Year: 1978	Nasa
1978	Aerial Photograph. Scale: 1"=1000'	Flight Year: 1978	Nasa
1996	Aerial Photograph. Scale: 1"=1000'	Flight Year: 1996	USGS
1996	Aerial Photograph. Scale: 1"=1000'	Flight Year: 1996	USGS
1996	Aerial Photograph. Scale: 1"=1000'	Flight Year: 1996	USGS
1996	Aerial Photograph. Scale: 1"=1000'	Flight Year: 1996	USGS
2002	Aerial Photograph. Scale: 1"=1000'	Flight Year: 2002	USGS
2002	Aerial Photograph. Scale: 1"=1000'	Flight Year: 2002	USGS
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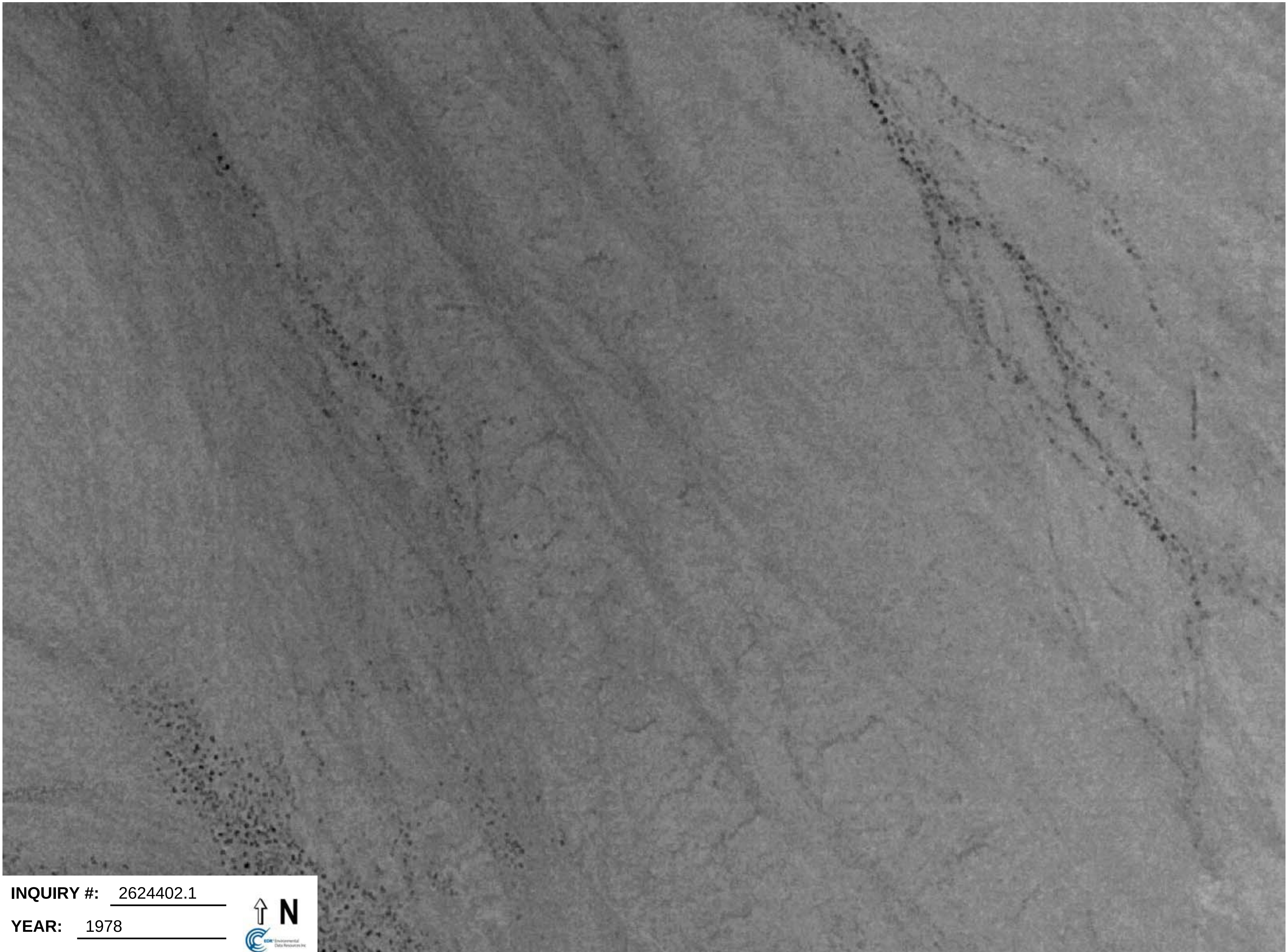

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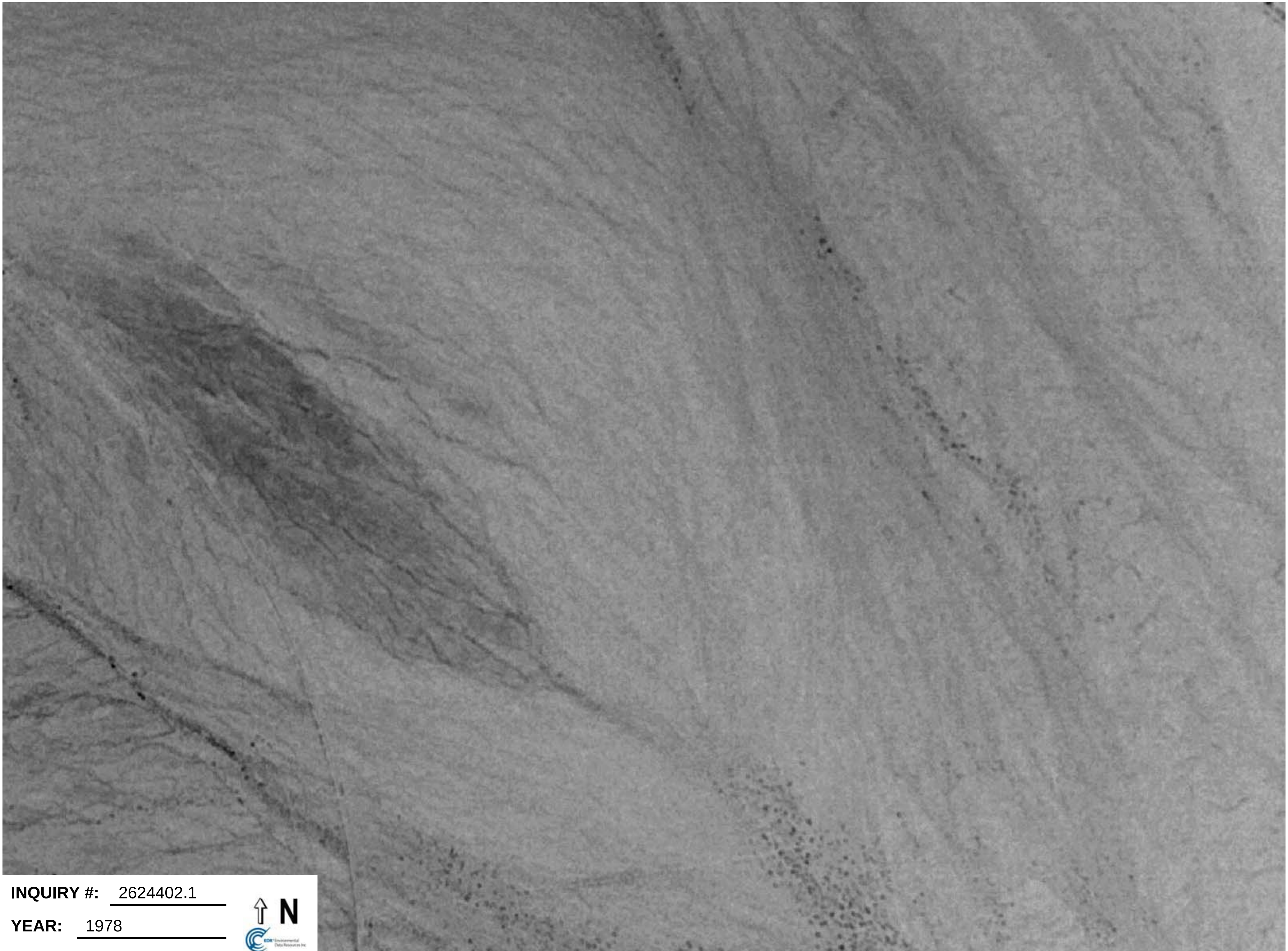

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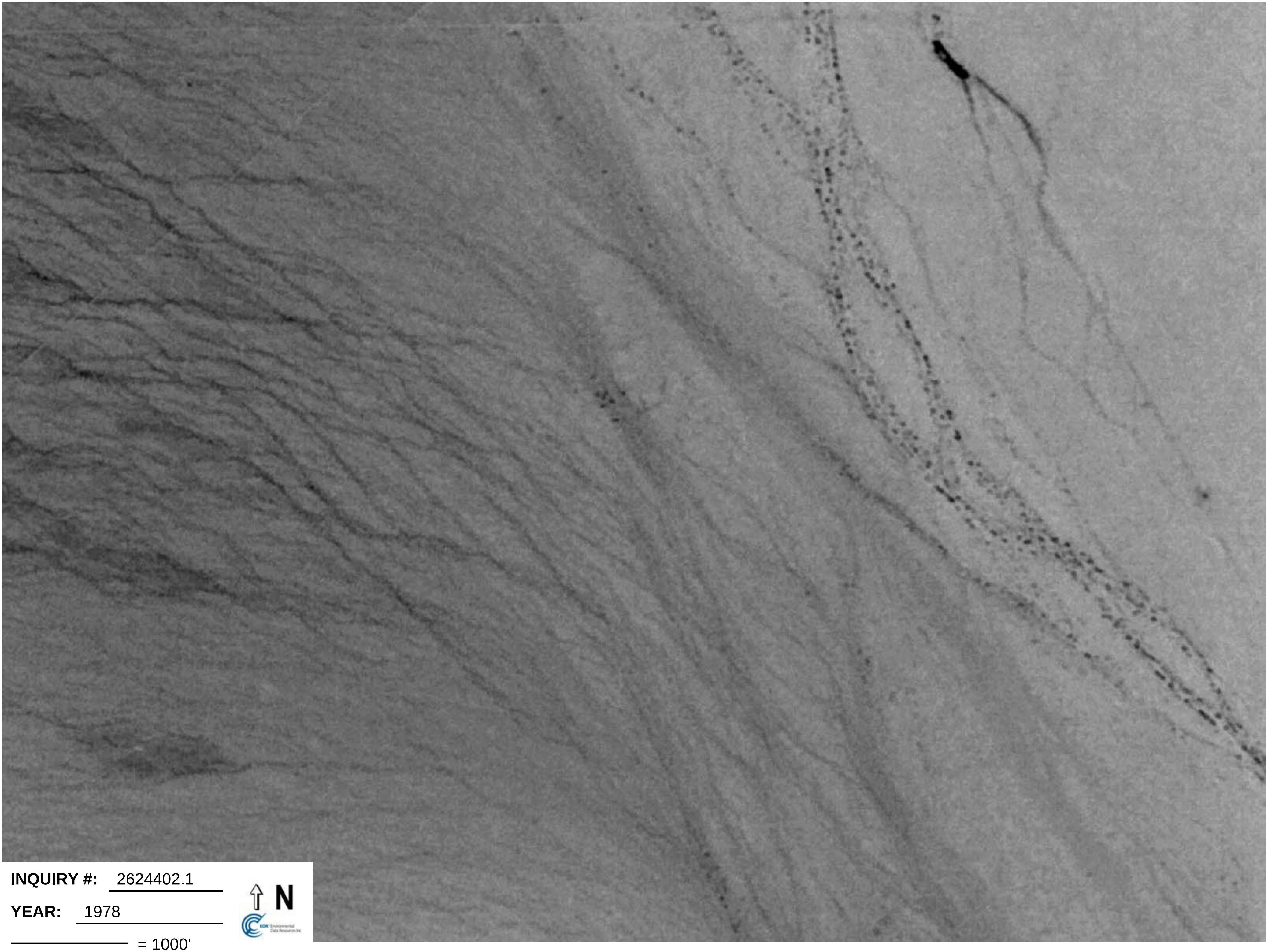

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INQUIRY #: 2624402.1
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EDR Environmental Data Resources Inc


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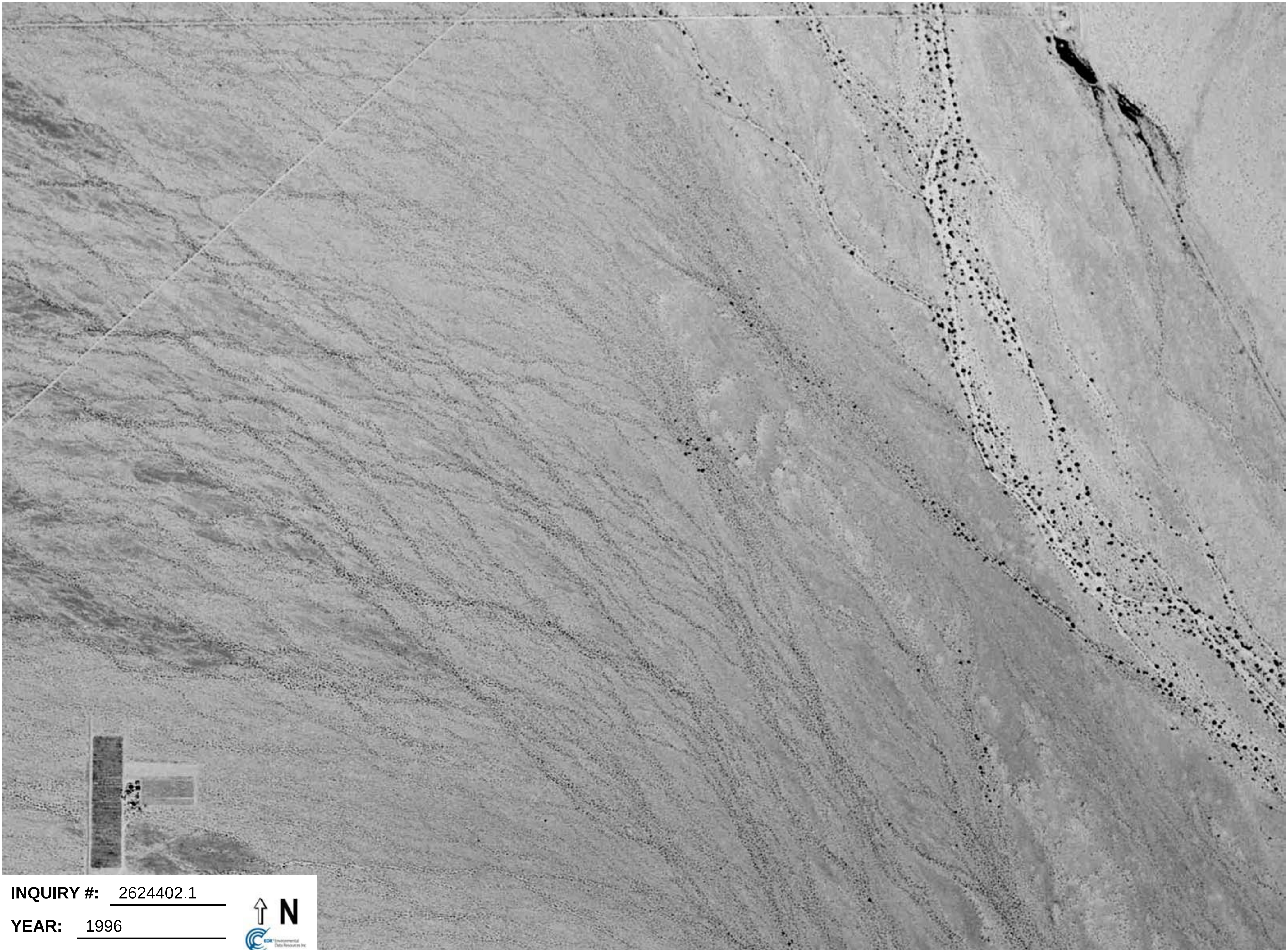
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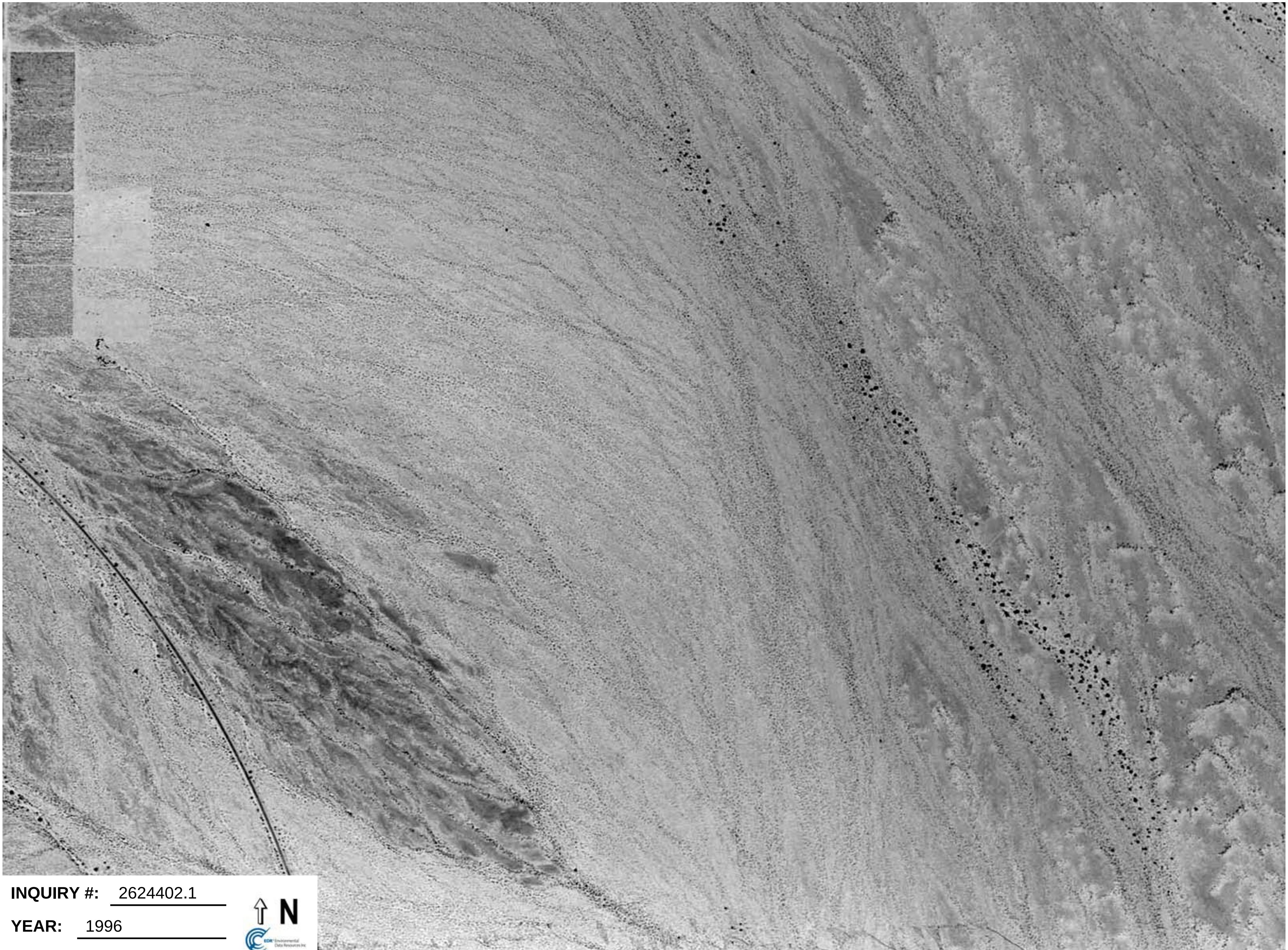

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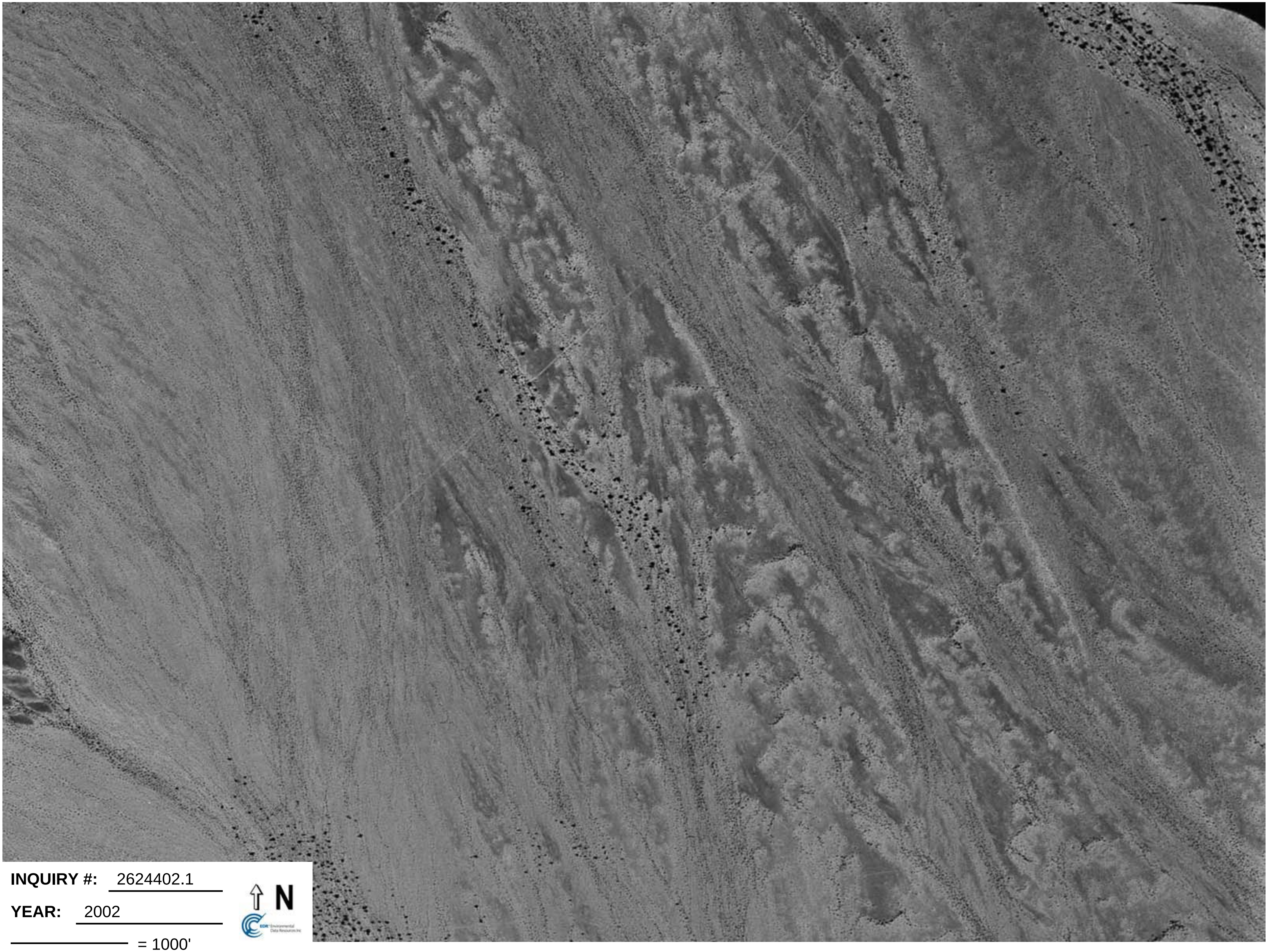

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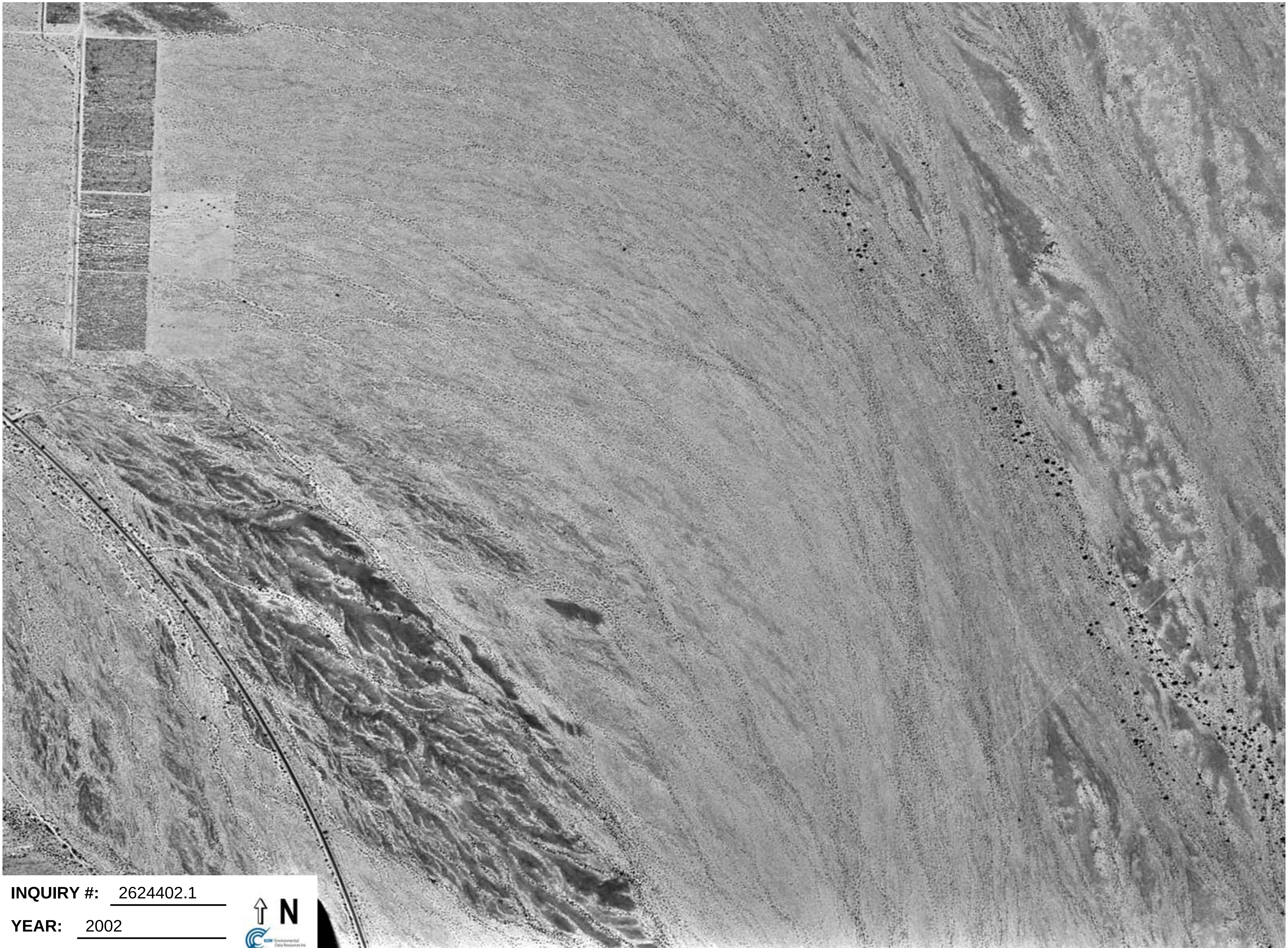

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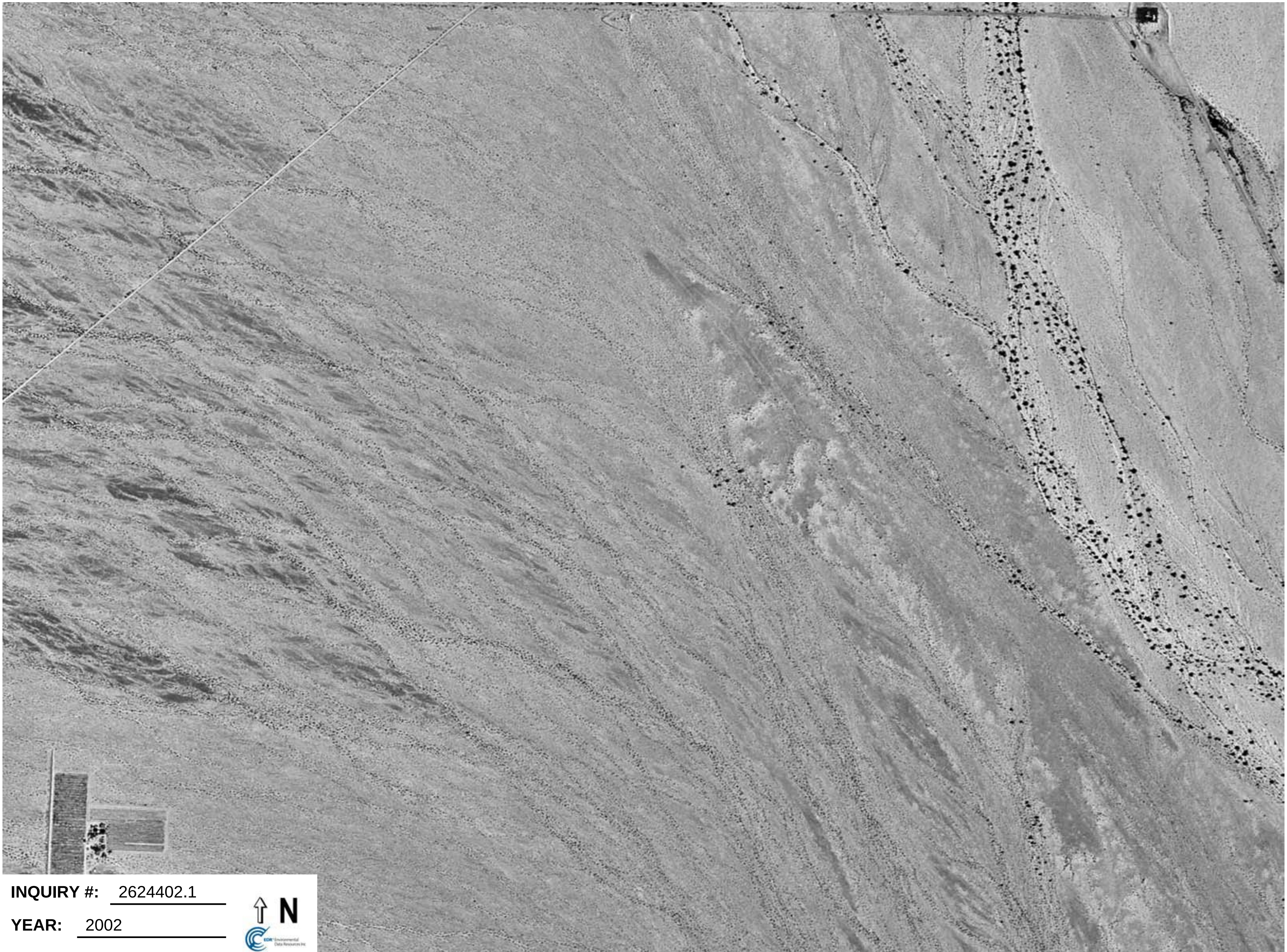

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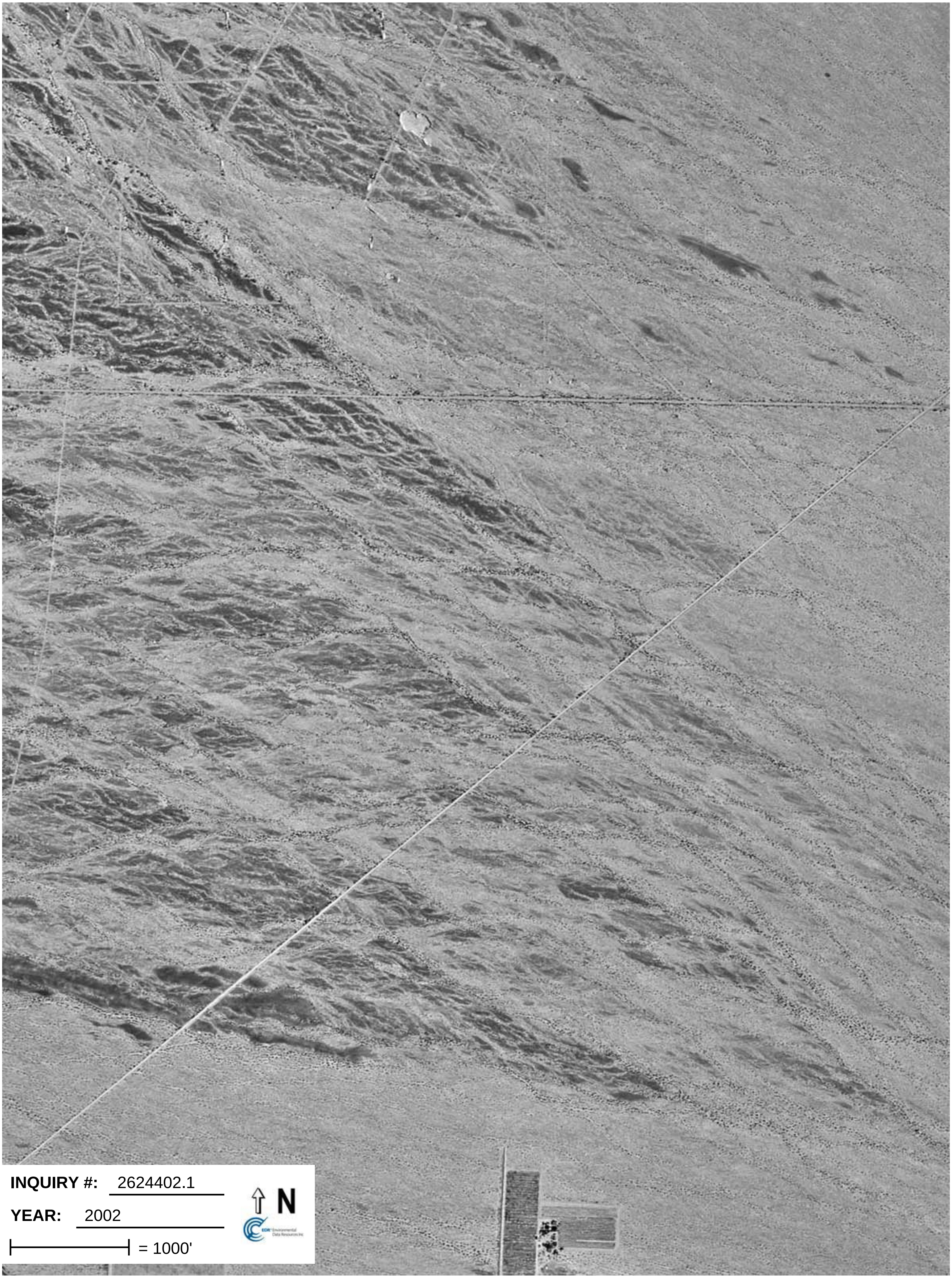
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YEAR: 2002
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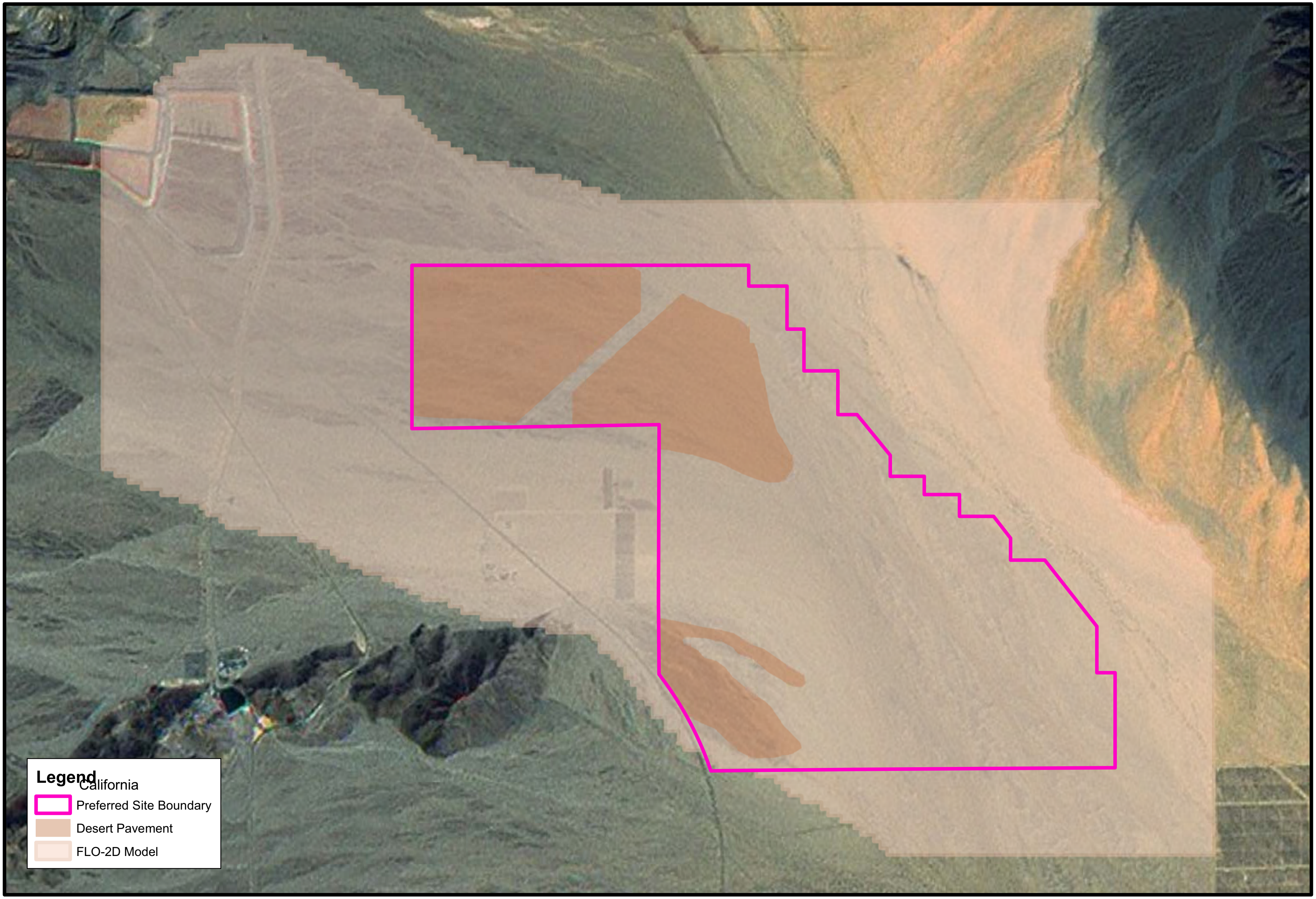
Appendix E: EUC Delineated Desert Pavement Areas

The following figure shows the locations where the infiltration capacity of desert pavement was applied to the hydraulic model.



AECOM

N

0 1 2 Miles

Appendix E
EUC Delineated Desert Pavement Areas

Storm Water Hydrology Report for Solar Farm Layout B

Desert Sunlight Solar Farm – Alternative B

Storm Water Hydrology Report: Hydrologic, Hydraulic, Sediment Transport and Scour Analyses

Project Site:
Desert Sunlight Solar Farm
Riverside County, California

Prepared for:
First Solar, Inc.
1111 Broadway, 4th Floor
Oakland, California 94607

Prepared by:
AECOM
South Portland, Maine/Camarillo, California
April 9, 2010

AECOM Project No. 60131167 (114785)

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APPENDICES

1 EXECUTIVE SUMMARY

AECOM has conducted hydrologic, hydraulic, sediment transport and scour analyses of storm water for Solar Farm Site – Alternative B of the First Solar, Inc. Desert Sunlight Solar Farm (DSSF or Project). The objectives of this Storm Water Hydrology Report (Report) are:

1. Establish design basis for the DSSF solar farm (Alternative B) improvements and infrastructure from a conservative (100-year) storm water event.
2. Verify that a low impact development measure (decompaction) with an additional measure will mitigate the hydrological impact to the upstream and downstream properties from the DSSF solar farm (Alternative B) improvements and infrastructure for a 10-year storm water event;

The significant results of this report are:

1. Results of the hydrologic analysis for the DSSF development indicated that implementing decompaction of the areas between the panels will reduce the post development hydraulic conditions to within +/-5% of the pre-development hydraulic conditions. An additional on-site mitigation measure such as basins with rip-rap protection, check dams or strip detention basins can be implemented to retain the remaining excess total off-site storm water volume increase. Please note that the accuracy of the model is approximately +/- 5% and so the differences (i.e. within 5%) calculated by the model are within this range.
2. Results of the hydrologic analysis for the post-development DSSF grading design without the addition of mitigation measures indicated that, in general, storm water off-site peak flow rates and volumes increased for the 10-year storm event. The storm water off-site peak flow rate and volume increased 4.7% and 5.9%, respectively for the 10-year storm event. The peak flow depth and velocity did not change on-site for the 10-year event.
3. Results of the hydrologic analysis for post-development design that only includes a decompaction mitigation measure indicated that the storm water off-site peak flow rate and volume increased 1.1% and 2.8% for the 10-year storm, respectively. Flow depth and velocity remain the same on-site, as compared to the existing conditions for the 10-year storm event. The additional storm water peak volume is reduced by decompaction of soils, which is the most significant measure to mitigate post-development conditions to within +/- 5% of the pre-development conditions.
4. The addition of mitigation measures such as basins with rip-rap protection, check dams, or strip detention basins to the DSSF development in addition to decompaction, will address excess post-development hydraulic impacts that are not addressed by decompaction. These additional measures are based on implementing storm water best management practices and have not been rigorously modeled, however they would be designed to retain excess total off-site storm water volume. The intent of an additional mitigation measure is to reduce overall flow depths, velocities and outflow volume by detaining run-on storm water volume. The additional measures would also be successful at reducing potential increases in sediment transport and would be designed to retain the excess total volume capacity which is on the order of 55 ac-ft for the 10-year storm event.
5. Results of the sediment particle size based transport model for post-development determined that the average degradation for the 10-year storm event within the project site does not change (the difference is 0.0%) for future conditions. The average degradation depth is 0.01 feet for the 10-year storm (i.e., general scour).
6. Results of the total scour analysis for post-development found that the average on-site scour depth would be 0.7 to 1.2 feet at the base of the PV supports for the 100-year storm, depending on the angle of flow to the supports. Placement of riprap will provide a less significant benefit to mitigate for additional runoff. However, riprap placed at the base of each support structure will help reduce the effects of local scour and lower storm water runoff velocities.

7. Results of the qualitative fluvial geomorphologic analysis indicates existing areas of relatively inactive sediments characterized by desert pavement and more active areas consisting of finer sand and gravel. The changes to the site resulting from Project development will create an area that has consistent compaction, soil type and grading compared to existing conditions. It is anticipated that these changes will create a geologic environment conducive to the formation of shallow channels up to two feet or less in depth (i.e. long-term scour). This long term scour can be mitigated by periodic monitoring to identify changes to the site grading and maintenance activities as/if needed to restore design conditions.
8. Along with the mitigation measures, a Monitoring and Response Plan will be prepared and submitted to the BLM. The Monitoring and Response Plan will indicate the procedures that will be followed to mitigate potential impacts to the site structures, storm water infrastructure or site grading that can occur from local scour, sediment transport and long term degradation (i.e. fluvial geomorphology) during the operation of the DSSF.

2 INTRODUCTION

AECOM has conducted a hydrologic, hydraulic, sediment transport and scour analyses of storm water conditions within and around Solar Farm Site – Alternative B of the Desert Sunlight Solar Farm (DSSF or Project) for First Solar, Inc. The DSSF is a future 550 MW solar photovoltaic (PV) electric generating facility. The Project is located in Riverside County on public lands under the jurisdiction of the Bureau of Land Management (BLM). This report provides a site description which includes an overview of the Project and its environment (climate, geology, land-use/soil-type, drainage areas), and a specific section on fluvial geomorphology. A quantitative hydrologic, hydraulic and sediment transport analysis was conducted using several computer models. In addition, scour evaluation was performed to assess scour potential around the PV support structures.

The objectives of the Report are:

1. Establish design basis for the DSSF solar farm (Alternative B) improvements and infrastructure from a conservative (100-year) storm water event.
2. Verify that a low impact development measure (decompaction) with an additional measure will mitigate the hydrological impact to the upstream and downstream properties from the DSSF solar farm (Alternative B) improvements and infrastructure for a 10-year storm water event;

The 100-year storm was used to focus on the storm water impacts on the development, and 10-year storm was used to evaluate impacts of the development on the storm water and sediment transport characteristics of the site. During a100-year storm event, the magnitude of the run-off is significant resulting in highest potential of structural impact; however, the difference in run-off between pre and post-development is higher during the 10-year storm, which is more probable to occur during the design life of the project. During the 10-year storm event, the percent difference is not overwhelmed by the shear amount of run-off volume associated with 100-year event, which quickly saturates the ground and effect of infiltration capacity diminishes. Therefore, using the 100-year event to evaluate storm water impacts on the development and the 10-year storm event to evaluate post-development stormwater and sediment transport characteristics represents a conservative approach to understanding the potential for stormwater impacts both on the Project and to the upstream and downstream properties.

The storm water analysis was based on the Riverside County Flood Control and Water Conservation District Hydrology Manual, which uses a 100-year storm event under antecedent moisture conditions (AMC) II criteria for the design basis criteria. A 10-year storm event was analyzed in addition to the 100-year storm event in order to evaluate the more probable event that will be experienced in the Project's lifespan.

The Report presents the results of a detailed hydrologic analysis and hydraulic/sediment-transport model of the DSSF for the existing (i.e., pre-development) conditions. It also includes the results of a watershed analysis that encompasses areas immediately upstream and downstream of the DSSF to determine and evaluate the Project's potential on-site and off-site peak flows during design storm events. The detailed analysis calculated off-site peak flow rate, off-site peak flow volume, maximum and average on-site peak flow depth, and on-site peak and average flow velocity. The off-site peak flow rate and volume are determined at the downstream boundary of the model, which is approximately1/4 mile south of the southern boundary

This report includes the results of the initial hydraulic analysis that modeled the pre-development conditions and compared them to the post-development conditions based on the Project's grading design submitted as part of the Project Description on March 19, 2010. The primary concepts relating to storm water characteristics that were incorporated into this DSSF grading design were contour grading. The intent of the contour grading concept is to smooth the existing surface into consistent graded slopes. Existing slopes on-site will be maintained such that the average cut/fill over the entire site is approximately 5-inches. The results of this comparison are discussed in Section 4.

The hydraulic analysis models the post-development conditions based on the Project's grading design that incorporates a decompaction mitigation measure. The intent of the de-compaction concept is to restore the soil infiltration capacity to the pre-development state. De-compaction will be applied to the areas between the rows of PV panels that were compacted during PV support structure and panel installation. The results of this comparison are discussed in Section 5.

Section 5.4 also includes discussions of other mitigation measures that are proposed to be in addition to the decompaction mitigation measure. These additional mitigation measures are recognized to have beneficial effects to the Project storm water characteristics, but are not as effective as the decompaction mitigation measure. Therefore these additional mitigation measures are discussed in qualitative terms.

Section 5.5 discusses the effect of the Project development on the storm water flows in Pinto Wash.

Sediment Transport characteristics comparing the pre-development conditions and post-development conditions based on the Project's grading design is presented in Section 6.

Fluvial geomorphology for the post-development conditions based on the Project's grading design is discussed in Section 7.

Local scour at the base of the PV solar panel supports for the post-development conditions based on the Project's grading design is discussed in Section 8.

3 PROJECT SITE DESCRIPTION

The DSSF is located on a vacant, largely undeveloped, and relatively flat tract of land in the Chuckwalla Valley area of the Sonoran Desert in eastern Riverside County, approximately four miles north of the rural community of Tamarisk Park and six miles north of the I-10 freeway and the rural community of Desert Center. The inactive Eagle Mountain Mine and the boundary of Joshua Tree National Park are located approximately 1.5 miles west and 1.4 miles east of the DSSF, respectively. The future DSSF location is shown on Figure 1.

Eagle Mountain Road, Kaiser Road, a paved road, and Eagle Mountain Railroad run from the Eagle Mountain Mine along the southwest portion of the DSSF before continuing south. Because the mine is no longer in operation, the various local roadways are lightly traveled.

Three existing transmission lines pass through the DSSF site. An existing 230-kV transmission line and a 33-kV distribution line, both owned by the Metropolitan Water District of Southern California (MWD), run along Power Line Road and traverse the DSSF.

3.1 Proposed Development

The DSSF, as proposed by First Solar, will be a solar photovoltaic (PV) energy generating facility producing 550 Megawatt AC (MWAC). The solar farm will occupy approximately 4,245 acres and includes the solar arrays, an on-site substation, access roads, a monitoring and maintenance facility, and other support facilities.

The First Solar PV modules, of which there will be a total of approximately 8.4 million on-site, are mounted on module framing assemblies made of steel, each holding 16 modules and measuring approximately eight (8) feet wide by 16 feet long. PV module assemblies are attached at an angle to vertical steel piles that are spaced eight (8) feet center-to-center and are driven into the ground to a depth of four (4) to seven (7) feet below grade. Each steel pile is a single W6x9 "I" beam. Once mounted, the front of each PV module assembly will be approximately 1.5 feet above grade, while the rear will be approximately five (5) to six (6) feet above grade. Each row of modules is spaced approximately seventeen (17) feet center-to-center from the adjacent row.

The PV modules are electrically connected by wiring harnesses running along the bottom of each assembly to combiner boxes that collect power from several rows of modules. The combiner boxes feed DC power from the modules to the Power Conversion Station (PCS) via underground cables. The inverters in the PCS convert the DC electric input into AC electric output and the isolation transformer steps the current up for on-site transmission of the AC power to the PV combining switchgear (PVCS). The PVCS collects the power for transmission to the Substation.

3.2 Climate

The National Oceanic and Atmospheric Administration (NOAA) Atlas 14, which was used to estimate precipitation frequency for the hydrologic model, defines southwestern California as a semi-arid region. The Riverside County Hydrology Manual describes the inland valley and desert areas as extremely hot and dry during the summer months and moderate during the winter. The mean seasonal precipitation is three inches in the eastern desert regions and 35 to 40 inches in the San Bernardino and San Jacinto Mountains. There are three types of storms within the region: (1) general winter storms, (2) general summer storms and (3) high intensity thunderstorms. General winter storms originate as tropical cyclones (warm Pacific air masses) that occur in the late fall or winter months. High rates of precipitation occur over the interior mountain ranges but precipitation decreases rapidly over the desert areas. General summer storms can result in heavy precipitation and have durations of several days. These typically occur between the months of July and September as a result of tropical air masses from either the Gulf of Mexico or the South Pacific Ocean. Thunderstorms that generate extremely high precipitation rates for short durations can occur at any time of year.

3.3 Geology

Regional and site surficial geology are discussed in the 2007 “Phase 1 Geologic Reconnaissance Report” prepared for the Project by Eberhart/United Consultants (EUC). The site is located within the southwestern portion of the Mojave Desert Geomorphic Province of southern California. The San Andreas Fault defines the southwestern boundary of the Geomorphic Province while the Garlock Fault forms the boundary to the north. The Mojave is a broad interior region of isolated mountain ranges separated by expanses of desert plains. It has an interior enclosed drainage and many playas. The proposed DSSF site is located in the Chuckwalla Valley, which is formed from multiple alluvial fans disseminating from the Eagle Mountains in the west and the Coxcomb Mountains in the east. The Pinto Wash bisects the valley and forms the eastern boundary of the solar farm site.

3.4 Land Use and Soil Type

Available data indicates that land use activities at the DSSF site have remained relatively consistent over the past 30 to 40 years. Several small agricultural plots have been established in the vicinity of the site with the use of irrigation. The site itself has remained as largely undeveloped desert with sparse vegetation.

Field reconnaissance by EUC in 2007 investigated the surficial sediments at the site. Two distinct sediment types were present, one associated with areas of desert pavement and the other with more active wash sediments. EUC collected samples with a hand auger at three locations within the proposed DSSF site. Table 1 below summarizes the sediment characteristics.

Table 1. Surficial Sediment Summary

Sample ID	Location	Depth (ft)	D_{50} (mm)	Description
A	Southwest	-	-	Well graded gravel (desert pavement) grading into well sorted sand with gravel
C	Northwest	0 to 0.5	9.5	Well graded gravel (desert pavement) grading into well sorted sand with gravel
C	Northwest	0.5 to 1.5	0.8	Well sorted sand with gravel
J	South	2.0 to 4.5	1.5	Well graded sand with gravel

3.5 Drainage Areas and Extent of the Modeling

The major drainage in the vicinity of the DSSF is the Pinto Wash. The Pinto Wash is located along the eastern boundary of the DSSF, continues southeast across undeveloped land, and drains into Palen Dry Lake to the east of the DSSF. Figure 2 shows a map of the model extents for both the hydrologic and hydraulic models. The basin delineation and model extents were developed utilizing automatic basin delineation tools available in the U.S. Environmental Protection Agency’s (EPA) BASINS software. Elevations from the United States Geological Survey’s (USGS) National Elevation Dataset were used for development of the model hydrology, which is discussed further in the following section.

4 HYDROLOGIC ANALYSIS

A two-dimensional (2D) model was constructed to simulate flow patterns and sediment transport within the DSSF. The hydrologic component of the 2D model was developed in HEC-HMS, a product of the Hydrologic Engineering Center (HEC) within the U.S. Army Corps of Engineers. The hydrologic analysis was performed using AMC II conditions utilizing guidelines outlined in the Riverside County Flood Control and Water Conservation District Hydrology Manual. The hydrologic analysis was repeated for the 10-year storm event incorporating various mitigation measures.

4.1 Hydrologic Analysis

The Riverside County Manual refers to the NOAA Atlas 2 for rainfall data. However, NOAA has superseded this source with Atlas 14 in the Project area. The website associated with NOAA Atlas 14 can provide rainfall intensity-duration-frequency (IDF) curves for any location based on latitude and longitude. The approximate coordinates of the DSSF site were entered into the website to develop rainfall totals for the 100- and 10-year storm events. A rainfall distribution was not specified by Riverside County; therefore, the balanced distribution recommended by the San Bernardino County Hydrology Manual (August 1986) was used for the analysis.

The Soil Conservation Service (SCS) curve number methodology was used to estimate flows to the hydraulic model. Curve numbers ranging from 79 in upstream areas to 63 in downstream areas were used for delineated basins. These curve numbers reflect AMC II, or normal moisture, conditions as specified by the Riverside County Manual. An initial abstraction of 0.15 was used. Lag times were calculated using the curve number method.

Hydrologic information was entered into HEC-HMS, which was then used to generate flows to the hydraulic model. Figure 3 presents the rainfall hyetograph at the Project site and Figure 4 shows the estimated total storm water peak flow running onto the entire project site over time during the 100-year and 10-year storm events. A summary of the hydrologic analysis is contained below in Table 2.

Table 2. Hydrologic Analysis Summary

Parameter	Value	Value
Design Storm Frequency	100-year	10-year
Peak Rainfall Depth	0.72 inches in 5 minutes	0.31 inches in 5 minutes
Total Rainfall Depth	3.58 inches	1.96 inches

5 HYDRAULIC ANALYSIS

Flow and sediment transport within the study area were simulated using FLO-2D. FLO-2D is a two-dimensional model designed to simulate unconfined overland flows. The extents of the FLO-2D model are shown in Figure 2 and include Solar Farm Site – Alternative Bas well as the Pinto Wash area immediately to the east. The northern and southern boundaries of the model were determined based on the path of water flow as per the USGS National Elevation Dataset. The upstream boundary extends approximately two miles upstream of the DSSF to establish flow patterns and sediment loads flow entering the site. The downstream boundary condition was set over half a mile downstream so that the downstream boundary condition would not affect flows on the Project site. FLO-2D model grid cells were set to dimensions of 200-feet by 200-feet.

Three configurations were analyzed: (1) existing conditions (2) proposed or future (post-development) conditions and (3) proposed or future conditions with soil decompaction. Future conditions were modeled without stormwater mitigation measures and with the inclusion of a storm water mitigation measure in the form of soil decompaction.

5.1 Inputs and Assumptions

Light Detection and Ranging (LIDAR) topographic survey data was collected within the DSSF. The LIDAR data was combined with USGS elevation data to populate the 2D model grid with elevations. These elevations represent the existing conditions of the site. For this analysis the same topographic data was used for both existing and proposed or future (post-development) conditions. Using the LIDAR data for both existing and future conditions will show the hydraulic changes at the project site as a result of grading and compaction by changing only the Manning's roughness and infiltration parameters. The grading plan would not greatly affect the model elevations that are averaged within the 200 foot by 200 foot grid elements created in FLO-2D.

The FLO-2D model uses the Green-Ampt method to simulate ground infiltration. The parameters for the Green-Ampt method were calibrated using information from the hydrologic HEC-HMS model. HEC-HMS uses the Curve Number infiltration method. The volume of flow that should runoff the site was estimated in HEC-HMS. The hydraulic conductivity in FLO-2D was adjusted so that the correct volume of flow was generated in the FLO-2D model. A curve number of 63 (i.e. barren land) was used for the majority of the existing conditions. The areas classified as "barren land" represent areas containing existing wash. The areas of desert pavement that occur within the project site were assumed to have similar infiltration capacity as the dirt roads introduced for the future conditions (i.e. curve number 72). Earth Systems Southwest (ESSW) provided an estimate that suggests approximately 20-30 percent of the total project area is covered in moderate to strong desert pavement. Delineation of the desert pavement areas were done by EUC (EUC, 2007). AECOM reviewed EUC's delineation against recent aerial images to confirm accuracy. This delineation is shown in Appendix E; the mapped desert pavement area is approximately 30 percent of the project site. The properties of desert pavement are discussed further in Section 7.1, Fluvial Geomorphologic Assessment Methodology. A curve number of 72 (i.e. dirt roads) was used for future conditions to account for compaction and loss of vegetation within the DSSF site. Outside the project site the existing conditions assignment of 63 representing barren land was retained.

A Manning's "n" value of 0.043 was used for existing conditions and was based on guidelines established by the USGS for developing Manning's roughness coefficients in floodplains (USGS Water-supply Paper 2339). For the post-development conditions, the Manning's "n" is reduced to 0.034, reflecting both the reduction in roughness due to smoothing the grade and removing existing vegetation and takes into account the increase in roughness due to the presence of the piles supporting the solar panels. See Appendix B for a detailed review of the Manning's value assignments.

5.2 Results: Future Conditions

The results presented in this section show the future hydraulic conditions without stormwater mitigation measures. The FLO-2D model was simulated for a 48-hour period for the 100- and 10-year design storm events. Plots of peak storm water depth and velocity for both future and existing conditions were produced with the FLO-2D model results. To be conservative in terms of peak velocities, sediment transport was not taken into account during these simulations. In reality, when sediment transport (scour) takes place flow depth will increase and the peak velocities will therefore decrease. Sediment transport models were developed separately, the results of the sediment transport analysis can be found in Section 6.

The 100-year future conditions model indicates that the storm water peak flow depth would be less than 2.3 feet in the center of the DSSF and towards the east due to the Pinto Wash. In general, the modeling results demonstrate that there would be very little change (less than one tenth (1/10) foot of difference) in flow depth as a result of Project-related changes to the site. The modeling results also demonstrate that there would be no increase in maximum storm water peak flow velocities as a result of the changes to the Project site.

A summary of the hydraulic analysis for the 100-year storm is contained in Table 3 below. In this table, “on-site location” essentially indicates the changes within the Project site and “off-site location” indicates the impacts to the areas immediately downstream of the DSSF site.

Table 3. Hydraulic Analysis Summary: 100-year

Parameter	Location	Existing Conditions	Future Conditions	Change
Peak Outflow	Off-site	23,952 cfs	24,263 cfs	311 cfs 1.3%
Total Outflow Volume	Off-site	6,645 acre-ft	6,813 acre-ft	168 acre-ft 2.5%
Maximum Peak Flow Depth	On-site	2.2 ft	2.3 ft	0.1 ft 4.5%
Average Peak Flow Depth	On-site	0.8 ft	0.8 ft	0.0 ft 0.0%
Peak Velocity	On-site	5.0 ft/s	5.0 ft/s	0.0 ft/s 0.0%
Average Velocity	On-site	1.9 ft/s	1.9 ft/s	0.0 ft/s 0.0%

The hydraulic model results of the 10-year storm can be found in Table 4, below. There was no change in peak flow depth from existing to proposed conditions and the average flow depth remained the same. Peak flow velocity and average velocities will not increase as a result of development for the 10-year storm.

Table 4. Hydraulic Analysis Summary: 10-year

Parameter	Location	Existing Conditions	Future Conditions	Change
Peak Outflow	Off-site	5,461 cfs	5,717 cfs	256 cfs 4.7%
Total Outflow Volume	Off-site	1,958 acre-ft	2,073 acre-ft	115 acre-ft 5.9%
Maximum Peak Flow Depth	On-site	1.5 ft	1.5 ft	0.0 ft 0.0%
Average Peak Flow Depth	On-site	0.4 ft	0.4 ft	0.0 ft 0.0%
Peak Velocity	On-site	3.5 ft/s	3.5 ft/s	0.0 ft 0.0%
Average Velocity	On-site	1.1 ft/s	1.1 ft/s	0.0 ft 0.0%

Table 3 and Table 4 do not reflect storm water mitigation measures that will be incorporated into the final design of the DSSF. See Section 5.3 below for the model results with incorporated LID design mechanisms.

5.3 Results: Future Conditions with Decompaction

The results presented in this section show the future hydraulic conditions with decompaction as stormwater mitigation. The FLO-2D model was simulated for a 48-hour period for the 100- and 10-year design storm events. Infiltration rates were adjusted to represent decompaction of the soil between the rows of the arrays. Plots of peak storm water depth and velocity for both future and existing conditions were produced with the FLO-2D model results. To be conservative in terms of peak velocities, sediment transport was not taken into account during these simulations. Sediment transport models were developed separately, the results of the sediment transport analysis can be found in Section 6.

The goal of the design is to minimize the change of hydraulics and sediment transport. The grading design incorporating the soil decompaction storm water mitigation measure was modeled to determine the impact caused by development of the DSSF site. Soil decompaction will be implemented between the rows of tables within each of the arrays. The decompaction operation will restore the infiltration to the pre-development original state. The intent of the decompaction mitigation measure is to increase the post-development soil infiltration that results in a lower total storm water outflow volume.

For the project areas located on existing desert pavement, the decompaction measure is not anticipated to restore the pre-development conditions. Project areas that are currently covered with desert pavement already have a low infiltration capacity. Although the decompaction measure is intended to increase post-development soil infiltration, the decompaction measure is not anticipated to significantly change the infiltration capacity as compared to pre-development conditions for desert pavement areas.

The values presented in Table 5 are the results from simulating decompaction of 37.3% of the total project site. This percentage was calculated based on the current array configuration and site layout that allows for approximately 9.4 feet of the area between rows to be decompacted with an allowance to minimize damage to the panels. Figure 9 shows the maximum peak flow depths, Figure 10 shows the change in maximum peak flow depth, Figure 11 shows the maximum peak velocity and Figure 12 shows the change in peak velocity. The change in total outflow volume is 81 acre-feet or a 1.2% increase from existing conditions when decompaction was considered.

Table 5. Hydraulic Analysis Summary: 100-year with Decompaction

Parameter	Location	Existing Conditions	Future Conditions with Decompaction Measure	Change
Peak Outflow	Off-site	23,952 cfs	24,068 cfs	116 cfs 0.5%
Total Outflow Volume	Off-site	6,645 acre-ft	6,726 acre-ft	81 acre-ft 1.2%
Maximum Peak Flow Depth	On-site	2.2 ft	2.2 ft	0 ft (0%)
Average Peak Flow Depth	On-site	0.8 ft	0.8 ft	0 ft (0%)
Peak Velocity	On-site	5.0 ft/s	5.0 ft/s	0 ft/s (0%)
Average Velocity	On-site	1.9 ft/s	1.9 ft/s	0 ft/s (0%)

The 10-year decompaction simulation resulted in a change in total outflow volume of 55 acre-feet or a 2.8% increase from existing conditions. Figure 13 shows the maximum peak flow depths, Figure 14 shows the change in maximum peak flow depth, Figure 15 shows the maximum peak velocity and Figure 16 shows the change in peak velocity.

Table 6. Hydraulic Analysis Summary: 10-year with Decompaction

Parameter	Location	Existing Conditions	Future Conditions with Decompaction Measure	Change
Peak Outflow	Off-site	5,461 cfs	5,519 cfs	58 cfs 1.1%
Total Outflow Volume	Off-site	1,958 acre-ft	2,013 acre-ft	55 acre-ft 2.8%
Maximum Peak Flow Depth	On-site	1.5 ft	1.5 ft	0 ft (0%)
Average Peak Flow Depth	On-site	0.4 ft	0.4 ft	0 ft (0%)
Peak Velocity	On-site	3.5 ft/s	3.5 ft/s	0 ft/s (0%)
Average Velocity	On-site	1.1 ft/s	1.1 ft/s	0 ft/s (0%)

The results presented in Table 5 and Table 6 do not include sediment transport functions.

The decompaction measure will mitigate the impact from pre to post development conditions to less than 5% change at the boundary of the model.

5.3.1 Discussion of Additional Mitigation Measures

Decompaction of soils is the most significant measure to mitigate post-development impact, by reducing added runoff. Decompacting the soil provides additional infiltration capacity which reduces runoff volume, peak flow rate, flow velocities and sediment transport. Placement of riprap can also be considered as an additional mitigation measure. Riprap increases surface roughness slowing down the velocities, decreasing sediment transport, and increasing flow depth. Riprap would be used in conjunction with decompaction, as riprap will not mitigate flow or volume.

An additional mitigation measure such as retention basins can be implemented to address specific post-development hydraulic characteristics that remain after implementation of the decompaction measure. These retention basins could be located along the upstream western boundary of the project site to intercept run on storm water flows. The intent of this measure is to reduce overall flow depths, velocities and outflow volume by retaining run-on storm water volume. They will also reduce sediment transport within the project site. Due to the size of the grid elements in FLO-2D (200 foot by 200 foot) an accurate representation of the basins cannot be distinguished in the model. However, it can be assumed that the basins can be designed to retain the excess total storm water volume. Once the basins are designed, their retention capacity volume can be subtracted from the total outflow volume of any of the simulations. Retentions basins would be designed to retain the excess total volume capacity which for the current modeling results is on the order of 55 ac-ft for the 10-year storm event.

An additional mitigation measure such as check dams can be implemented to address specific post-development hydraulic characteristics that remain after implementation of the decompaction measure. These check dams could be located near the downstream southern boundary of the project site to intercept run off storm water flows. The intent of this measure is to reduce outflow volume by retaining run-off storm water volume. Check dams would have an effect on the storm water upstream of each dam because the storm water would back up behind each dam. Check dams would also reduce flow velocities and sediment transport leaving the project site. Check dams would change the Manning's roughness ("n") values used in the model at their immediate vicinity. It can be assumed that the check dams can be designed to retain the excess total storm water volume. Once the check dams are designed, their retention capacity volume can be subtracted from the total outflow volume of any of the simulations. Check dams would be designed to retain the excess total volume capacity which for the current modeling results is on the order of 55 ac-ft for the 10-year storm event.

An additional mitigation measure such as strip detention basins can be implemented to address specific post-development hydraulic characteristics that remain after implementation of the decompaction measure. The strip detention basins would be approximately 6-inches deep and 70 feet wide. The strip detention basins would be designed to follow the contours, so the lengths would be dependent on the locations of the basins on the site. These detention basins could be located near the downstream southern boundary of the project site to intercept run off storm water flows. The intent of this measure is to reduce outflow volume by detaining run-off storm water volume, similar to the check dam measures. Strip detention basins would not have an effect on the storm water upstream of each basin but would reduce flow velocities and sediment transport leaving the project site. Strip basins would not appreciably change the Manning's roughness ("n") values used in the model for the project. The strip detention basins would not be as effective a measure as the check dams. Check dams can be designed to hold more volume than the strip detention basins when placed on flatter slopes and also check dams will act as a bigger obstacle than strip detention basins attenuating storm water flow. It can be assumed that the strip detention basins can be designed to retain the excess total storm water volume and would have a retention volume capacity equivalent to that for the check dams. Strip detention basins would be designed to retain the excess total volume capacity which for the current modeling results is on the order of 55 ac-ft for the 10-year storm event. Once the strip detention basins are designed, their detention capacity volume can be subtracted from the total outflow volume of any of the simulations.

5.3.2 Discussion of Effect on the Pinto Wash

As shown on the pre-development and post-development figures, the development will not significantly affect the storm water flow in the Pinto Wash. For the most part, the storm water flow in the Pinto Wash will encroach onto the DSSF for 10-year and 100-year storm events. The figures show that the flow on the DSSF does not enter the Pinto Wash along the DSSF boundary (or within the boundaries of the model), rather the storm water outflow from the site will enter the Pinto Wash in an area several miles downstream of the DSSF. The volume of storm water in the Pinto Wash is on the order of 4,072 ac-ft for the 100-year storm event and 1,545 ac-ft for the 10-year storm event. The DSSF does not increase Pinto

Wash flows at the downstream end of the project; however, an additional 81 ac-ft for the 100-year event from the DSSF would eventually make its way into Pinto Wash at which point the increase is expected to be less than 1%. Velocities and depths within the pinto wash will not change as a result of development. The DSSF development would not have a significant impact to a storm water flow in the Pinto Wash.

6 SEDIMENT TRANSPORT ANALYSIS

This section describes sediment transport for the project as predicted by FLO-2D. The sediment transport analysis is conservative because degradation depths presented do not reflect sediment deposition which may occur within the same model cell. The model does not account for local scour at the supports for the solar panels. Local scour is evaluated later in this report; see Section 8 LOCAL SCOUR ANALYSIS.

6.1 Methodology

The existing and proposed model configurations discussed in the Hydraulics Section were modified to account for sediment transport. FLO-2D has the capability of simulating sediment transport and offers several different methodologies. The Zeller and Fullerton methodology was selected for sediment transport analysis of the DSSF since this methodology is appropriate for alluvial floodplain conditions (FLO-2D User's Manual, 2007). Sediment profile information was obtained from the geotechnical study (EUC, 2007).

6.2 Results

The existing and future conditions with decompaction were modeled under AMC II conditions to determine the loss in depth of the sediment (degradation or scour) during the 100- and 10-year storm events. Maps presenting the results of the existing conditions 100-year and 10-year peak degradation are shown in Figure 18 and 20, respectively. Maps presenting the results of the 100-year and 10-year peak degradation are shown in Figure 18 and Figure 21 respectively. Graphs showing change in sediment transport depth can be found in Figure 19 and Figure 22. Table 7 presents average degradation depths for the 100-year storm event within the DSSF for the simulations. The modeling results determined that the average degradation for the 100-year storm event within the project site does not change (the difference is 0.0%) for the future conditions with decompaction.

Table 7. Sediment Transport Summary: 100-year storm

Simulation	Average Degradation Depth	Change
Existing Conditions	0.03 ft	NA
Future Conditions with Decompaction	0.03 ft	0.00 ft (0.0%)

The 10-year simulation results are presented below in Table 8. The modeling results determined that the average degradation for the 10-year storm event within the project site does not change (the difference is 0.0%) for future conditions.

Table 8. Sediment Transport Summary: 10-year storm

Parameter	Average Degradation Depth	Change
Existing Conditions	0.01 ft	NA
Future Conditions with Decompaction	0.01 ft	0.00 ft (0.0%)

Sediment transport, based on the sediment particle size, showed that the proposed installation did not have any impact on degradation; the average degradation depth is 0.03 feet for the 100-year storm and 0.01 feet for the 10-year storm over most of the DSSF for both pre- and post-development conditions. The results show that the average degradation within the project site remains the same for existing conditions and all development options.

Although the modeling results indicate that the average degradation depth is not significant for both pre- and post-development conditions, sediment transport may occur as a result of either a large storm event or a series of smaller storm events. This issue can be mitigated by periodic monitoring and maintenance of the site. For example, monitoring conducted after storm events would indicate sediment depth at that time and maintenance activities would be conducted as/if needed to add/remove material to restore design conditions. A Monitoring and Response Plan will be incorporated into the final design of the DSSF to ensure that the storm water infrastructure is in good working order on an ongoing basis during Project operation.

7 FLUVIAL GEOMORPHOLOGIC ASSESSMENT

7.1 Methodology

AECOM reviewed existing data including geologic literature, site reports, aerial mapping and topographical survey to qualitatively determine the fluvial geomorphology of the DSSF. Aerial photographs from the years 1978, 1996 and 2002 were analyzed to determine changes in land use and stream channel configurations.

As noted earlier, the DSSF is located in the Chuckwalla Valley, which is bounded by a series of alluvial fans that slope gently to moderately toward the southwest and southeast. The Pinto Wash runs through the center of the valley. The DSSF facilities are to be located to the west of the Pinto Wash. Vegetation at the site generally consists of sage and other scrub-type brush that is typical for the arid regions of southern California (EUC, 2007).

The geomorphology of alluvial fans is described by John Field and Philip Pearthree in their article "Geomorphologic Flood-Hazard Assessment of Alluvial Fans and Piedmonts" published in the Journal of Geoscience Education, Vol. 45, 1997:

> "Alluvial fans are generally cone-shaped depositional landforms with distributary drainage patterns that emanate from a discrete source and increase in width downslope. Older, inactive, alluvial fans commonly are isolated from active depositional processes and dendritic drainage patterns are developed on them."
>
> "Surfaces that are subject to flooding are undissected, display well preserved bar-and-swale topography, and lack desert pavement and varnish. In contrast, surfaces that have not been flooded for hundreds of thousands of years are moderately to deeply dissected, have well developed desert pavements and abundant shattered cobbles on the surface; their soils include substantial accumulations of clay and calcium carbonate (caliche)."
>
> "Several criteria can be used to distinguish between a permanent and temporary trench. Fanhead trenches dissecting inactive surfaces with well developed soils, desert pavement, and rock varnish are permanent features, since it is the incision of the trench itself that is largely responsible for the isolation of the adjacent old surfaces. A trench dissecting a young surface, on the other hand, is potentially only a transient feature. The depth of incision alone should not be used to determine whether a trench is permanent. Trenches as deep as 8 m can be filled and/or cut during a single debris flow event. …Regardless of the absolute depth of the incision, a fanhead trench is not a permanent feature if floodwaters can overtop or backfill the channel under the prevailing hydrologic conditions."

Review of recent aerial imagery and site photographs indicates that there are two significant geologic environments occurring at the DSSF. The first geologic environment is characterized as older alluvial sediments with developed desert pavement. This environment occurs in the northwest portion of the site in the vicinity of Power Line Road. It also occurs in the southwest corner of the site adjacent to Kaiser Road (Co Route R2). Based on LIDAR topographic survey data, alluvial stream channel depths near Power Line Road approach four feet at the northwest end of the project while the channels near Kaiser Road are generally two (feet or less.

The second significant geologic setting at the DSSF site consists of an area of active younger sediments with no evidence of desert pavement. Topography in these areas tends to be very consistent with channels depths generally less than one foot deep.

The EUC "Phase 1 Geologic Reconnaissance Report" corroborates the two significant conditions encountered at the site. EUC describes the established alluvial sediments as follows:

> "Older alluvial fan deposits consisting of Pleistocene non-marine sediments extend outward into the valley from both the Eagle Mountains on the west and the Coxcomb Mountains on the east. Desert pavement type deposits (manganese and iron oxidized coatings on cobbles and sand) blanket the top three (3) to six (6) inches of the older alluvial fan material."

EUC describes the area near Power Line Road and Kaiser Road as the "Northwest fan – includes sediments derived from the Eagle Mountain Quartz Monzonite, Pleistocene volcanic rocks, and Pre-Cretaceous metamorphosed sediments." In contrast, they describe the younger active sediments as "of Holocene age. These soils consist of fine to coarse sand, interbedded with clay, silt and gravel."

Lateral migration of stream channels is typically evaluated based on the analysis of historical aerial photographs. AECOM reviewed aerial photographs from the years 1978, 1996 and 2002 at the proposed site. Based on the data available, stream channels at the site have been relatively stable over the period evaluated. It is more difficult to determine the stability of smaller channels located in the more active portions of the site due to their scale. Based on knowledge of similar environments, it would be expected that alluvial stream channels in the older alluvial regions remain relatively stable. It is anticipated that the shallow channels that exist within the younger sediment would exhibit frequent channel avulsion and lateral migration during flood flows.

7.2 Results

Changes to the vertical profile of the stream channels are difficult to quantify without detailed survey data of Project site topography over time. However, existing conditions at the site indicate channel depths of two to four feet in the older alluvial sediments and less than two feet in the younger sediments.

The grading design of the DSSF includes grading of the entire site with varying levels of compaction depending on proposed land use (primary road, secondary road, etc.). Existing slopes on the site vary from zero to two percent in the active alluvial areas to two to four percent in the regions of less active older alluvial sediments. Planned slopes will be zero to two percent across the entire site.

The proposed changes to the site will have an impact on future geomorphic conditions. Instead of relatively inactive areas characterized by desert pavement in combination with more active areas, the geologic conditions at the site will change to a more consistent geological condition. Changes to existing site grades will also have an impact on flood flows. It is anticipated that these changes will create a geologic environment conducive to rapidly migrating shallow channels, approximately two feet deep or less. Channel formation from fluvial geomorphology occurs as a result of multiple storm events over time. This long term scour or channel formation can be mitigated by periodic monitoring to identify changes to the site grading, followed by maintenance measures to address these changes as/if needed. Development of a Monitoring and Response plan would address monitoring of the drainage control devices after storm events and development of appropriate maintenance responses so that the drainage control devices are operational for subsequent storm events. Flatter slopes may also contribute to areas of sediment deposition during storm events.

If further evaluation of existing and post-development conditions at the site is needed, a detailed quantitative fluvial geomorphologic assessment will be conducted. The quantitative evaluation would include a detailed analysis of stream migration based on historical aerial images, additional historical information including interviews with local inhabitants, and site reconnaissance to determine channel characteristic, extent of desert pavement and soil properties.

Photo 1. Stream channel in older alluvial sediments (desert pavement)

Photo 2. View of desert pavement material

8 LOCAL SCOUR ANALYSIS

The total predicted scour depth is the sum of the following components: general scour, long term scour and local scour. General scour is discussed in the Sediment Transport Analysis Section 6 of this report. Long term scour depth is estimated in the previous Fluvial Geomorphologic Assessment Section 7. It is assumed that the long term scour can be mitigated by periodic monitoring to identify changes to the site grading and followed by maintenance measures to address these changes as/if needed. Therefore, the total scour depth presented in this section is assumed to be the local scour and general scour that the site structures could experience. The local scour is discussed herein for the future conditions 100-year storm event. Local scour is measured at an instantaneous point in time as a result of turbulent flow at the pylons. Sediment is suspended at the base of these structures within the turbulent flow. As the sediment moves away from the turbulent zone the flow can no longer support the sediment load and it is typically deposited a short distance downstream. Local scour occurs at the base of a structure as a result of the change in direction and velocity of storm water as the water flows around the structure. The effect of the local scour is limited to the area immediately adjacent to the base of the PV solar panel support structures.

8.1 Methodology

For the purpose of this study, local scour was analyzed at the base of the PV solar panel support structures. Scour depths were calculated using a local pier scour equation from the Federal Highway Administration's Hydraulic Engineering Circular No. 18 (HEC-18), "Evaluating Scour at Bridges" (4^{th} Edition).

Scour depths were calculated for each element in the 2D model within the DSSF. Velocity and depth outputs from the model were used to determine scour at each element. The dimensions of a model element are 200-feet by 200-feet and velocities and depths predicted by the model are averaged across the element area. Therefore, the velocities may not be conservative because high concentrations at portions of the element are lost and larger scour depths than predicted may occur.

The local scour equation and the various parameters and assumptions are as follows:

$$\frac{y_s}{a} = 2.0 K_1 K_2 K_3 K_4 \left(\frac{y}{a}\right)^{0.35} Fr^{0.43}$$ **(Equation 1)**

Where:

y_s = Local scour depth (ft);
K_1 = Correction factor for pier nose shape;
K_2 = Correction factor for angle of attack of flow;
K_3 = Correction factor for bed condition;
K_4 = Correction factor for armoring;
a = Pier width (ft);
y = Flow depth (ft);
Fr = Froude number:

$$Fr = \frac{V}{\sqrt{gy}}$$ **(Equation 2)**

Where:

V = Average velocity (ft/s);
g = Acceleration due to gravity (ft/s^2).

8.2 Approach

Two (2) different scour depth analyses were performed to encompass the best and worst case scour depths by varying the pile geometry. The only parameters of the scour equation that change in each case are the pier width (a) and the correction factor for angle of attack (K_2). All other values (velocity, depth, etc.) remain the same for a given element within the modeled domain. A plane bed was assumed for the bed condition, resulting in a K_3 factor of 1.1. The grain size analyses collected during the EUC Phase 1 Geologic Reconnaissance Report all contained a median particle diameter of less than two (2) millimeters, resulting in a K_4 factor of 1.0.

8.3 Inputs and Assumptions

The proposed pile configuration consists of steel wide flange I-beams (W6X9). The shape correction factor was assumed to be square for both cases, resulting in a K_1 factor of 1.1. The worst case analysis assumed the pier width was the largest flange dimension (5.9 inches) and the angle of attack was assumed to be 90 degrees. A 90 degree angle of attack produces the largest K_2 value (1.3). The equation for determining K_2 is shown below (HEC-18):

$$K_2 = \left(Cos\theta + \frac{L}{a} Sin\theta \right)^{0.65} \qquad \textbf{(Equation 3)}$$

Where:

L = Length of pile (ft);
Θ = Angle of attack of flow (degrees).

The worst case angle of attack assumptions mentioned above produce the most conservative scour depth results. The best case scour analysis assumed the pier width was the smallest flange dimension (3.94 inches) and the angle of attack was assumed to be zero degrees. A zero degree angle of attack produces the smallest K_2 value (1.0). The best case angle of attack assumptions produce less conservative scour depths and are not presented herein. A visual representation of the 100-year worst case scenario is shown on Figure 23.

8.4 Results

The maximum local scour depth (i.e. when the flow is aligned with the widest part of the support structure) for the DSSF using the worst case assumptions described above for the 100-year storm was 2.1 feet. The maximum total scour within the project site was 2.6 feet. This was the combination of local scour and general scour within the same model cell. This scour depth occurred for both the future conditions and future conditions including the decompaction mitigation measure. The areas of maximum scour potential are along the northwest portion of the site. The average scour depth was found to be 1.2 feet. Table 9 shows the frequency of occurrence for the more-erosive scour depths within the project site. Figure 23 shows the distribution of maximum local scour depths using worst case assumptions within the Project area for the future conditions 100-year storm.

Formation of local areas of scour can occur as a result of a large storm event or a series of smaller storm events. Local scour can be mitigated by periodic monitoring and maintenance of the site. A Monitoring and Response Plan will be utilized during operations of the DSSF to ensure that PV supports remain in stable operational condition and are not compromised by local scour impacts.

Table 9. Local Scour Summary: 100-year Worst Case Frequency of Occurrence within the Project Site for Decompaction

Depth of Scour	Local Scour	Total Scour
0.0 to 0.5 feet	0.1%	0.1%
0.5 to 1.0 foot	24.9%	24.4%
1.0 to 1.5 feet	68.2%	63.8%
1.5 to 2.0 feet	6.7%	11.2%
2.0 to 2.5 feet	0.1%	0.4%
2.5 to 3.0 feet	0.0%	0.0%
Average Scour Depth	1.2 ft	1.2 ft
Maximum Scour Depth	2.1 ft	2.6 ft

The less erosive-case (i.e. when flow direction is aligned with the narrow side of the support structure) maximum local scour depth was 1.2 feet and total scour was 1.9 feet. Frequency of occurrence can be found in Table 10 for the less-erosive case.

Table 10. Local Scour Summary: 100-year Best Case Frequency of Occurrence within the Project Site for Decompaction

Depth of Scour	Local Scour	Total Scour
0.0 to 0.5 feet	14.9%	14.3%
0.5 to 1.0 foot	84.7%	81.2%
1.0 to 1.5 feet	0.4%	4.3%
1.5 to 2.0 feet	0.0%	0.2%
Average Scour Depth	0.7 ft	0.7 ft
Maximum Scour Depth	1.2 ft	1.9 ft

9 CONCLUSIONS

The results of the storm water modeling are:

1 Results of the hydrologic analysis for the DSSF development indicated that implementing decompaction of the areas between the panels will reduce the post development hydraulic conditions to within +/-5% of the pre-development hydraulic conditions. An additional on-site mitigation measure such as basins with rip-rap protection, check dams or strip detention basins can be implemented to retain the remaining excess total off-site storm water volume increase. Please note that the accuracy of the model is approximately +/- 5% and so the differences (i.e. within 5%) calculated by the model are within this range.

2 Results of the hydrologic analysis for the post-development DSSF grading design without the addition of a mitigation measure indicated that, in general, storm water off-site peak flow rates and volumes increased for both the 100-year and 10-year storm events. The storm water off-site peak flow rate and volume increased 1.3% and 2.5% respectively for the 100-year storm event and 4.7% and 5.9%, respectively for the 10-year storm event. The maximum on-site peak flow depth for the 100-year event increased 4.5% and there was no change for the 10-year event. The on-site peak flow depth and velocity did not change for 100-year and 10-year events.

3 Results of the hydrologic analysis for post-development design that only includes a decompaction mitigation measure indicated that the storm water off-site peak flow rate and volume increased 0.5% and 1.2% respectively for the 100-year storm event and 1.1% and 2.8% respectively for the 10-year storm event. Flow depths and velocities remain the same on-site, as compared to the existing conditions for both the 100-year and 10-year storm events. The additional storm water peak volume is reduced by decompaction of soils, which is the most significant measure to mitigate post-development conditions to within +/- 5% of the pre-development conditions

4 The addition of mitigation measures such as basins with rip-rap protection, check dams, or strip detention basins to the DSSF development in addition to decompaction, will address excess post-development hydraulic impacts that are not addressed by decompaction. These additional measures are based on implementing storm water best management practices and have not been rigorously modeled, however they would be designed to retain excess total off-site storm water volume. The intent of an additional mitigation measure is to reduce overall flow depths, velocities and outflow volume by detaining run-on storm water volume. The additional measures would also be successful at reducing potential increases in sediment transport and would be designed to retain the excess total volume capacity which is on the order of 55 ac-ft for the 10-year storm event.

5 Results of the sediment transport analysis for post-development determined that the average degradation for the 100-year and the 10-year storm event within the project site does not change (the difference is 0.0%) for future conditions. The average degradation depth for the 100-year storm would be 0.03 feet, and 0.01 feet for the 10-year storm (i.e., general scour);

6 Results of the total scour analysis for post-development found that the average on-site scour depth would be 0.7 to 1.2 feet at the base of the PV supports for the 100-year storm, depending on the angle of flow to the supports. Placement of riprap will provide a less significant benefit to mitigate for additional runoff. However, riprap placed at the base of each support structure will help reduce the effects of local scour and lower storm water runoff velocities.

7 Results of the qualitative fluvial geomorphologic analysis indicates existing areas of relatively inactive sediments characterized by desert pavement and more active areas consisting of finer sand and gravel. The changes to the site resulting from Project development will create an area that has consistent compaction, soil type and grading compared to existing conditions. It is anticipated that these changes will create a geologic environment conducive to the formation of shallow channels up to two feet or less in depth (i.e., long-term scour). This long term scour can be mitigated by periodic monitoring to identify changes to the site grading and maintenance activities as/if needed to restore design conditions.

The results of the modeling indicate that the DSSF development would have a small impact on off-site peak flow rate and a negligible increase in maximum degradation depth comparing pre-development conditions to post-development conditions. These impacts are relatively small. However, the implementation of storm water mitigation measures will minimize impacts of the DSSF development on sedimentation and erosion characteristics in downstream areas with the result that post-development downstream conditions are essentially the same as pre-development existing conditions.

Along with the mitigation measures, a Monitoring and Response Plan will be prepared and submitted to the BLM. The Monitoring and Response Plan will indicate the procedures that will be followed to mitigate potential impacts to the site structures, storm water infrastructure or site grading that can occur from local scour, sediment transport and long term degradation (i.e. fluvial geomorphology) during the operation of the DSSF. This plan will address monitoring of the mitigation measures after storm events and development of appropriate maintenance responses so that the mitigation measures are in good working order and continue to be effective for subsequent storm events. Because the differences are so small (i.e. within +/- 5%) and there are a number of unknowns associated with real life conditions (i.e. compared to computer simulation), it is recommended that after each significant event (e.g. a 1-year storm or larger) hydrologic, hydraulic and sediment transport characteristics to be monitored. If acute or chronic problems are detected then modifications can be made as necessary.

10 REFERENCES

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Figure 1
Desert Sunlight Solar Farm
Locus Map

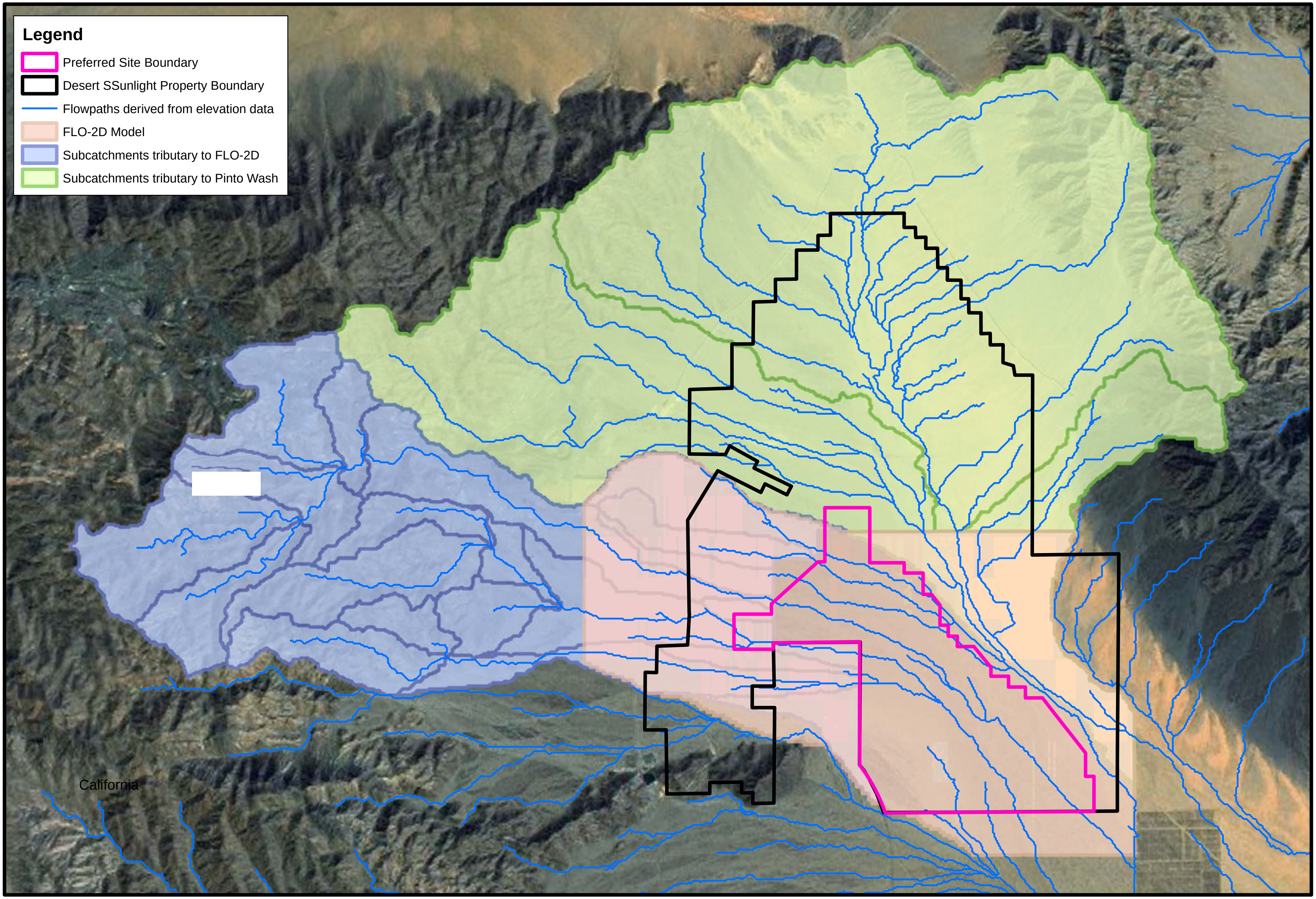

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Figure 2
Map of FLO-2D and HEC-HMS
Model Extents

Figure 3. Hyetograph of 100-year and 10-year Storm Events

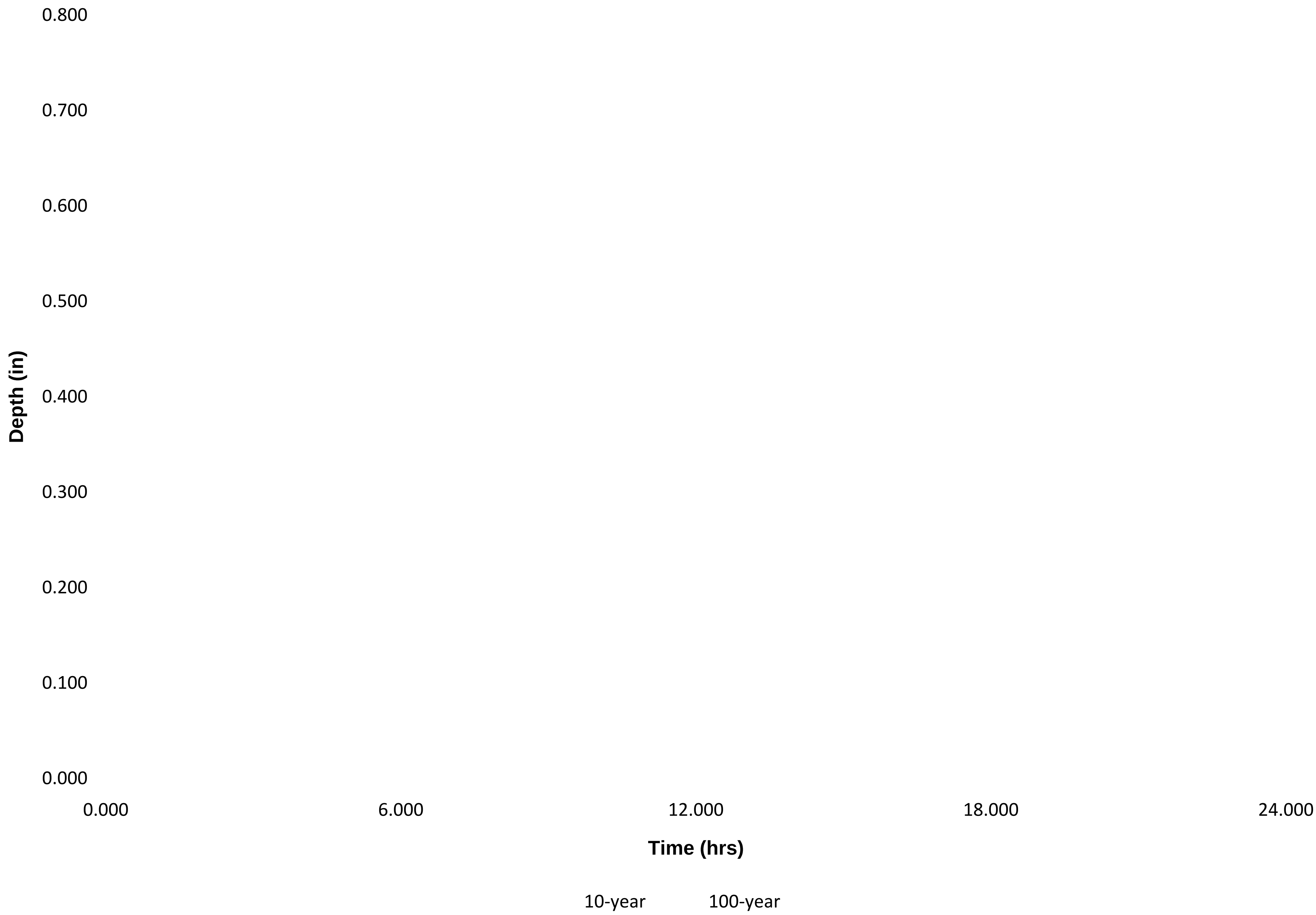
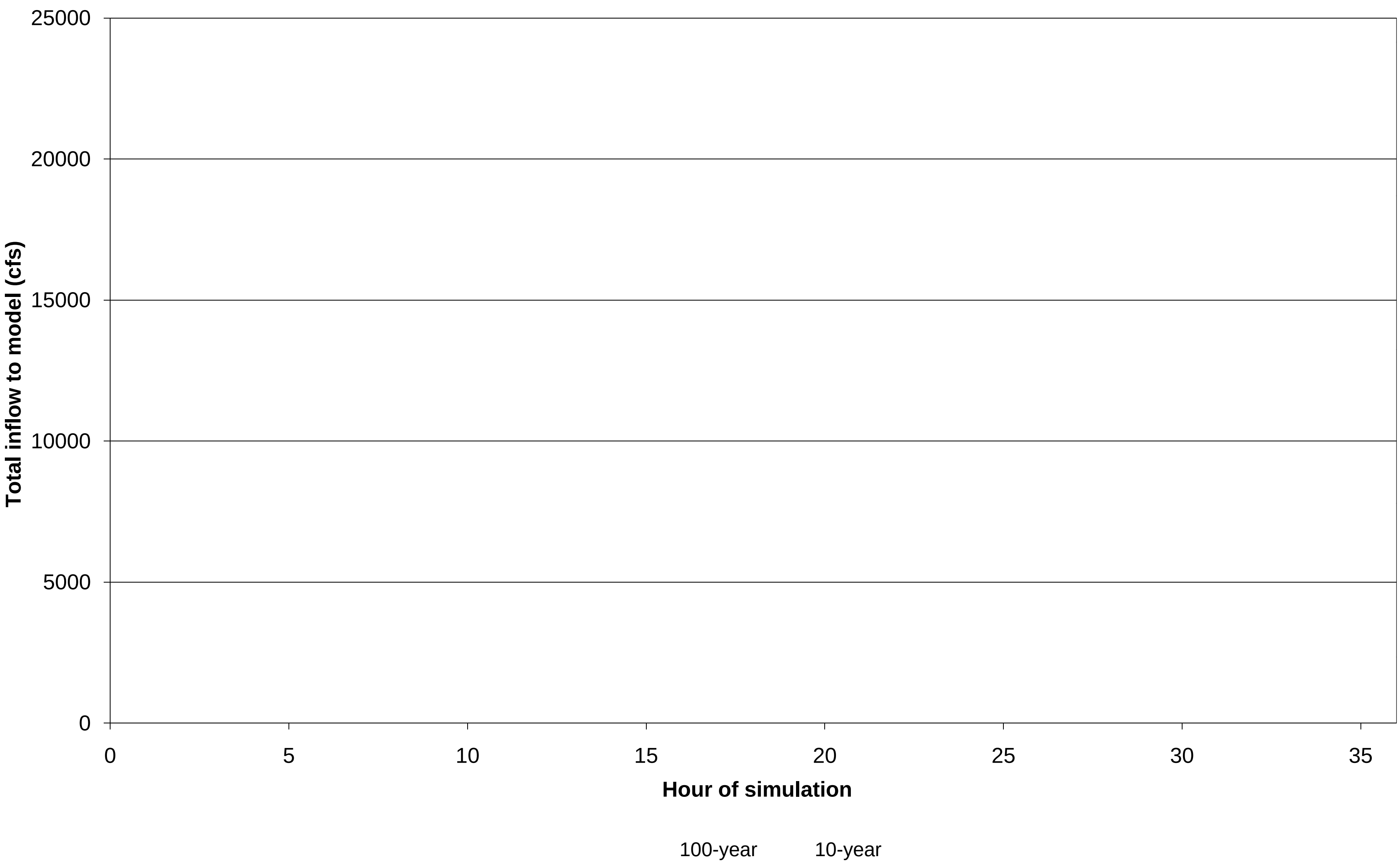
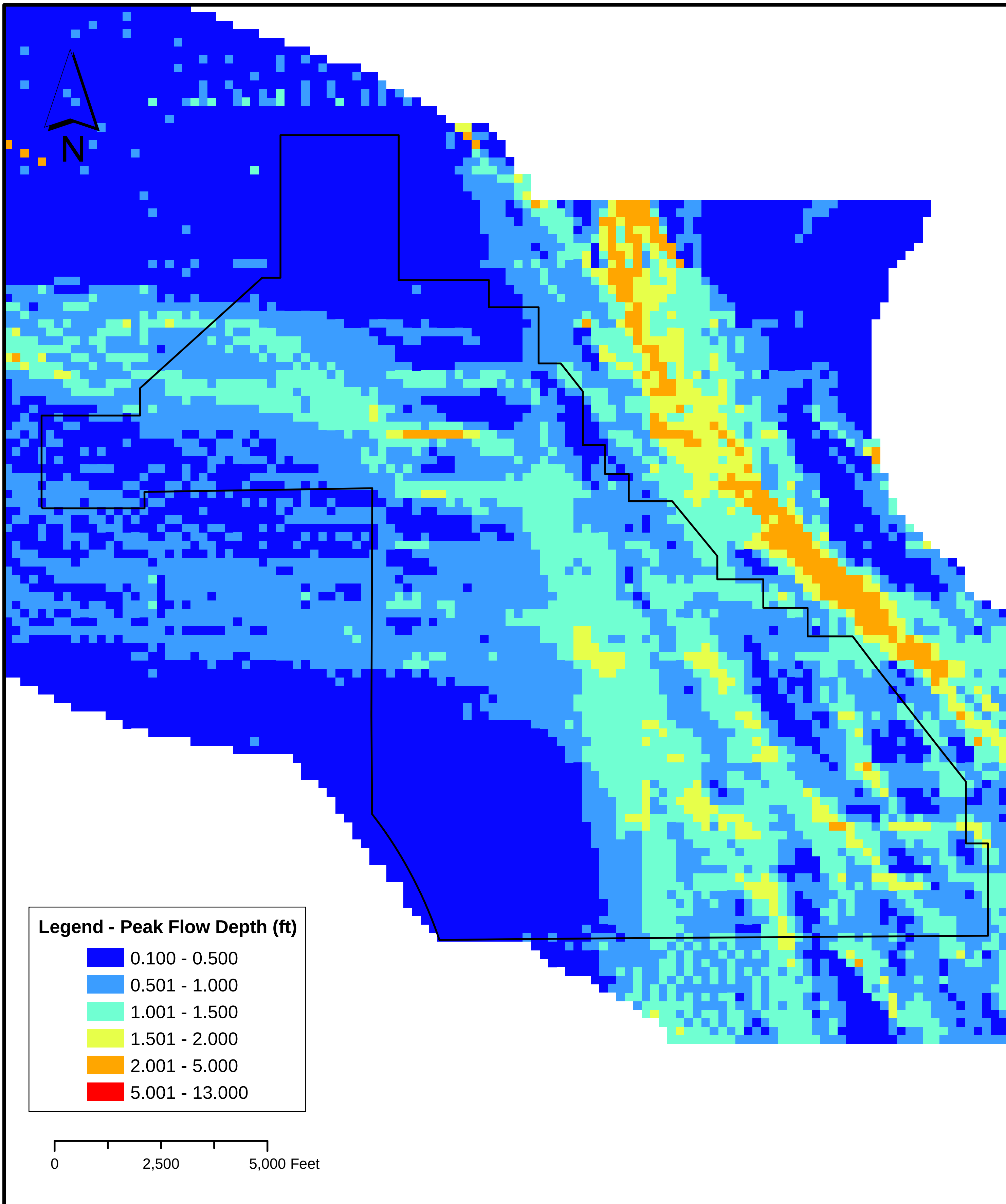


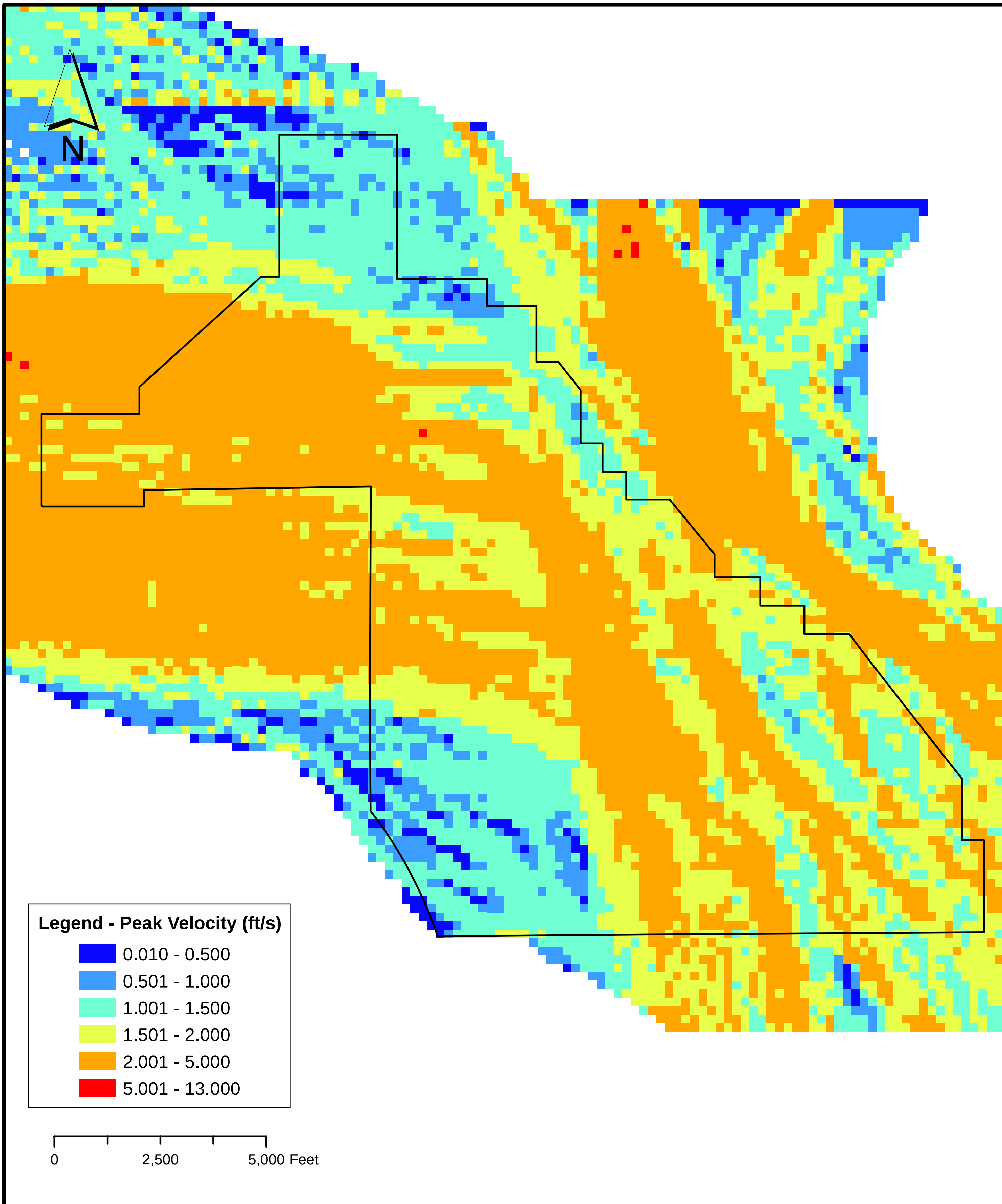
Figure 4. Estimated Inflow to Model





AECOM	DSSF - Storm Water Hydrology Report Hydrologic, Hydraulic, Sediment Transport and Scour Analyses 100 Year - Existing Conditions Peak Flow Depth

GIS FILE:	SCALE: AS NOTED	DATE: 03/12/2010	Figure 5



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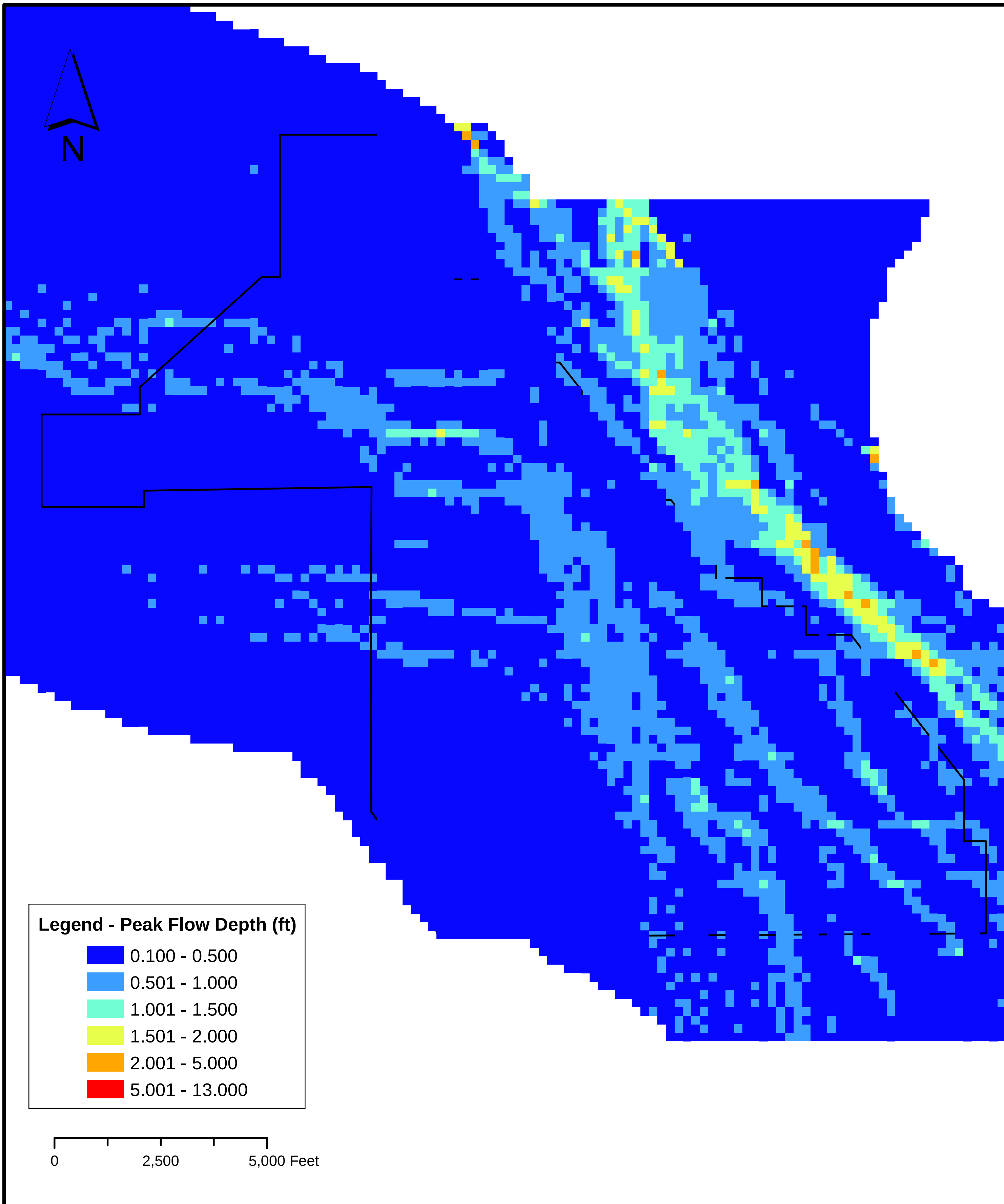
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Existing Conditions Peak Velocity

GIS FILE:	SCALE: AS NOTED	DATE: 03/12/2010	Figure 6



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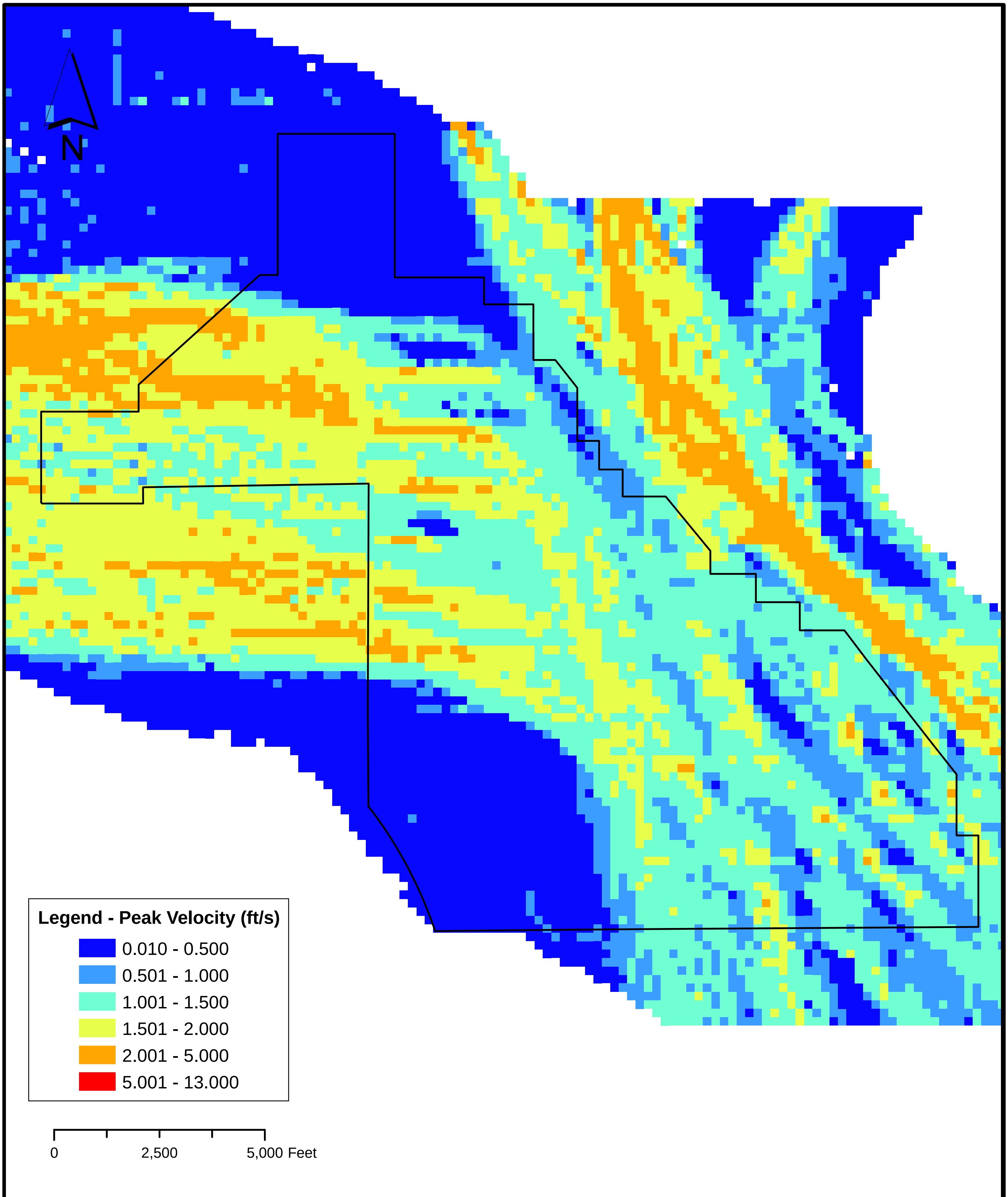
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
10 Year - Existing Conditions Peak Flow Depth

GIS FILE:	SCALE: AS NOTED	DATE: 03/12/2010	Figure 7



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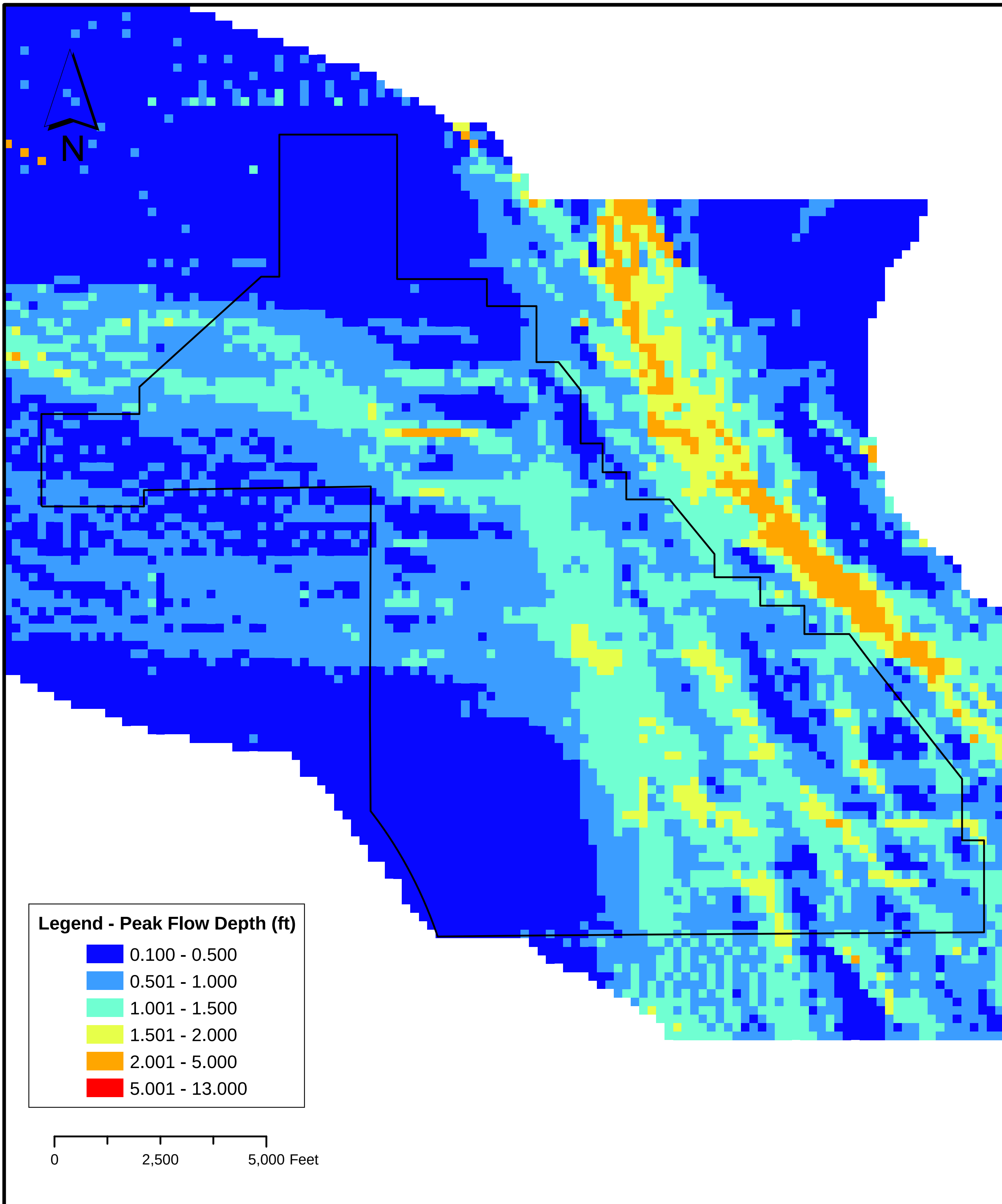
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
10 Year - Existing Conditions Peak Velocity

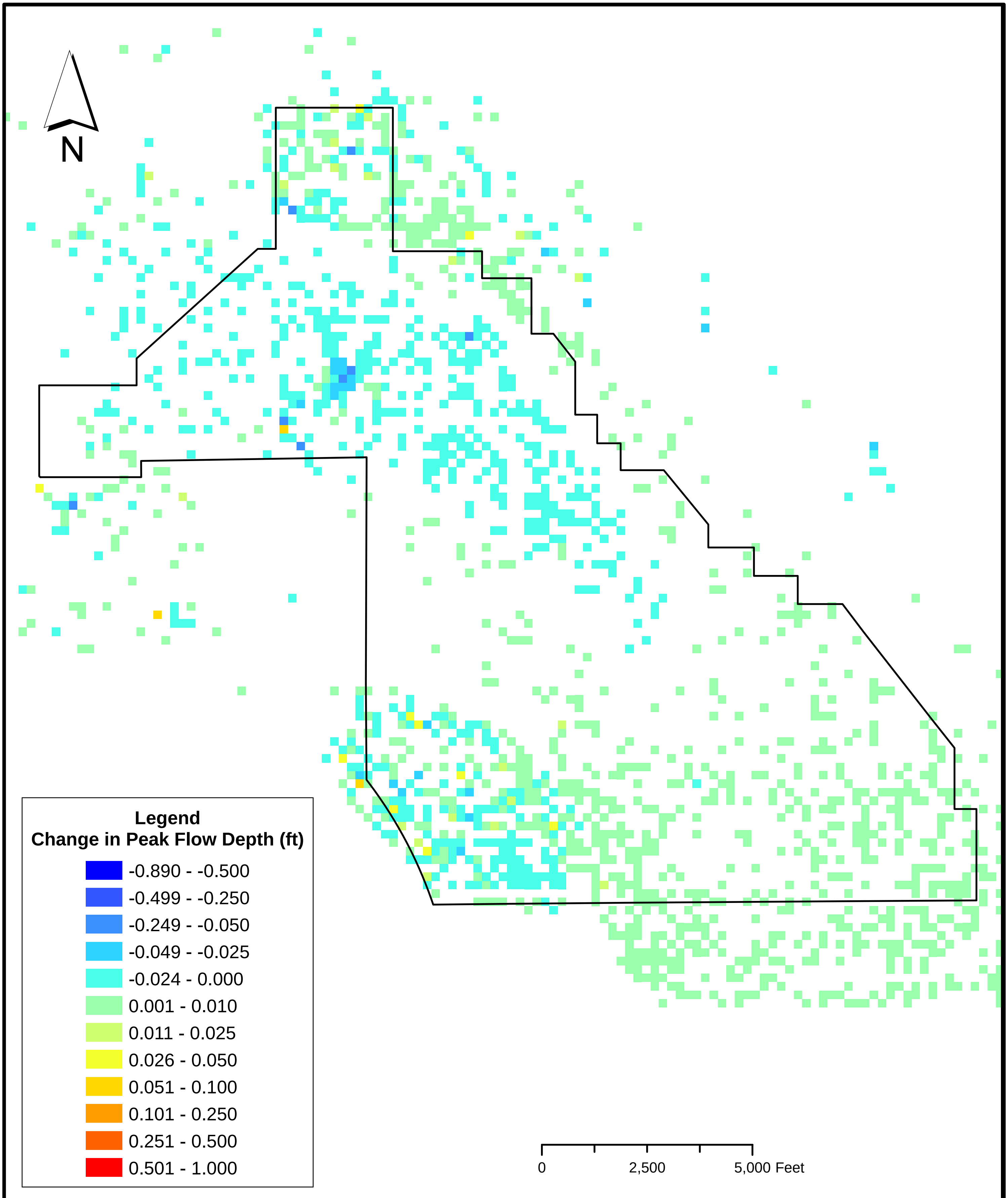
GIS FILE:	SCALE: AS NOTED	DATE: 03/12/2010	Figure 8



AECOM

DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Future Conditions Peak Flow Depth

GIS FILE:	SCALE: AS NOTED	DATE: 03/12/2010	Figure 9



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DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Change in Peak Flow Depth (Future - Existing)

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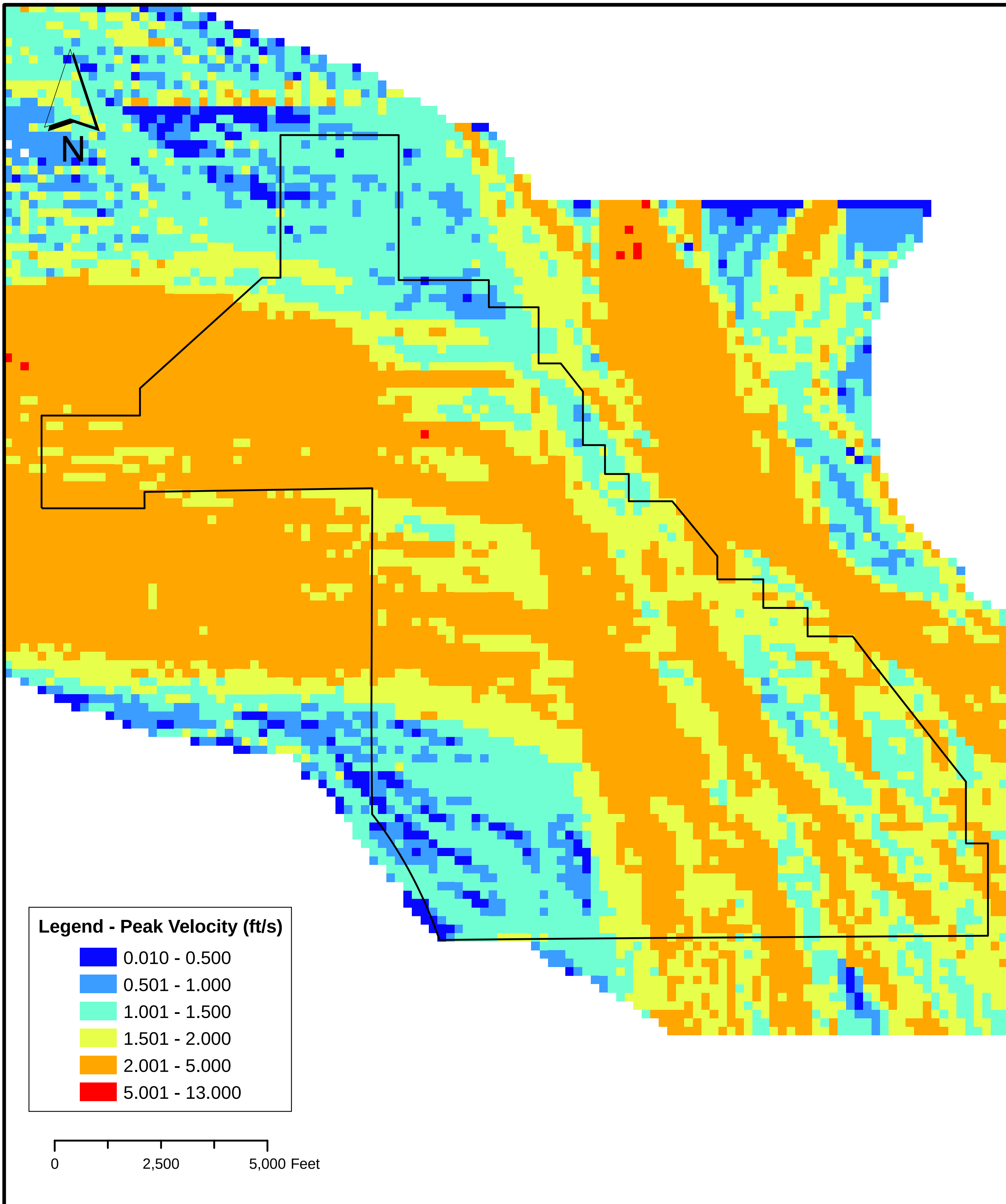

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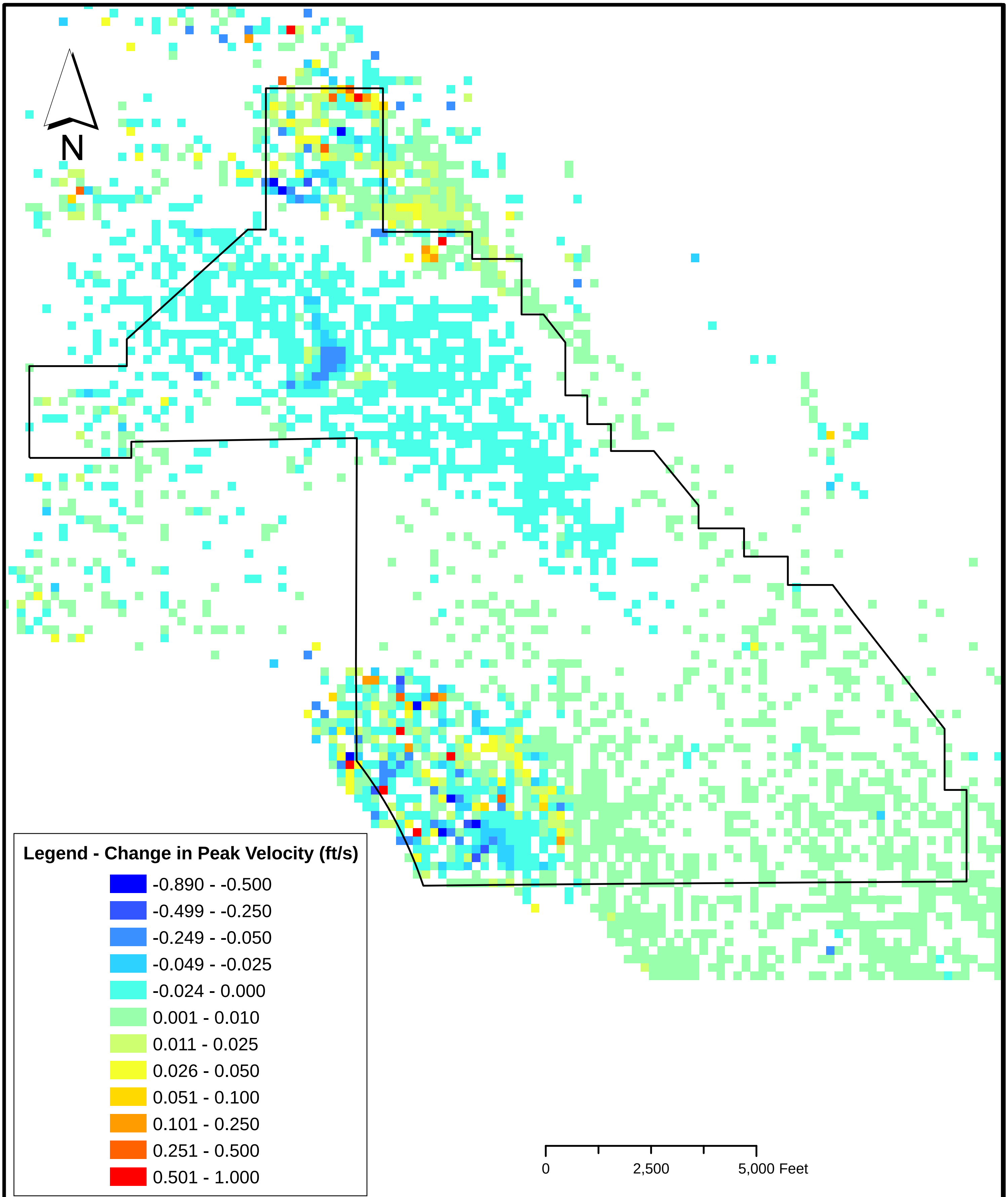
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Future Conditions Peak Velocity

GIS FILE:	SCALE: AS NOTED	DATE: 03/12/2010	Figure 11



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DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Change in Peak Velocity (Future - Existing)

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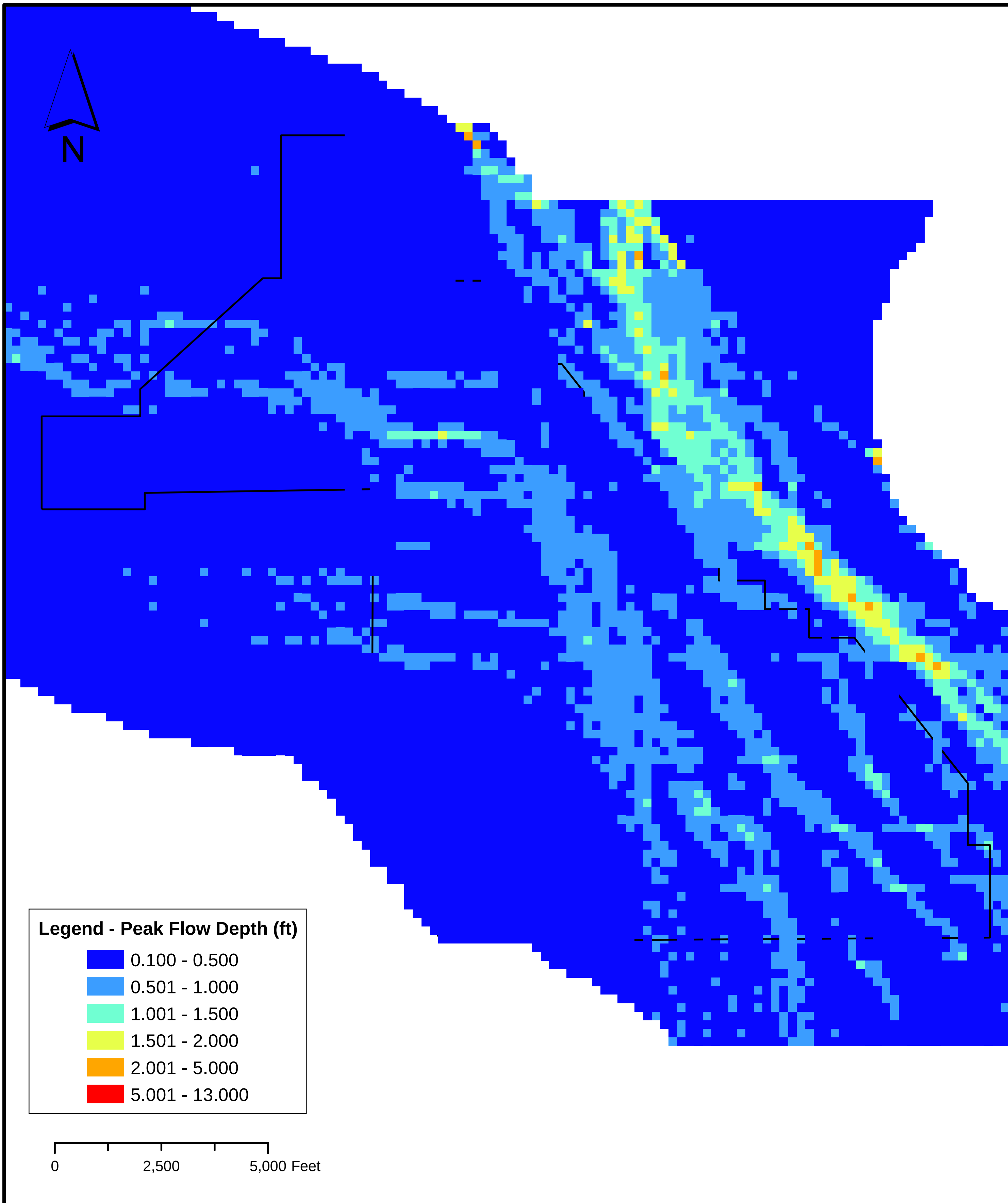

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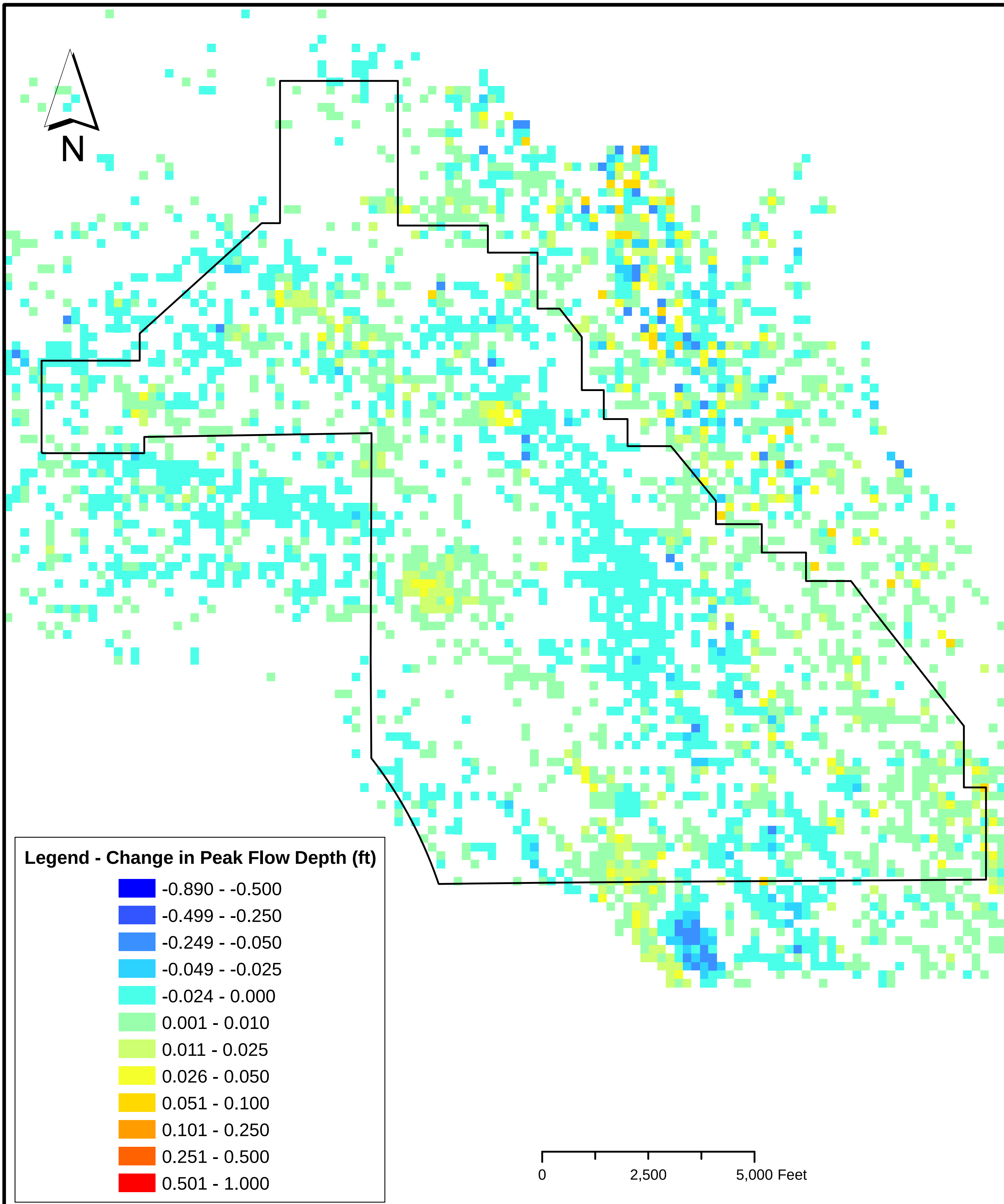
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
10 Year - Future Conditions Peak Flow Depth

GIS FILE:	SCALE: AS NOTED	DATE: 03/12/2010	Figure 13



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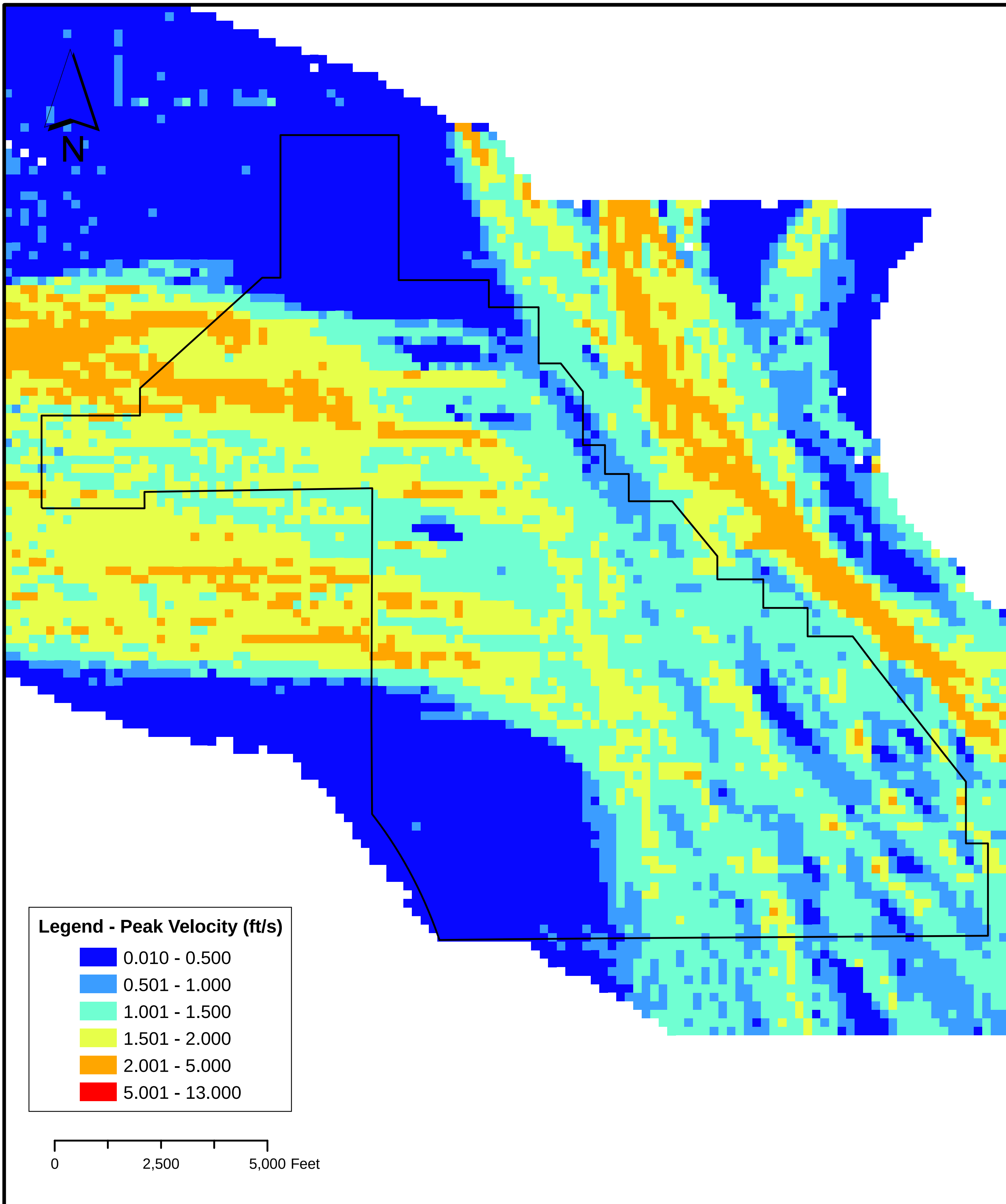
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
10 Year - Change in Peak Flow Depth (Future - Existing)

GIS FILE:	SCALE: AS NOTED	DATE: 03/12/2010	Figure 14



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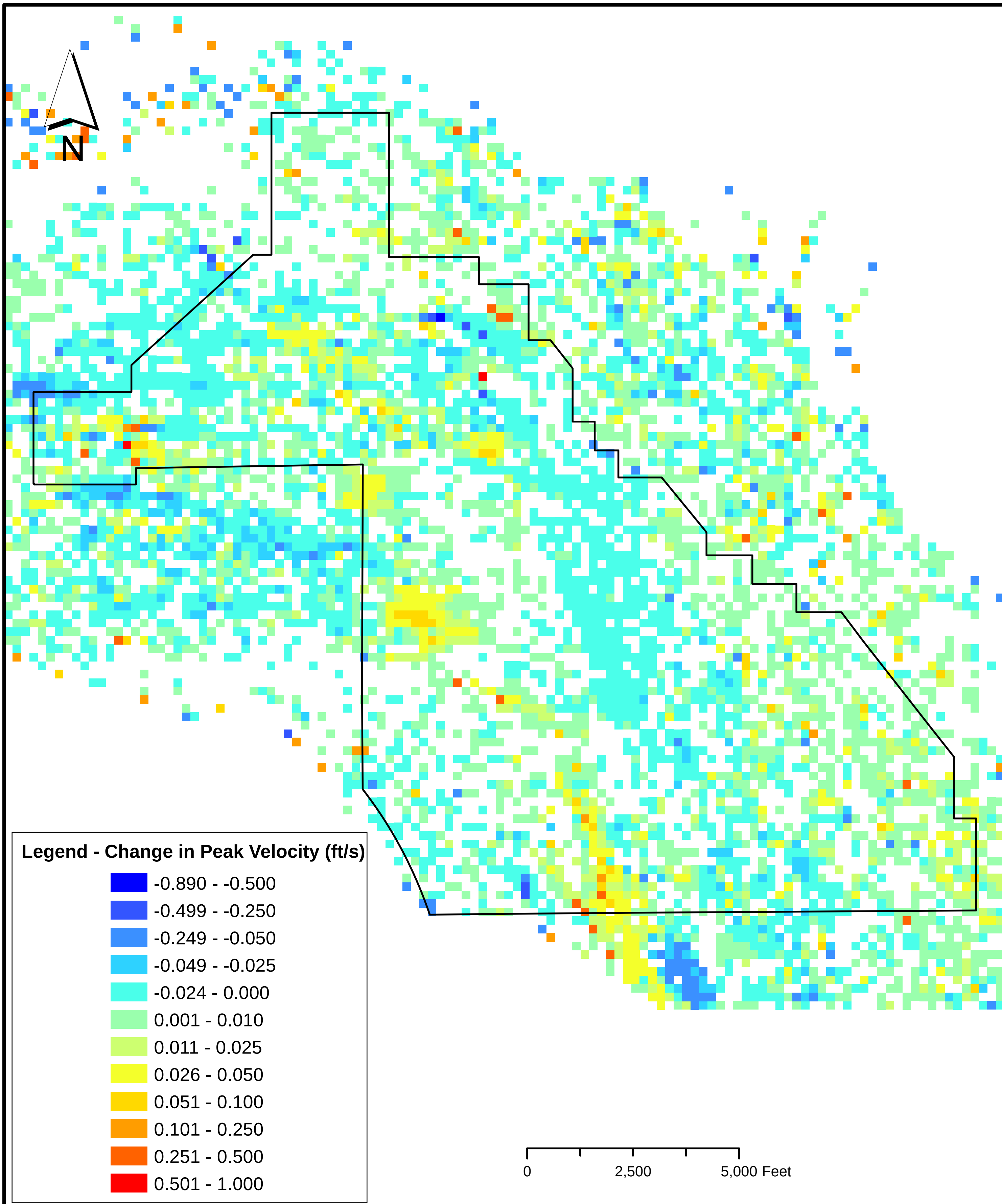
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
10 Year - Future Conditions Peak Velocity

GIS FILE:	SCALE: AS NOTED	DATE: 03/12/2010	Figure 15



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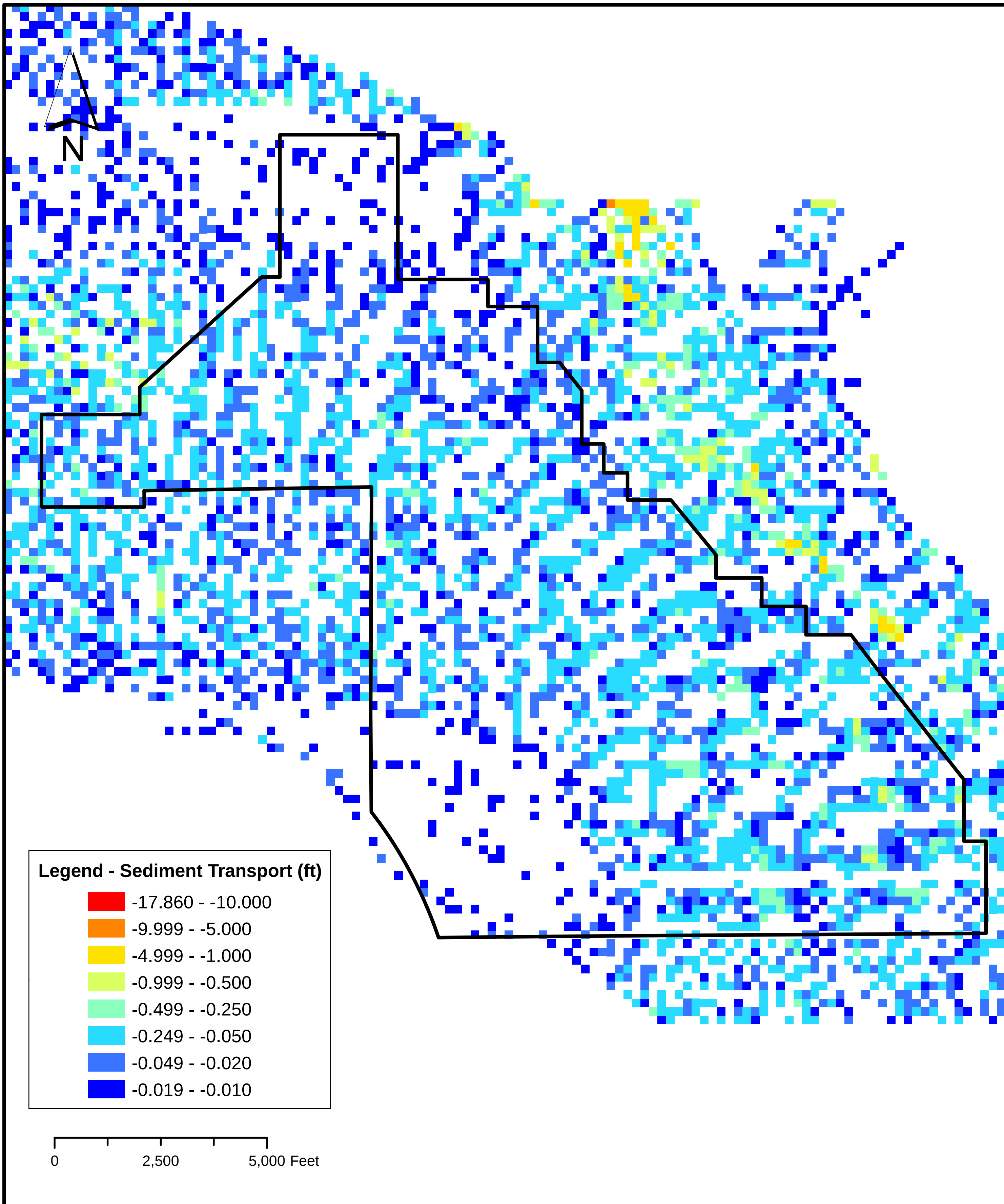
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
10 Year - Change in Peak Velocity (Future - Existing)

GIS FILE:	SCALE: AS NOTED	DATE: 03/12/2010	Figure 16



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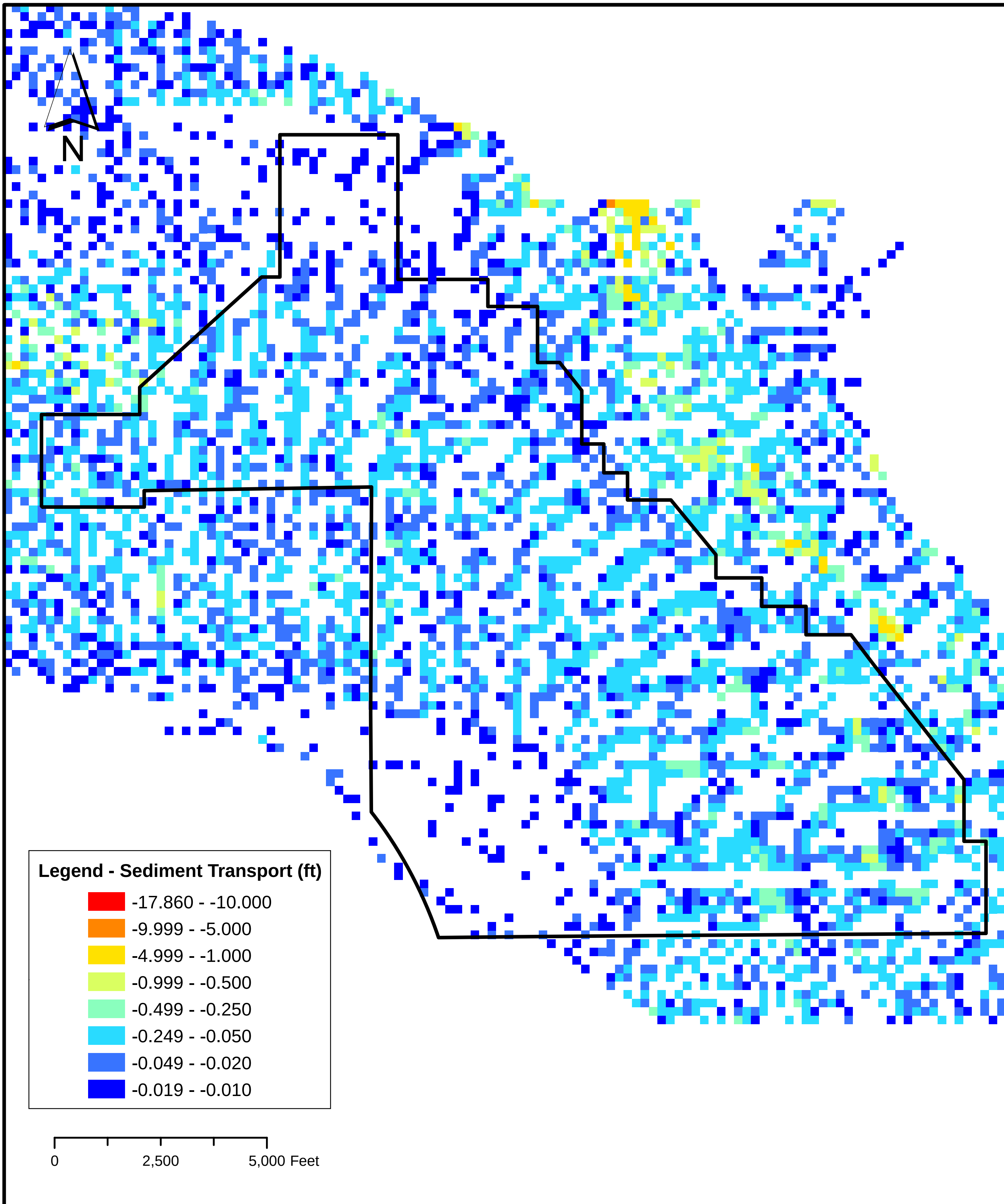
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Exisitng Conditions Maximum Sediment Transport

GIS FILE:	SCALE: AS NOTED	DATE: 03/12/2010	Figure 17



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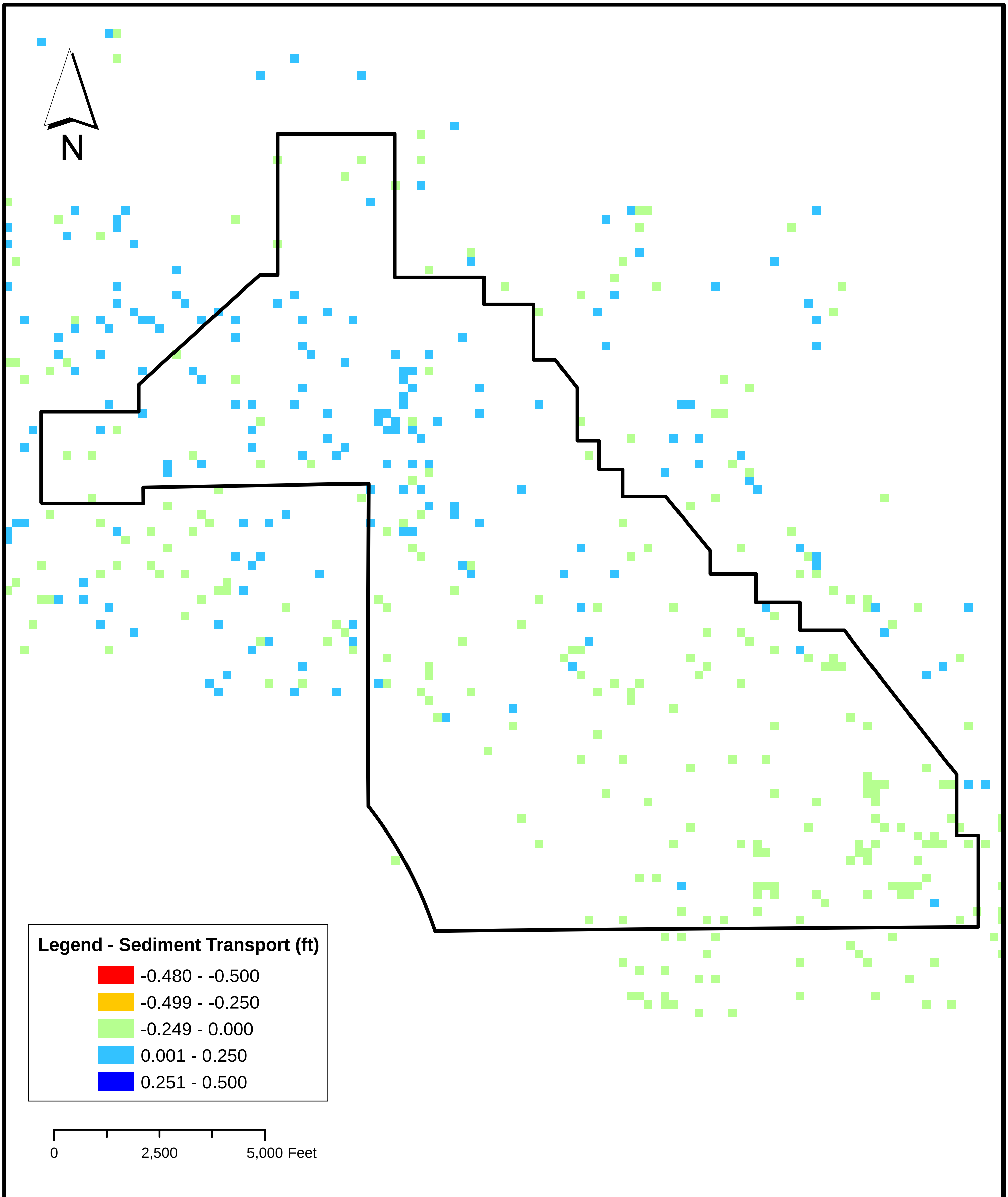
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Future Conditions Maximum Sediment Transport

GIS FILE:	SCALE: AS NOTED	DATE: 03/12/2010	Figure 18



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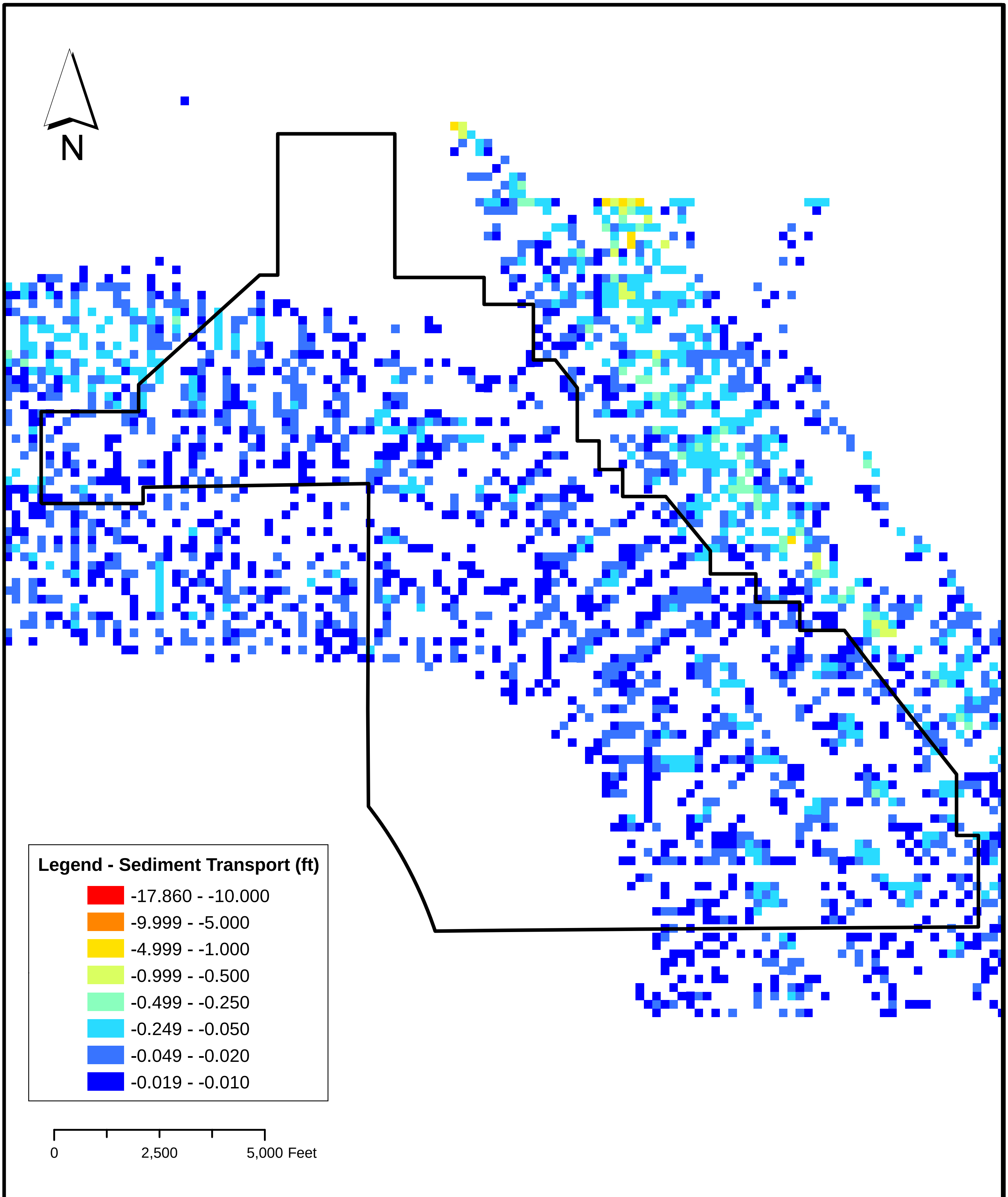
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Max Sediment Transport Change (Future - Existing)

GIS FILE:	SCALE: AS NOTED	DATE: 03/12/2010	Figure 19



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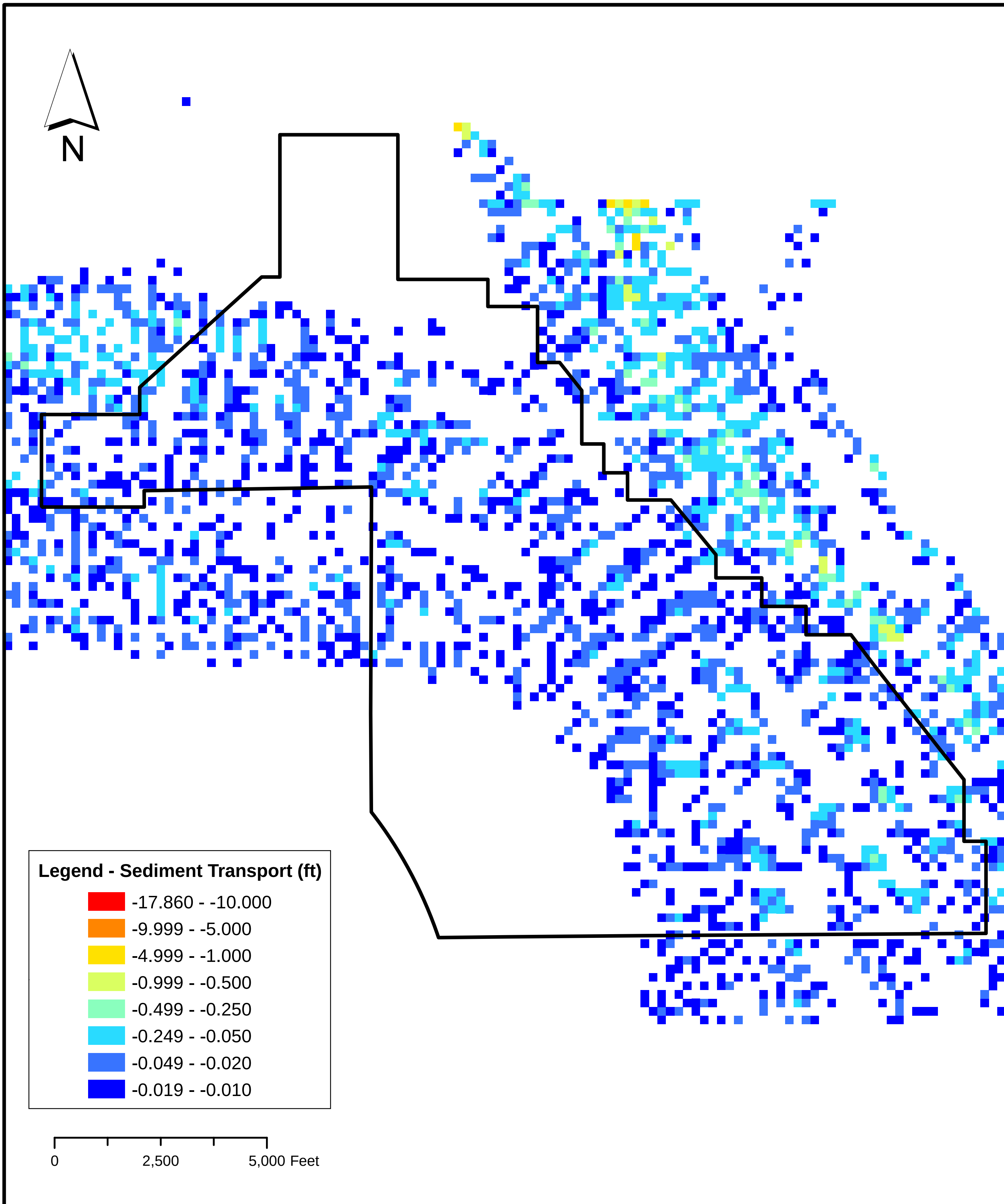
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
10 Year - Existing Conditions Maximum Sediment Transport

GIS FILE:	SCALE: AS NOTED	DATE: 03/12/2010	Figure 20



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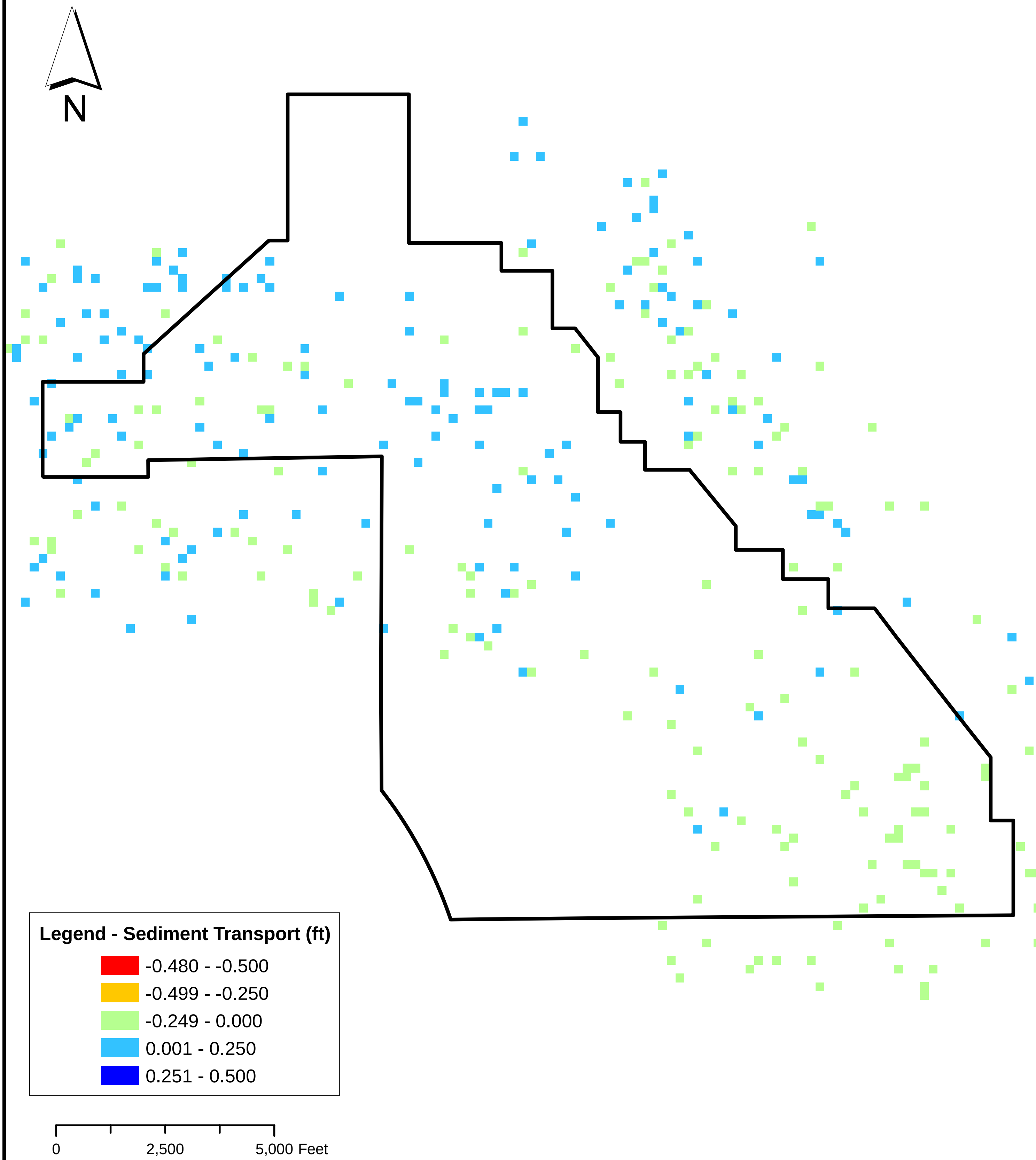
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
10 Year - Future Conditions Maximum Sediment Transport

GIS FILE:	SCALE: AS NOTED	DATE: 03/12/2010	Figure 21



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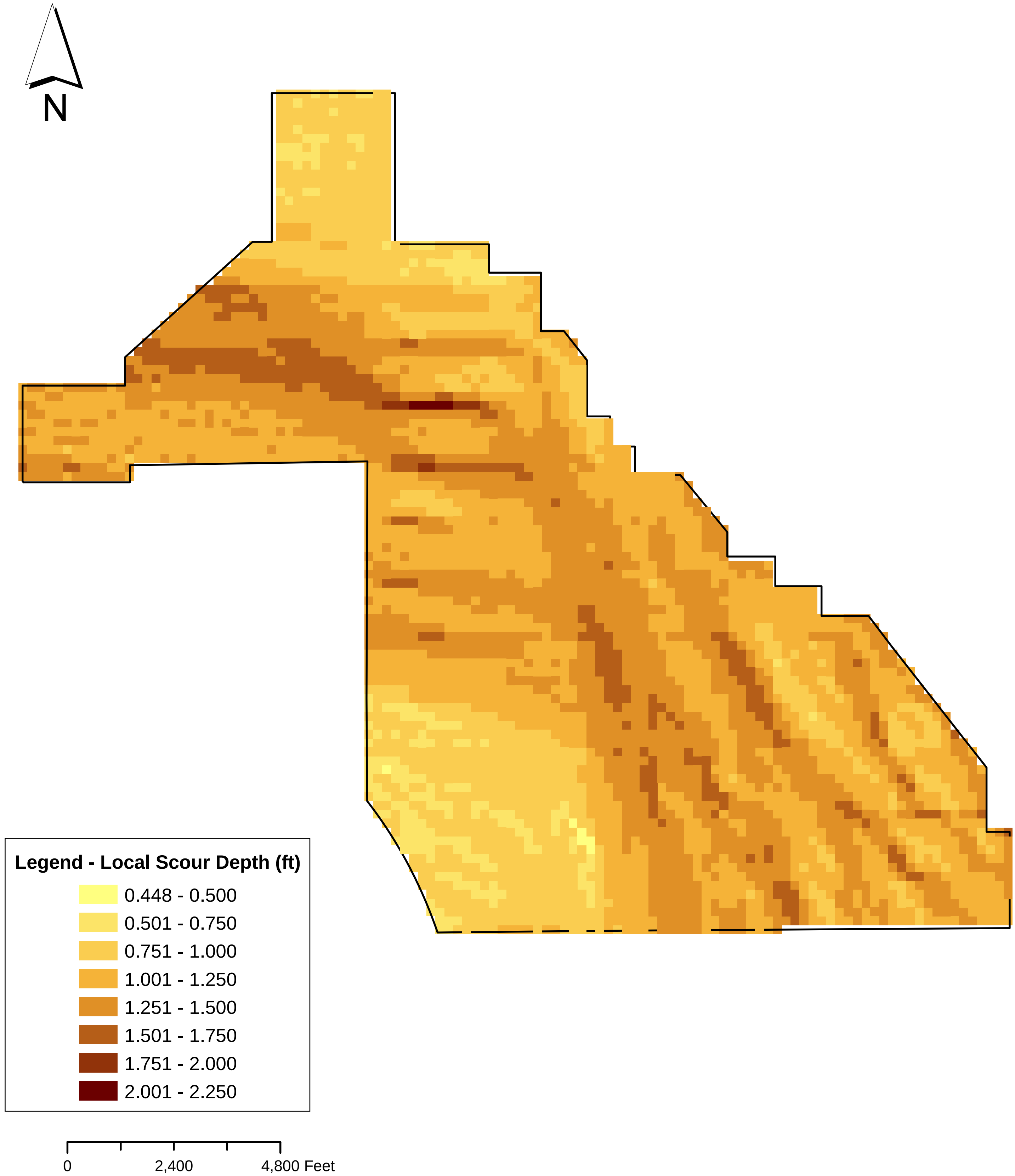
DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
10 Year - Max Sediment Transport Change (Future - Existing)

GIS FILE:	SCALE: AS NOTED	DATE: 03/12/2010	Figure 22



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DSSF - Storm Water Hydrology Report
Hydrologic, Hydraulic, Sediment Transport and Scour Analyses
100 Year - Local Scour Depth (Worst Case Scour)

GIS FILE:	SCALE: AS NOTED	DATE: 03/18/2010	Figure 23

Appendix A: Hydrologic Analysis Supporting Data

U.S. Department of Agriculture, Natural Resources Conservation Service, Urban Hydrology for Small Watersheds, Technical Release 55 (TR-55) dated June 1986 was used to estimate runoff/infiltration characteristics. Following is the table from TR-55 that contains the curve numbers used in this analysis.

Table 2-2a Runoff curve numbers for urban areas [1]

Cover description: Cover type and hydrologic condition	Average percent impervious area [2]	Curve numbers for hydrologic soil group: A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

[1] Average runoff condition, and $I_a = 0.2S$.
[2] The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.
[3] CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.
[4] Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.
[5] Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4 based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

Appendix B: Hydraulic Analysis Supporting Data

Manning's n value was used to describe surface roughness. The roughness was calculated as shown below:

Estimate for existing conditions n

Coarse Sand Floodplain	0.03
Minor irregularities	0.003
Small-Medium Vegetation	0.01
Total	**0.043**

Estimate for future conditions n

Coarse Sand Floodplain	0.03
Add Poles/Obstructions	0.004
Total	**0.034**

The addition of poles and other obstructions was assumed to be negligible to minor; occupying between 5% and 15% of the cross-sectional area.

Estimate for six (6)-inch rip-rap n

Cobble	0.039
Add Poles/Obstructions	0.004
Total	**0.043**

In order to achieve a roughness of 0.039 for the 100-year future conditions approximately 54% of the project site would need to be covered in six (6) inch diameter rip-rap. The roughness value of 0.057 for the 10-year storm event cannot be obtained with six (6) inch rip-rap

The methodology is based on the following USGS methodology:

Arcement, Jr., G.J., Schneider, V.R., "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains," United States Geological Survey Water-Supply Paper 2339.

Appendix C: Sediment Transport Analysis Supporting Data

Zeller and Fullerton equation was used in sediment transport modeling within the Flo2D modeling software framework. Flo2D model user manual states the following:

"Zeller-Fullerton Equation. Zeller-Fullerton is a multiple regression sediment transport equation for a range of channel bed and alluvial floodplain conditions. This empirical equation is a computer generated solution of the Meyer-Peter, Muller bed-load equation combined with Einstein's suspended load to generate a bed material load (Zeller and Fullerton, 1983). The bed material discharge qs is calculated in cfs per unit width as follows:

$$qs = 0.0064\ n1.77\ V4.32\ G0.45\ d\text{-}0.30\ D50\text{-}0.61$$

where n is Manning's roughness coefficient, V is the mean velocity, G is the gradation coefficient, d is the hydraulic depth and D50 is the median sediment diameter. All units in this equation are in the ft-lb-sec system except D50, which is in millimeters. For a range of bed material from 0.1 mm to 5.0 mm and a gradation coefficient from 1.0 to 4.0, Julien (1995) reported that this equation should be accurate with 10% of the combined Meyer-Peter Muller and Einstein equations. The Zeller-Fullerton equation assumes that all sediment sizes are available for transport (no armoring). The original Einstein method is assumed to work best when the bedload constitutes a significant portion of the total load (Yang, 1996)."

Also the Flo2D model user manual recommends the following:

"Summary. Yang (1996) made several recommendations for the application of total load sediment transport formulas in the absence of measured data. These recommendations have been expanded to all the equations in the FLO-2D and are slightly edited:

- *Use Zeller and Fullerton equation when the bedload is a significant portion of the total load.*
- *Use Toffaleti's method for large sand-bed rivers.*
- *Use Yang's equation for sand and gravel transport in natural rivers.*
- *Use Ackers-White or Engelund-Hansen equations for subcritical flow in lower sediment transport regime.*
- *Use Lausen's formula for shallow rivers with silt and fine sand.*
- *Use MPM-Woo's relationship for steep slope, arroyo sand bed channels and alluvial fans. "*

Appendix D: Fluvial Geomorphology Analysis Supporting Data

The following historical aerial photos were used in studying fluvial geomorphology of the Project site.

Desert Solar Farm Site
Riverside County
Desert Center, CA 92239

Inquiry Number: 2624402.1
October 30, 2009

The EDR Aerial Photo Decade Package

EDR® Environmental Data Resources Inc

440 Wheelers Farms Road
Milford, CT 06461
800.352.0050
www.edrnet.com

EDR Aerial Photo Decade Package

Environmental Data Resources, Inc. (EDR) Aerial Photo Decade Package is a screening tool designed to assist environmental professionals in evaluating potential liability on a target property resulting from past activities. EDRs professional researchers provide digitally reproduced historical aerial photographs, and when available, provide one photo per decade.

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with any questions or comments.

Date EDR Searched Historical Sources:
Aerial Photography October 30, 2009

Target Property:
Riverside County
Desert Center, CA 92239

Year	***Scale***	***Details***	***Source***
1978	Aerial Photograph. Scale: 1"=1000'	Flight Year: 1978	Nasa
1978	Aerial Photograph. Scale: 1"=1000'	Flight Year: 1978	Nasa
1978	Aerial Photograph. Scale: 1"=1000'	Flight Year: 1978	Nasa
1978	Aerial Photograph. Scale: 1"=1000'	Flight Year: 1978	Nasa
1996	Aerial Photograph. Scale: 1"=1000'	Flight Year: 1996	USGS
1996	Aerial Photograph. Scale: 1"=1000'	Flight Year: 1996	USGS
1996	Aerial Photograph. Scale: 1"=1000'	Flight Year: 1996	USGS
1996	Aerial Photograph. Scale: 1"=1000'	Flight Year: 1996	USGS
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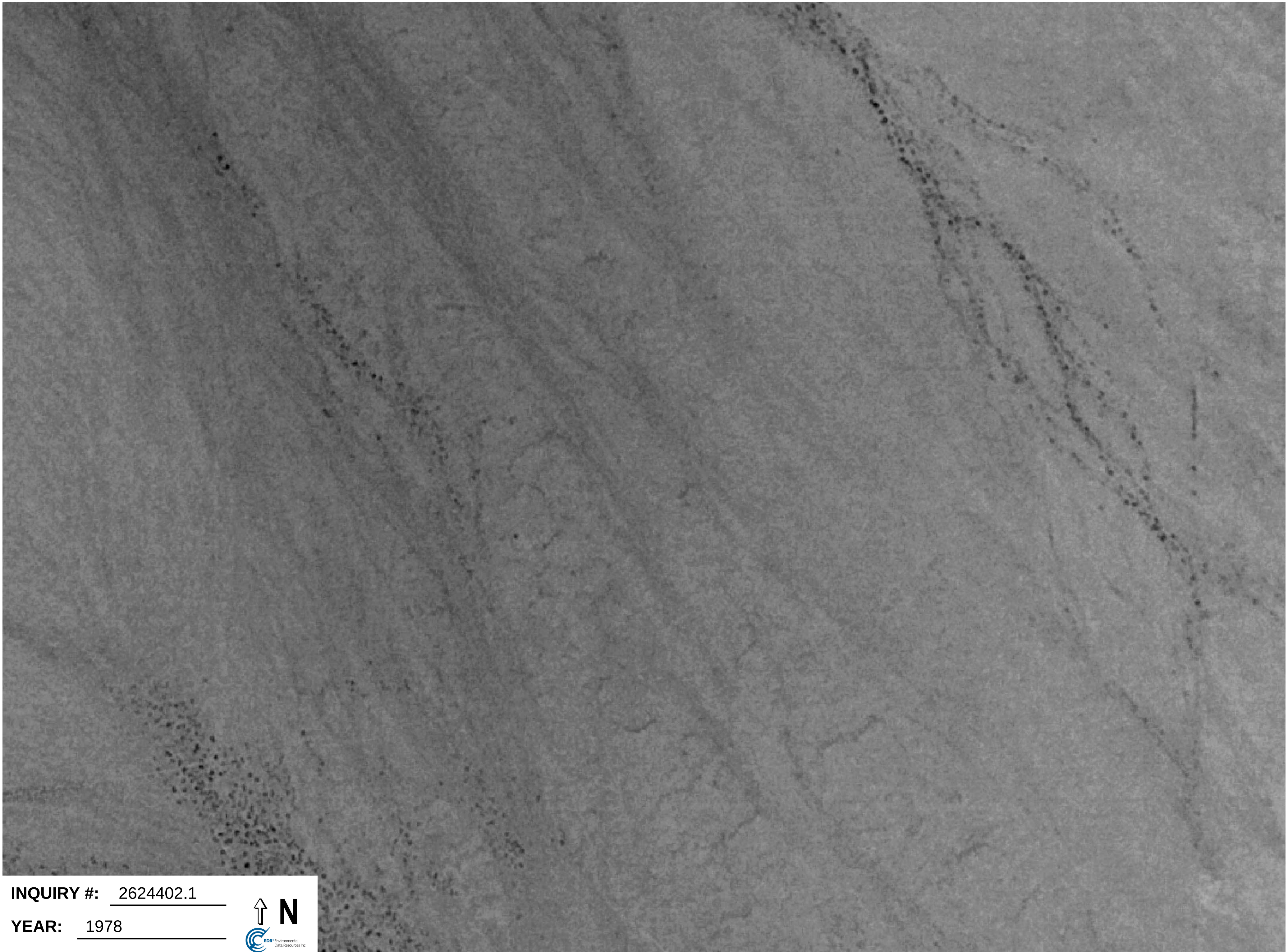

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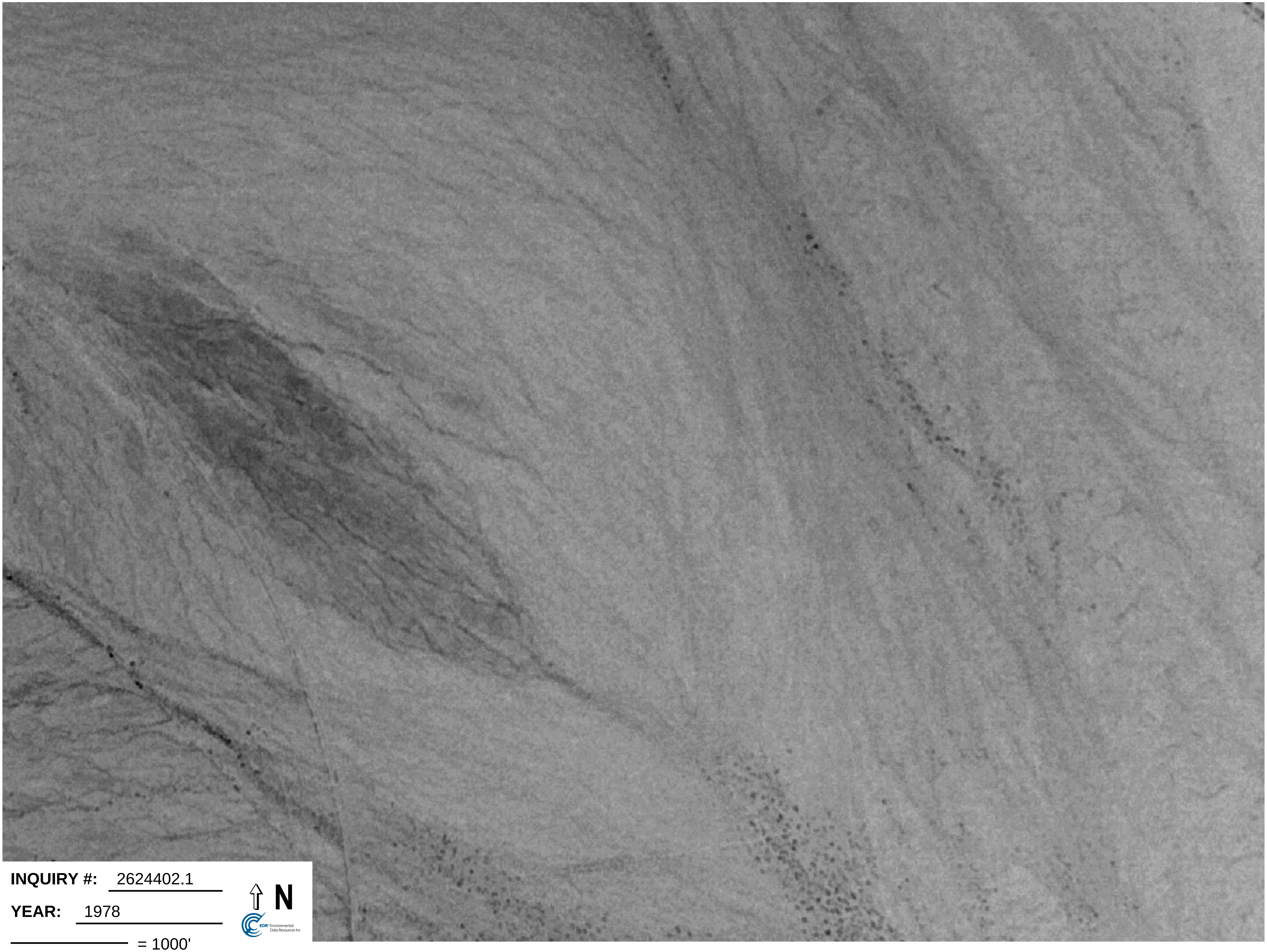

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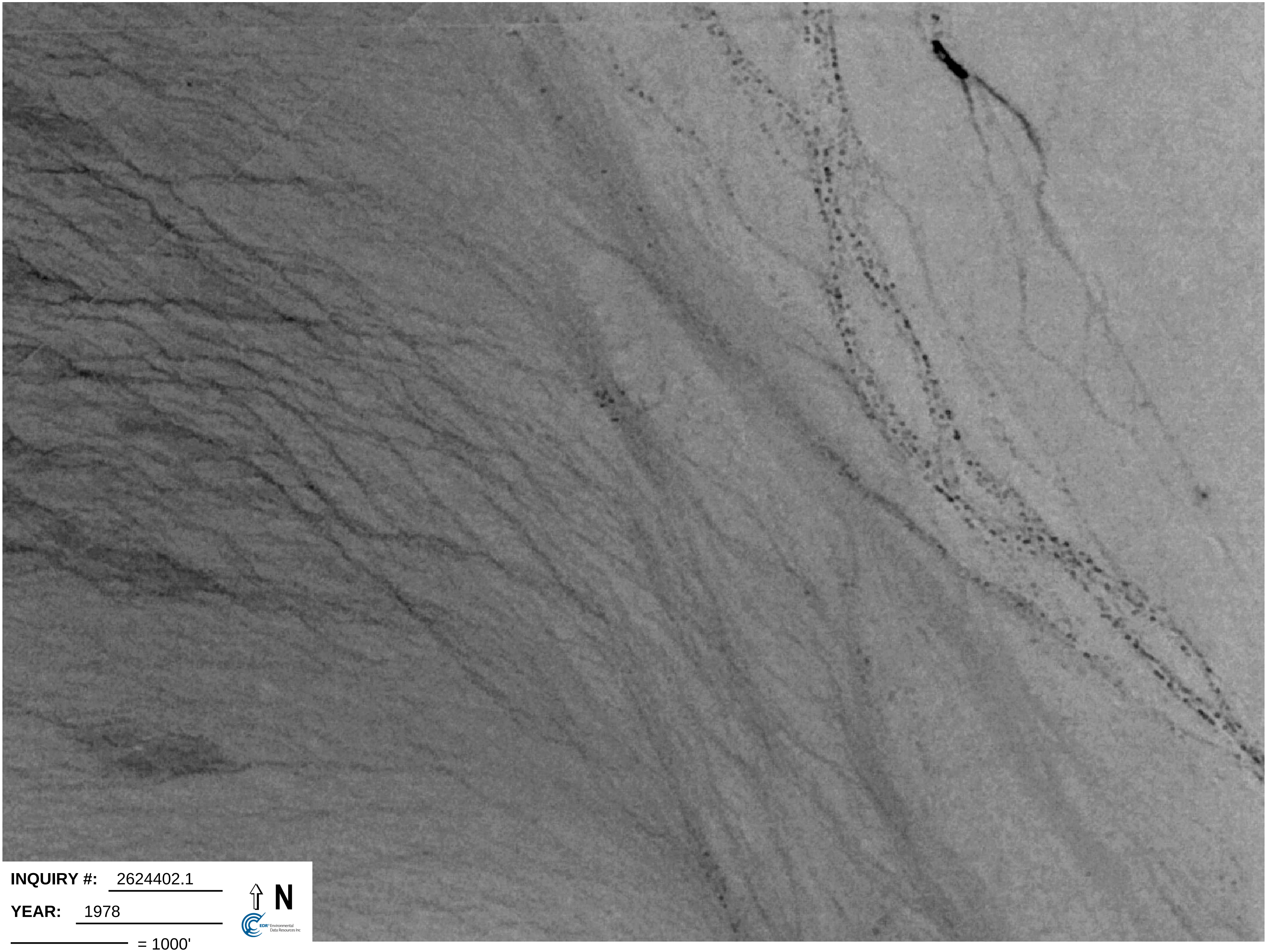

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INQUIRY #: 2624402.1
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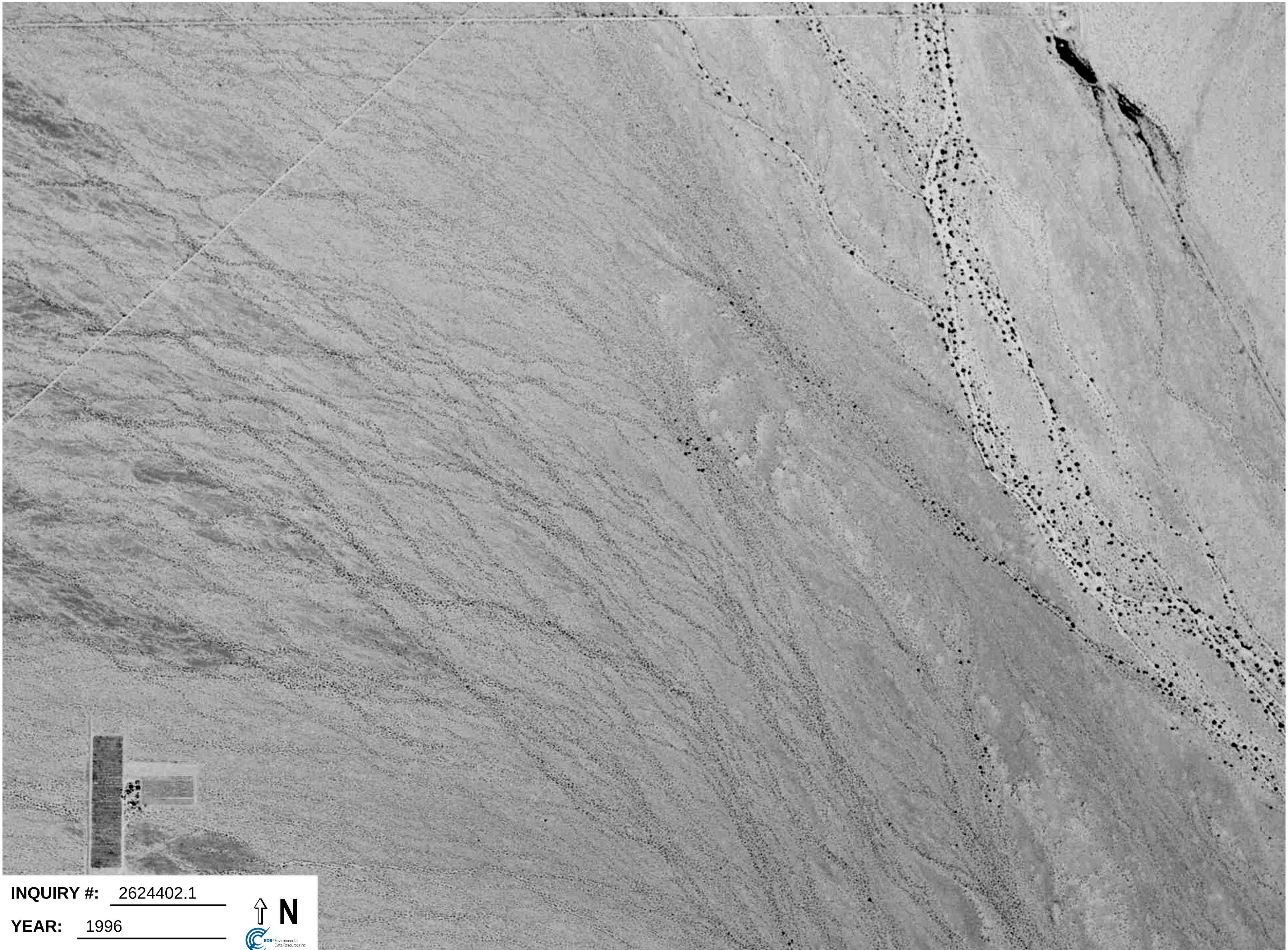
EDR Environmental Data Resources Inc



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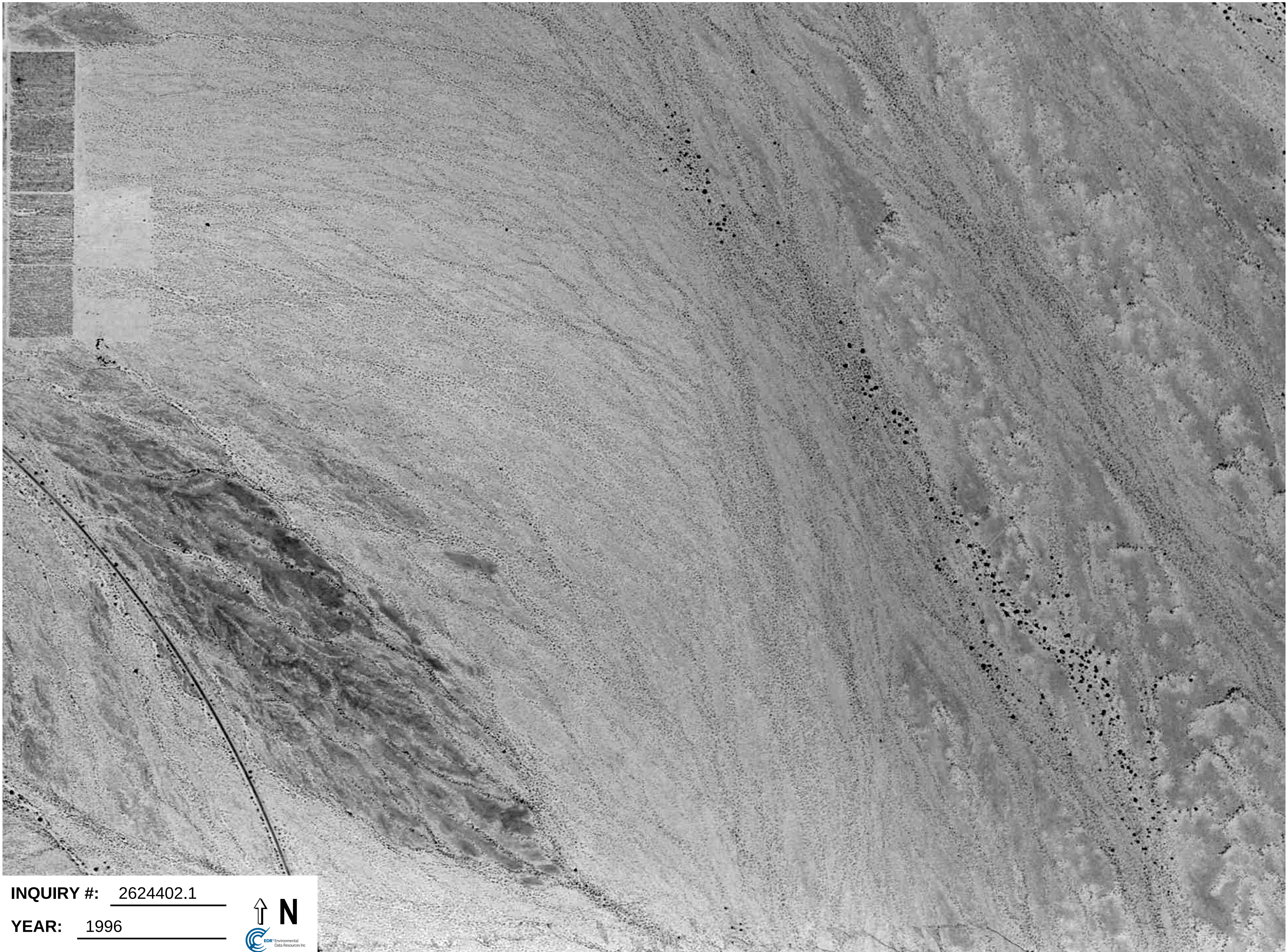

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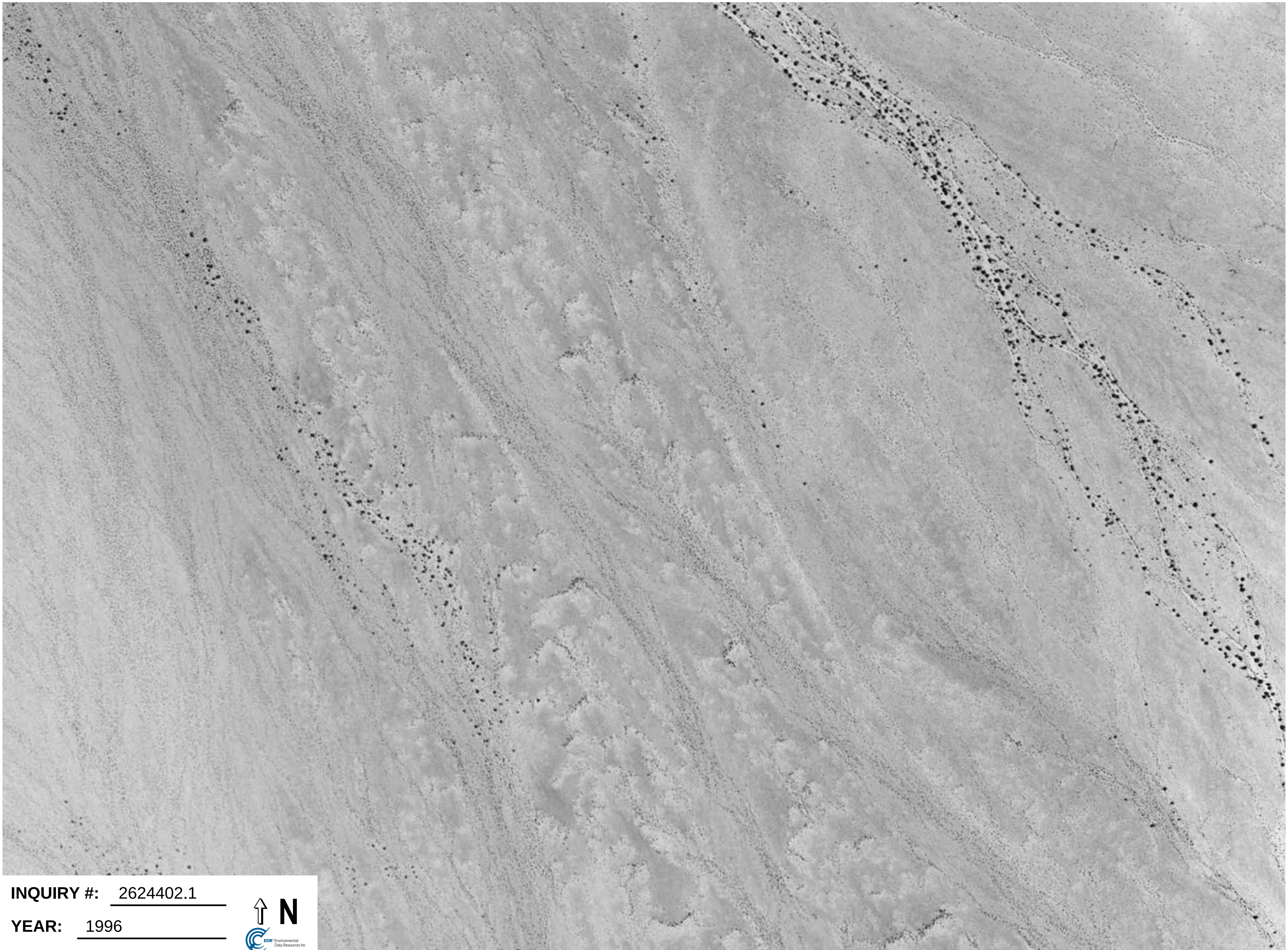

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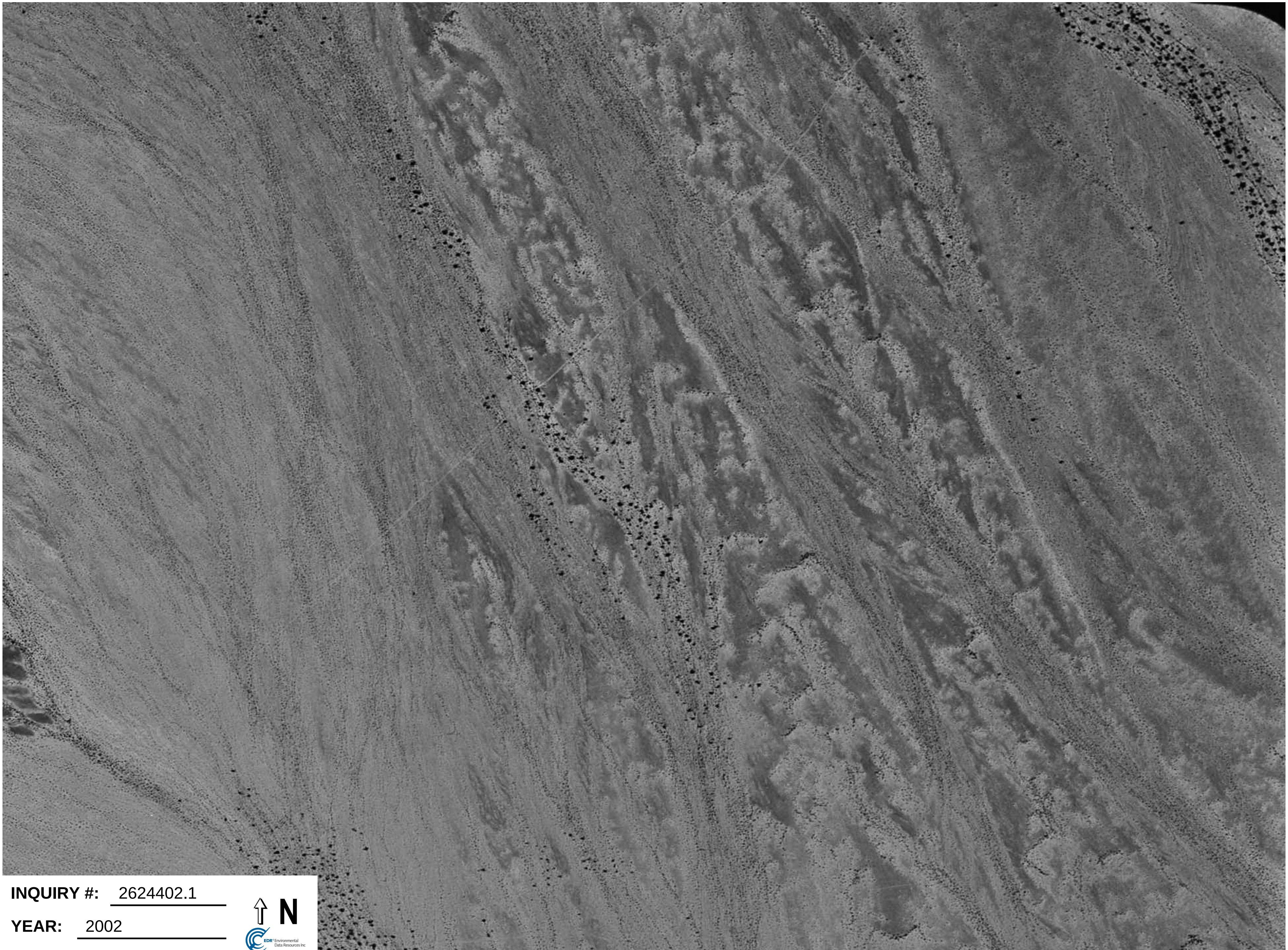

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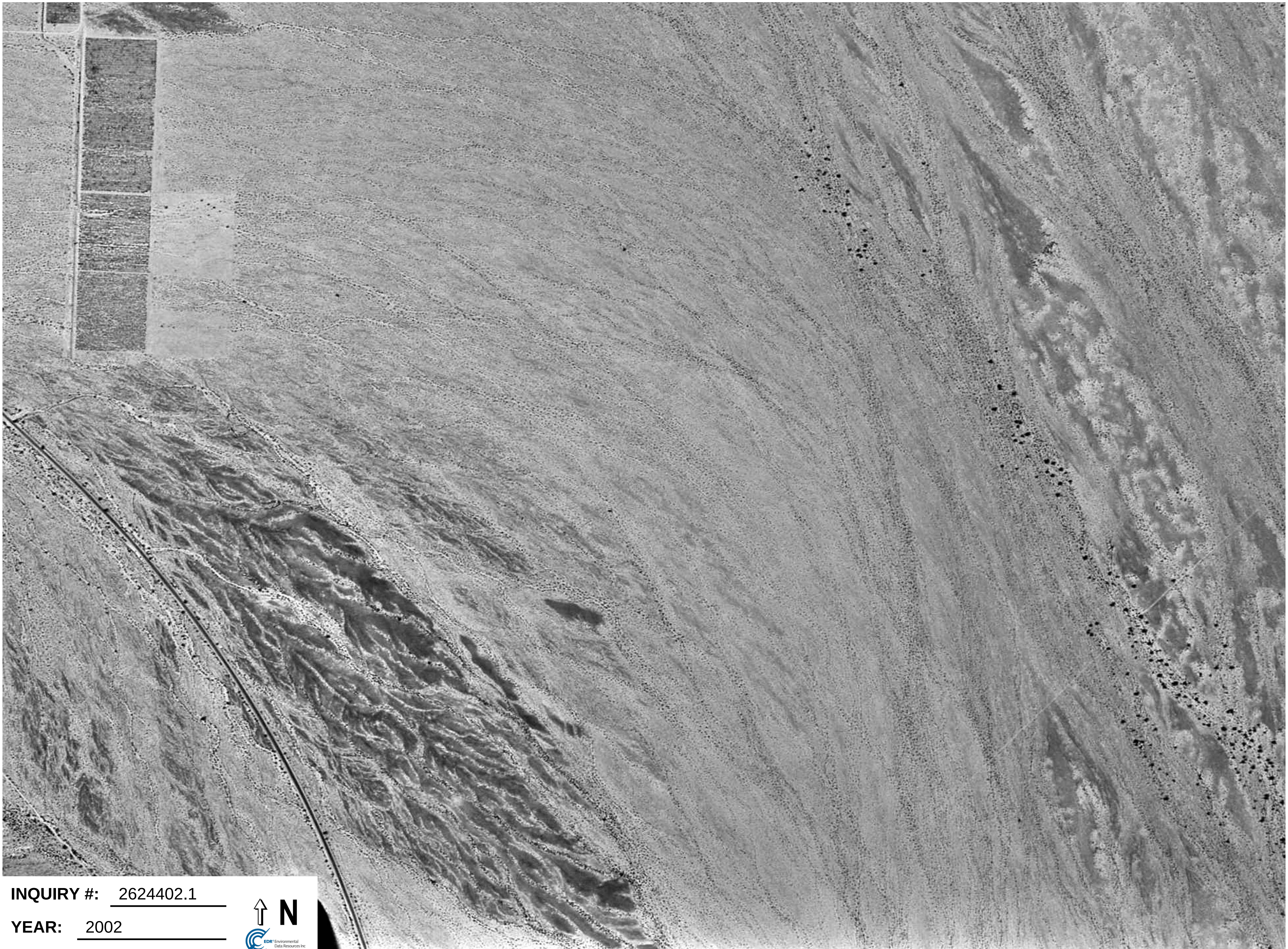

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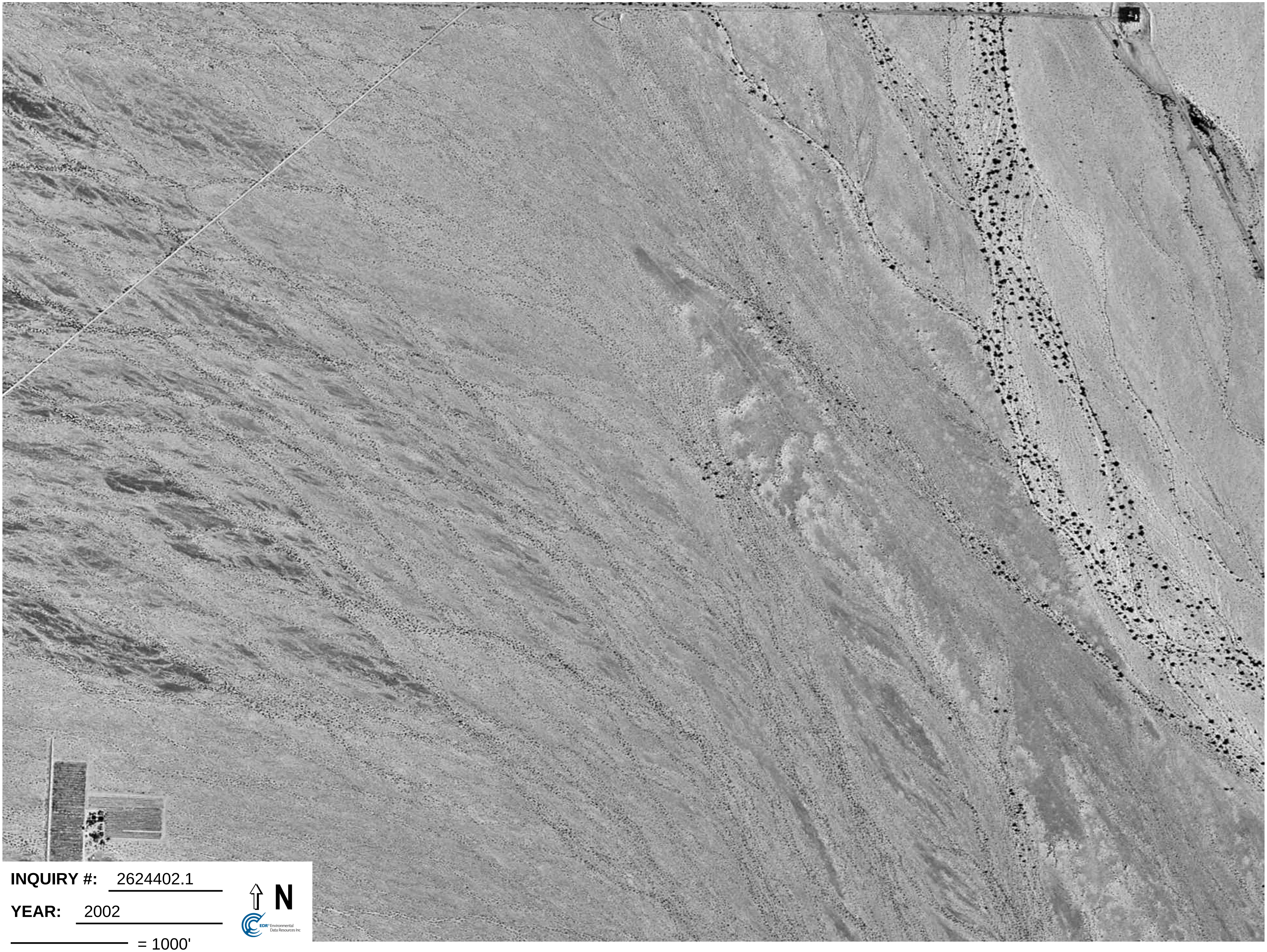

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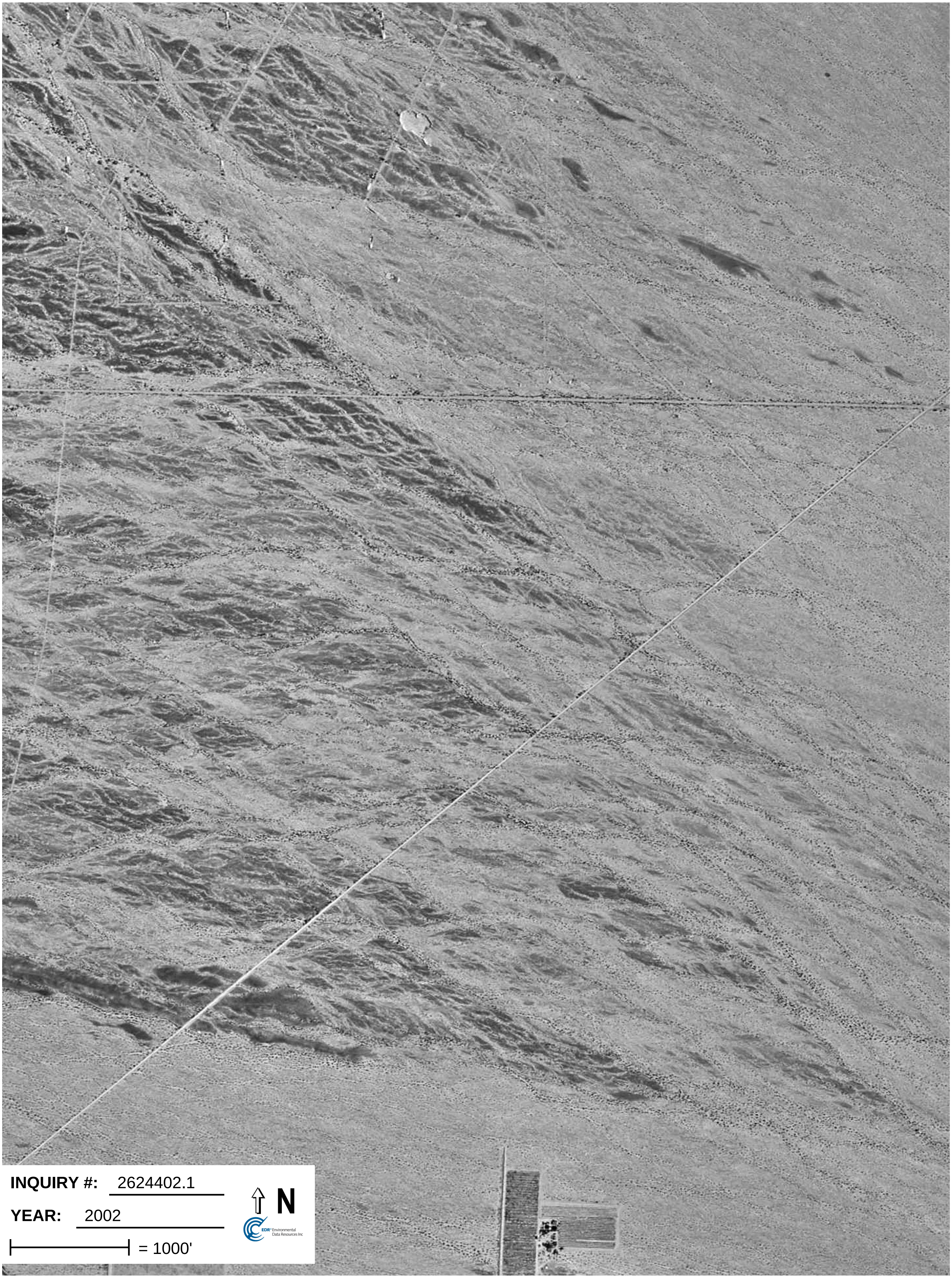

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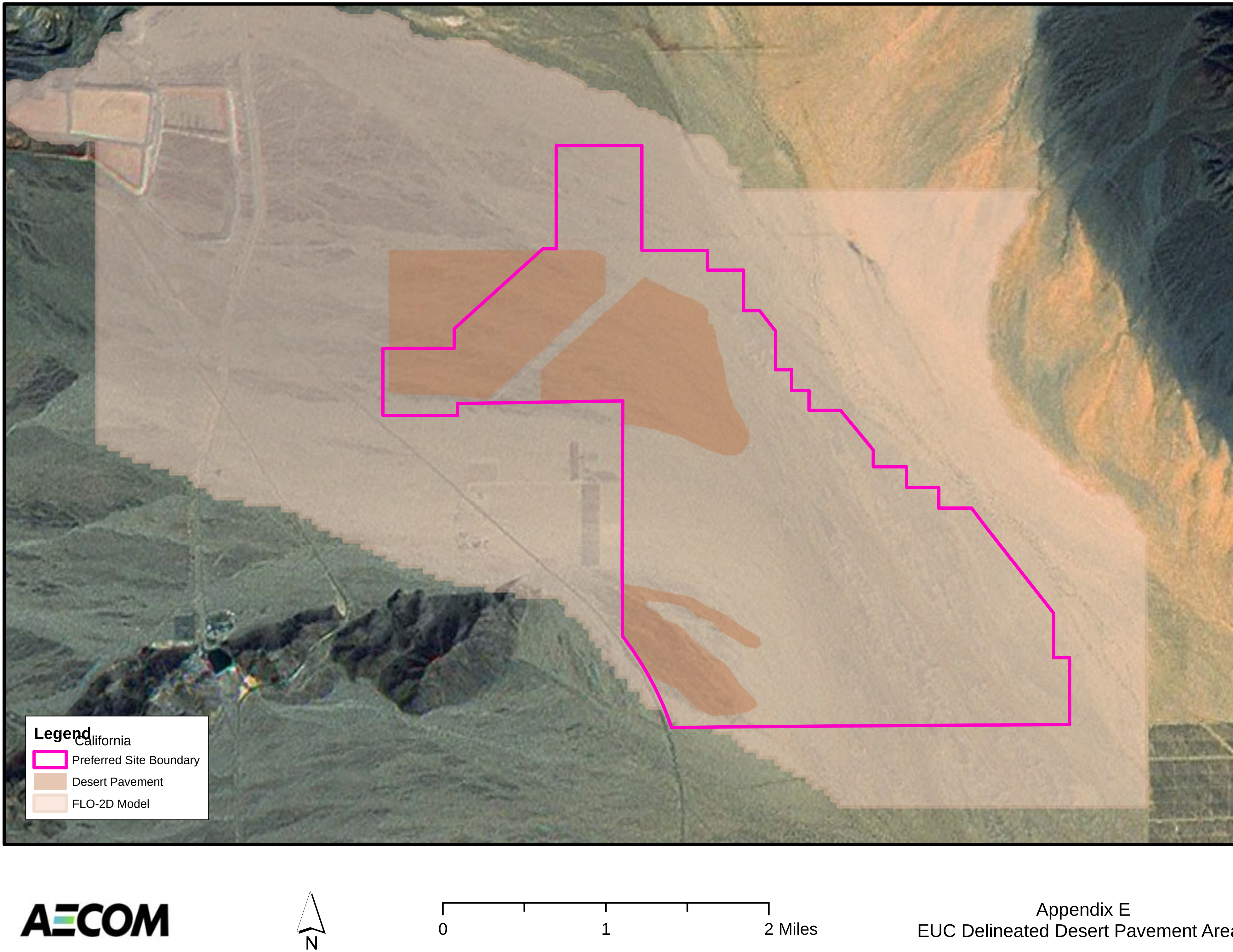
EDR Environmental Data Resources Inc

Appendix E: EUC Delineated Desert Pavement Areas

The following figure shows the locations where the infiltration capacity of desert pavement was applied to the hydraulic model.



Appendix E
EUC Delineated Desert Pavement Areas

Numerical Groundwater Model

AECOM Environment

Prepared for:
Desert Sunlight Holdings LLC
Riverside, CA

Prepared by:
AECOM
Camarillo, CA
60139386
June 2010

NUMERICAL GROUNDWATER MODEL EVALUATION OF PROPOSED PROJECT GROUNDWATER PUMPING

Desert Sunlight Solar Farm
Chuckwalla Valley, Riverside, CA

Environment

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NUMERICAL GROUNDWATER MODEL EVOLUTION OF PROPOSED PROJECT GROUNDWATER PUMPING

Desert Sunlight Solar Farm
Chuckwalla Valley, Riverside, CA

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Project Hydrogeologist, PH

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RC&E Group Manager, PG, CEG

Arrie Bachrach
Program Manager

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List of Abbreviations

af	acre-feet
afy	acre-feet per year
Basin	Chuckwalla Valley Groundwater Basin
bgs	below ground surface
DWR	Department of Water Resources
ft/d	feet per day
ft^2/d	square feet per day
Project	Desert Sunlight Solar Farm Project
USBR	United States Bureau of Reclamation
USGS	United States Geological Survey

1.0 Introduction

The Desert Sunlight Solar Farm Project (Project) is a proposed 550-megawatt solar photovoltaic generating facility that will be constructed in the westernmost portion of the Chuckwalla Valley, in Riverside County, California (Figure 1). Project water use during operation will be minimal (0.2 acre-feet per year [afy]) over a 30-year Project life for a total of only 6 acre-feet (af). Project water use during construction of the Project is expected to total between 1,300 and 1,400 af over a 26-month construction period. The Project will obtain its water supply from groundwater. This document provides an assessment of potential impacts on adjacent water supply wells from the proposed groundwater pumping for Project construction and operation.

Two new water supply wells, one each for construction and operational water supply, are proposed for the Project (Figure 2). There are four existing water supply wells within a 2-mile radius of each of the proposed water supply wells. A review of available data and an online database search of the United State Geological Survey (USGS) National Water Information System show that wells in the vicinity of the proposed construction water supply well have been or are used for supply to the Kaiser Eagle Mountain Mine, located northwest of the Project site, and for domestic and agricultural supply and are completed (i.e., well screen interval) to depths between about 200 and 1300 feet below the ground surface (bgs). A review of available information shows that there is limited information on the well construction details in the vicinity of the proposed operational supply well.

The goal of the impact assessment provided herein is to use a previously published USGS numerical groundwater model to assess if the proposed pumping of groundwater for Project construction would impact the water supply wells adjacent to the proposed construction supply well, and how the pumping might affect groundwater basin storage. Though the proposed operational supply is insignificant (roughly 0.12 gallons per minute assuming the well would operate continuously all year long), the proposed operational pumping was evaluated to provide a comprehensive assessment of the Project water supply impacts.

2.0 Hydrogeology

The Project is located in the alluvial-filled basin of the Chuckwalla Valley. Regionally, this valley formed as a structural depression or a pull-apart basin and is composed of two broad geologic units, consolidated rocks and unconsolidated alluvium. The consolidated rocks consist of pre-Tertiary age igneous and metamorphic rocks, which form the basement complex. Water-bearing units include the Quaternary- to Pliocene-age continental deposits that are divided into the Quaternary alluvium, Pinto and Bouse Formations (DWR 2004). The Quaternary alluvium is reported to be the most important aquifer in the area (DWR 1979). In the area of the Project site, coarse-grain sand and gravel deposits are reported to overlay fine-grained lacustrine sediments, and geophysical surveys show that the depth to bedrock is over 1,000 feet bgs in the area of the Project site (GEI 2009, Figure E2-6, Cross Section C-C'). Estimates show that the coarse-grain sediments above the lacustrine deposits in the Upper Chuckwalla Valley(and in the Project site vicinity), are about 300 feet in thickness; based on available water level measurements, about 150 feet of the coarse-grained sediments are saturated above the lacustrine deposits. The coarse-grain sediments thicken dramatically to the south of the Project site, and in the area of Desert Center the saturated thickness is estimated to be over 600 feet. The saturated alluvial sediments increase in thickness eastward to over 1,000 feet in the area of Ford Dry Lake (GEI 2009, WorleyParsons 2009). The Department of Water Resources (DWR) (2004) estimates recoverable storage in the Basin at between 9,100,000 and 15,000,000 af.

Groundwater within the Basin generally flows from the west to east through the gap in the McCoy and Mule Mountains. Below the Project site, groundwater flow is generally north to south from the gap separating Pinto Valley from the Chuckwalla Valley, then southeasterly below the Project site toward Palen Dry Lake. This flow pattern is a result of a groundwater recharge mechanism from the Pinto Valley and Orocopia Valley Groundwater Basins as groundwater flows into the Basin from the west and then exits to the Palo Verde Mesa Groundwater Basin through the gap in the McCoy and Mule Mountains. Groundwater in the Basin is reportedly contained under generally unconfined conditions in the western portion of the Basin, and semi-confined and confined conditions in the central and eastern portion of the Basin, as there are near-surface lacustrine sediments that form a confining layer in these areas (AECOM 2009, WorleyParsons 2009).

Properties used to define aquifer characteristics include hydraulic conductivity, transmissivity, and storage coefficient. Hydraulic conductivity is the property of the aquifer material to transmit water, and is expressed in units of feet per day (ft/d). Transmissivity is the hydraulic conductivity multiplied by the thickness of the sediments capable of transmitting water, and is expressed in units of gallons per day per foot or feet squared per day (ft^2/d). Storage coefficient refers to the percentage of water that can be released from the aquifer material pore space. A higher storage coefficient indicates a slower progression of the cone of depression in the aquifer resulting from groundwater extraction, and a lower storage coefficient indicates a much faster progression of the cone of depression.

In general, there is limited reliable information on the aquifer characteristics within the Basin. The available data are variable and appear related to the heterogeneity of the water-bearing materials throughout the Basin, and possibly the variability in the approach to aquifer testing and analysis between investigators. In the area of Desert Center and in the Upper Chuckwalla Valley, hydraulic conductivity has been reported at between about 2 to 30 ft/d (CH2M-Hill 1996) and up to 125 ft/d (GEI 2009). This range of values is typical for a complex alluvial aquifer system that is characterized by discontinuous layers of sandy alluvial channels inter-bedded with low-permeability fine-grained silt and clay. The aquifer storage coefficient has been reported between 0.05 and 1.03 (GEI 2009).

3.0 Numerical Groundwater Model

A previously constructed numerical groundwater model developed by the USGS was selected to evaluate the impacts of the proposed Project groundwater pumping. This regional model was developed by the USGS in cooperation with the United States Bureau of Reclamation (USBR) to evaluate the potential for depletion of the Colorado River from groundwater pumping in areas outside the flood plain and sub-adjacent groundwater basins (Leake et al 2008).

3.1 USGS Groundwater Model

The regional model is a simple two-dimensional superposition model developed using MODFLOW 2000 code (Harbaugh et al 2000) for the Parker-Palo Verde-Cibola area, which includes the Basin. The model employs a single layer geometry and a large grid spacing to assess how groundwater pumping affects the flux or recharge from the Colorado River. The model assumes a uniform saturated thickness throughout the model domain and sets a constant value of storativity (0.20). In the development of the model, a range of 25 transmissivity values was evaluated by the USGS using a statistical analysis of available aquifer data along the Colorado River in consideration of data gathered from the younger and older alluvium above the Laguna Dam (above the Yuma area). In their model of potential depletion of the Colorado River, the transmissivity was from a low value (6,300 ft^2/d) to an average value of 26,000 ft^2/d. The lower value is the point where the probability is 0.05 (5 percent) that the transmissivity was equal or less than this value. The average value (26,000 ft^2/d) was selected with a probability of 0.5 (50 percent). The model grid uses a spacing of 1,320 feet throughout the domain, which includes the Chuckwalla Valley and Palo Verde Mesa as well as the Cibola area of Arizona (Figure 3). The Palo Verde Valley is not modeled, as groundwater there was assumed to be within the flood plain and directly connected to part of the Colorado River.

Several important elements of the model impact the way the model would predict the extent of drawdown from pumping. The outline of the model domain is assumed to be a no-flow boundary, and as such, there is no recharge to the model from underflow from other groundwater basins (i.e., Pinto or Orocopia) or inflow from mountain front runoff that would originate from precipitation along the margin of the groundwater basin. The way the model is constructed, in response to pumping, groundwater would be supplied solely from storage in the model domain and from changes in flux from the Colorado River. As this is not a flow model that considers groundwater head distribution and movement, the model "sees" the water table as a flat surface. When estimating pumping, the cone of depression develops as a circle since there is no consideration of groundwater flow and gradient. Under normal conditions, the cone of depression for a pumping well would be a parabola with the apex located in the down-gradient direction and fanning or opening in the up-gradient direction. As the model does not consider groundwater flow and the cone develops as a circle, this exaggerates the extent of the down- and cross-gradient influence and underestimates the up-gradient influence from proposed pumping.

As constructed, this model provides a conservative (i.e., tends to "over predict") estimate in the change in storage from proposed pumping; this is because in the model there are only limited sources of water to the pumping well, and the model excludes recharge. Estimates of drawdown during construction are less affected by the model architecture, as most of the water pumped during the short construction period would come from aquifer storage.

This existing USGS numerical groundwater model was selected to evaluate the impacts from proposed Project pumping because:

- The model includes the Project site and is of sufficient detail and complexity to adequately evaluate impacts from the modest pumping proposed for the Project.
- The model has been reviewed by the USGS and USBR, which represents adequate pre-publication peer review.
- The model provides a conservative estimate of potential change in storage from pumping as it is constructed without consideration of flow into the domain from sub-adjacent groundwater basins and precipitation from runoff.

3.2 Project Model Setup and Input Parameters

While the USGS model incorporates the Project site, several changes to the model were required for it to adequately evaluate proposed Project pumping and the influence from the pumping on adjacent water supply wells within a 2-mile radius of the Project site. For the analysis of influence, the model grid was modified by refining the grid spacing (i.e., made much smaller around the proposed pumping well). This allowed for a better assessment of the influence from Project pumping as the grid spacing around the pumping well was varied from about 30 feet around the well and gradually increased to a spacing of 1,320 feet one mile away from the pumping well.

The superposition model (Leake et al 2008) adopts a uniform grid spacing of 0.25 miles (1,320 feet). To better resolve the rapid change in drawdown near the proposed pumping well, the model grid spacing was refined as follows:

- 30 feet from the pumping well for the first 300 feet;
- 100 feet spacing further out from the well for one mile or 5,280 feet; and
- Gradual increase in spacing from 100 feet to 1,320 feet for the remainder of the model domain.

In the application of model stress periods, Project pumping was set on an annualized basis for an initial 2-year period to reflect construction water supply, followed by a 30-year period to simulate the affects of operational supply. Construction activities are expected to take place over a period of approximately 26 months, so the application in the model is slightly more conservative. For the construction period, the pumping well in the model was set at 700 afy for a 24-month period; for the Project's operational period, the pumping well was set at 0.2 afy for 30 years.

The transmissivity was not varied in the USGS model, as one value was uniformly applied over the whole model domain (Leake et al 2008). For this application, the transmissivity was revised to reflect an updated interpretation based on recent investigations of the Chuckwalla Valley Groundwater Basin. The distribution of transmissivity was based on published data from across the basin (AECOM 2009, 2010; GEI 2009; WorleyParsons, 2009) that were used to refine and remap the zones for the groundwater model (Figure 4).

- Zone 1 includes the Project site and the western portion of the Basin. The transmissivity was evaluated in the model at a range of values between 6,300 ft^2/d to 8,500 ft^2/d, which is generally within the mid-range reported for this area (GEI 2009).

- Zone 2 includes the Palo Verde Mesa Groundwater Basin east of the Chuckwalla Valley and was set at a transmissivity of 26,000 ft^2/d, which is the average value reported by the USGS (Leake et al 2008). This value was not varied in the simulations.
- The lowest transmissivity zone (Zone 3 = 1,000 ft^2/d) was applied to the central zone within the basin, Palen Dry Lake and the area around Ford Dry Lake. This value was not varied in the simulations.
- Zone 4, at a transmissivity of 6,300 ft^2/d, was established at the very easternmost portion of the Basin. This value was not varied in the simulations.

In most respects the distribution of transmissivity represents a simplification of a heterogeneous environment to the analysis of water supply impacts from the Project, as it presumes through-going uniformity of aquifer characteristics that are not documented in the hydrostratigraphy for the Basin. The range in transmissivity values in the Project area provided for a sensitivity analysis, as the model was run using a range of different values that have been reported for the Project area. As hydraulic conductivity is the ratio of transmissivity to aquifer thickness (T/b), the values selected fall within the range of hydraulic conductivity estimates reported by CH2M-Hill for the Upper Chuckwalla Valley (CH2M-Hill 1996).

The model depth or assumption of saturated thickness was varied between 150 feet and 500 feet to reflect varied interpretations of the aquifer thickness in the area of the Project site. A value of 500 feet was selected for the model because it is the value used in the USGS model and is close to the saturated thickness (i.e., the interval from top of the water table to the base of the well screen interval) of 450 feet for the nearby Kaiser Ventures Chuckwalla Number 4 Well (GEI 2009, Figure E2-6). The Kaiser Ventures well is about a mile due north of the proposed construction water supply well. In contrast, a value of 150 feet was also used in some of the model scenarios. The value follows the GEI (2009) interpretation that the saturated thickness of the coarse-grain alluvial deposits in the area of the Project site is about 150 feet (GEI 2009, Figure E2-6). This is also the value used in their modeling of the proposed Eagle Crest Project.

Lastly, the aquifer storage coefficient was varied from 0.05 to 0.2. The variation of values corresponds to the lowest value reported by GEI (2009) (0.05) for the Desert Center area and the value used in their groundwater model for the Eagle Crest Project. A value of 0.2 was modeled to reflect the interpretation that the aquifer in the western portion of the Basin is unconfined (GEI 2009; AECOM 2009; WorleyParsons 2009). Further, this value is within the lower to middle range of values reported for the area around the Project site (GEI 2009). The variation of storage coefficient was applied to Zone 1 only, which included the Project site. The remainder of the model domain was left at a storage coefficient of 0.2 that was used by the USGS (Table 1).

In summary, several of the key model variables were changed from what was used in the USGS model to reflect a range of interpretations and available data for the western portion of the Chuckwalla Valley. The variation of these input variables in the model provides a measure of uncertainty analyses to better evaluate the potential effects of drawdown around the pumping and surrounding wells. In general, the input values selected tended to produce a conservative estimate of impacts from proposed pumping.

4.0 Numerical Groundwater Model Results

The results of the modeling are provided on Table 1 and shown on Figures 5 through Figure 9 for the proposed Project construction period and the range of transmissivity values of 6,300 ft^2/d and 8,500 ft^2/d, storage coefficients (0.05 to 0.2) and aquifer thickness (150 feet to 500 feet). Based on the model scenarios, the maximum drawdown predicted for the construction well is about 18 feet and for the operational supply well about 0.3 foot (Table 1). The maximum drawdown of the construction well is approximately 3 percent of the assumed model layer thickness of 500 feet. The modeling result of 3 percent maximum drawdown demonstrates that it is appropriate to apply a superposition model, because a superposition model can be applied if the basin-wide drawdown of the unconfined aquifer is 10 percent or less of saturated thickness.

As would be expected, the drawdown at the well and the radius of influence increase with lower transmissivity and a lower storage coefficient and decrease with higher transmissivity and higher storage coefficient. The lower aquifer thickness (150 feet) tended to produce smaller values of drawdown at the pumping well and correspondingly a smaller cone-of-depression defined to the one-foot contour (Figure 7, 8 and 9). In general, there were some difference in the results as a function of all the model variables, but the most sensitive were the aquifer thickness and transmissivity.

Under any of the scenarios and range of input variables, the model predicts that no well within a 2-mile radius of the proposed construction well will be impacted by a drawdown of 5 feet or more during the construction period, and for most of the simulations only one well (4S/15E-31C1) is predicted to be within the one-foot drawdown contour. The exception is the model scenario that employs the lowest transmissivity and storage coefficient and predicts that four wells (4S/15E-31C1, 4S/16E-19M1, 4S/16E-19N1 and CW#4) are within the one-foot drawdown contour, though none within the 5-foot drawdown contour (Figure 7). This scenario represents a combination of the lowest estimated values and as such, is not anticipated.

These results indicated that the Project will not significantly impact off-site water supply wells during the construction period. The operational period was not illustrated as the drawdown at the pumping well is less than 1 foot after 30 years.

The storage change was also calculated using the model flow budget. As can be seen on Table 1, the largest net change occurs at the end of construction, and the change represents about 1,400 af (Table 1). Assuming a conservative total recoverable storage of 9,100,000 af in the Basin (DWR, 2004), the impact of basin storage is insignificant (0.00015 percent) even for the largest storage change at the end of construction.

Based on the results of these numerical groundwater simulations, the proposed Project pumping will not significantly impact adjacent water supply wells or the groundwater basin storage.

5.0 References

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Table

TABLE-1
RESULTS OF PREDICTIVE SIMULATIONS
NUMERICAL GROUNDWATER MODEL
DESERT SUNLIGHT SOLAR FARM
DESERT SUNLIGT HOLDINGS, LLC
CHUCKWALLA VALLEY GROUNDWATER BASIN
RIVERSIDE COUNTY, CALIFORNIA

Model Runs[1]	RESULTS SHOWN (FIGURE)	SATURATED THICKNESS	Zone 1[2]		Zone 2[3]		Zone 3[4]		Zone 4[5]		Period of interest	Maximum Predicted Drawdown at the Pumping Well		Change in Storage (af)
		feet	T (ft^2/d)	S (--)	T (ft^2/d)	S (--)	T (ft^2/d)	S (--)	T (ft^2/d)	S (--)		Construction Supply[6]	Operational Supply[7]	
FS_T6300	5	500	6,300	0.2	26,000	0.2	1,000	0.2	6,300	0.2	2013	15.46	--	1,401.49
											2043	--	0.13	1,407.67
FS_T8500	6	500	8,500	0.2	26,000	0.2	1,000	0.2	6,300	0.2	2013	11.89	--	1,401.48
											2043	--	0.12	1,407.67
FS_T6300, ADD-1	7	500	6,300	0.05	26,000	0.2	1,000	0.2	6,300	0.2	2013	17.80	--	1,401.48
											2043	--	0.297	1,407.67
FS_T8500, ADD-2	--	500	8,500	0.05	26,000	0.2	1,000	0.2	6,300	0.2	2013	13.18	--	1,401.48
											2043	--	0.276	1,407.67
FS_T6300, ADD-3	8	150	6,300	0.2	26,000	0.2	1,000	0.2	6,300	0.2	2013	6.78	--	1,401.48
											2043	--	0.055	1,407.66
FS_T8500, ADD-4	--	150	8,500	0.2	26,000	0.2	1,000	0.2	6,300	0.2	2013	5.24	--	1,401.47
											2043	--	0.048	1,407.65
FS_T6300, ADD-5	9	150	6,300	0.05	26,000	0.2	1,000	0.2	6,300	0.2	2013	6.64	--	1,401.47
											2043	--	0.124	1,407.65
FS_T8500, ADD-6	--	150	8,500	0.05	26,000	0.2	1,000	0.2	6,300	0.2	2013	6.46	--	1,401.46
											2043	--	0.123	1,407.64

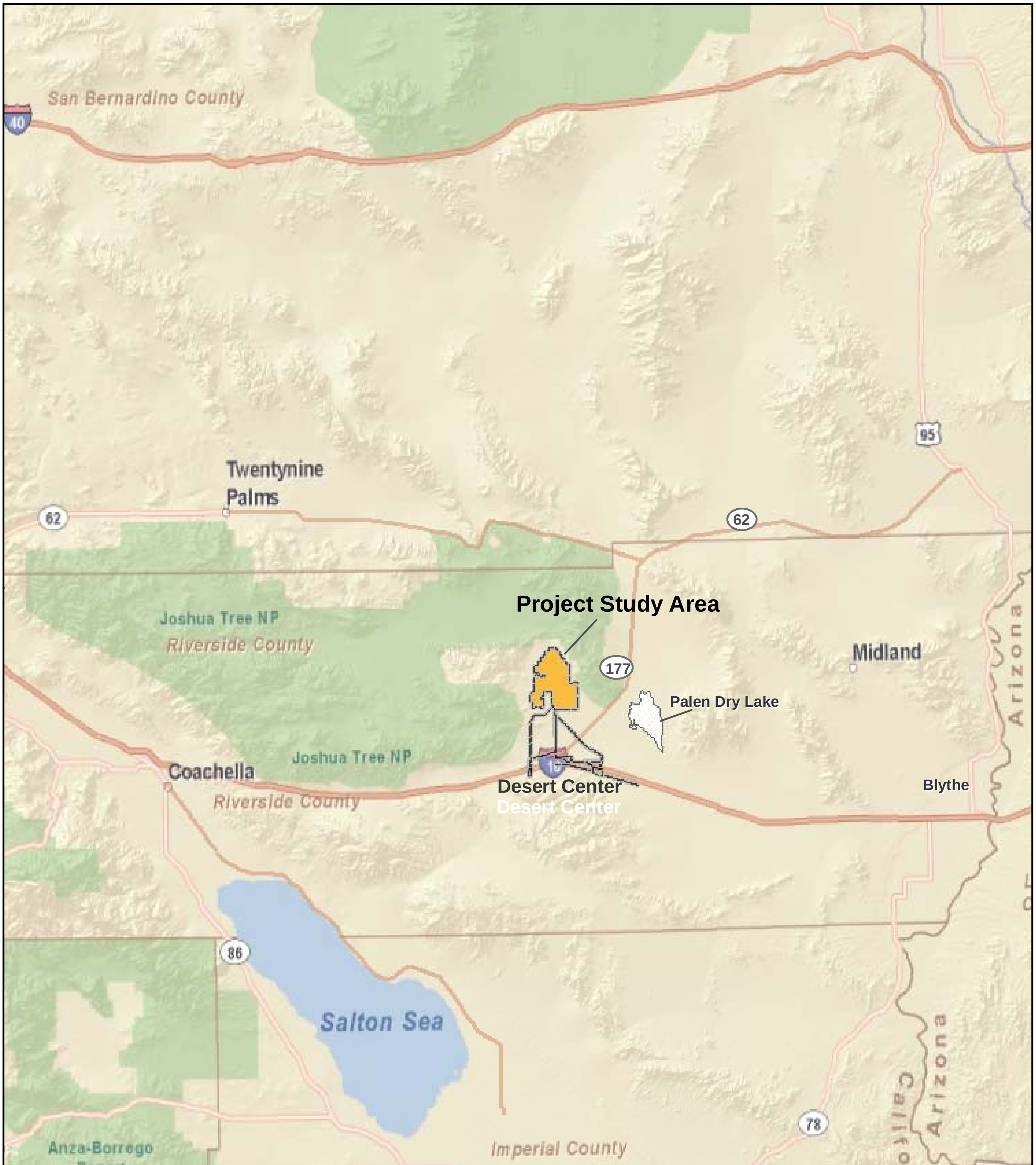
Notes

1 FS_T6300 & FS_T8500 are the "Project Only" simulations
2 Zone 1 - Western Portion of the Chuckwalla Valley Groundwater Basin (Project Area) (See Figure 4)
3 Zone 2 - Palo Verde Mesa Groundwater Basin (See Figure 4)
4 Zone 3 - Central portion of the Chuckwalla Valley Groundwater Basin (See Figure 4)
5 Zone 4 - Easternmost portion of the Chuckwalla Valley Groundwater Basin (See Figure 4)
6 Construction supply modeled at 700 acre-feet per year for 2011 and 2012 for a total supply of 1,400 acre-feet in 24 months.
7 Operational supply modeled at 0.2 acre-feet per year from years 2013 to 2042. Total supply of 6 acre-feet over 30 years.

Definitions

T Transmissivity in feet squared per day
S Storage coefficient (unitless)
af Acre-feet (one acre-foot = 325,829 gallons)

Figures



1:1,000,000

0 20 40 Miles

Desert Sunlight Solar Farm Project

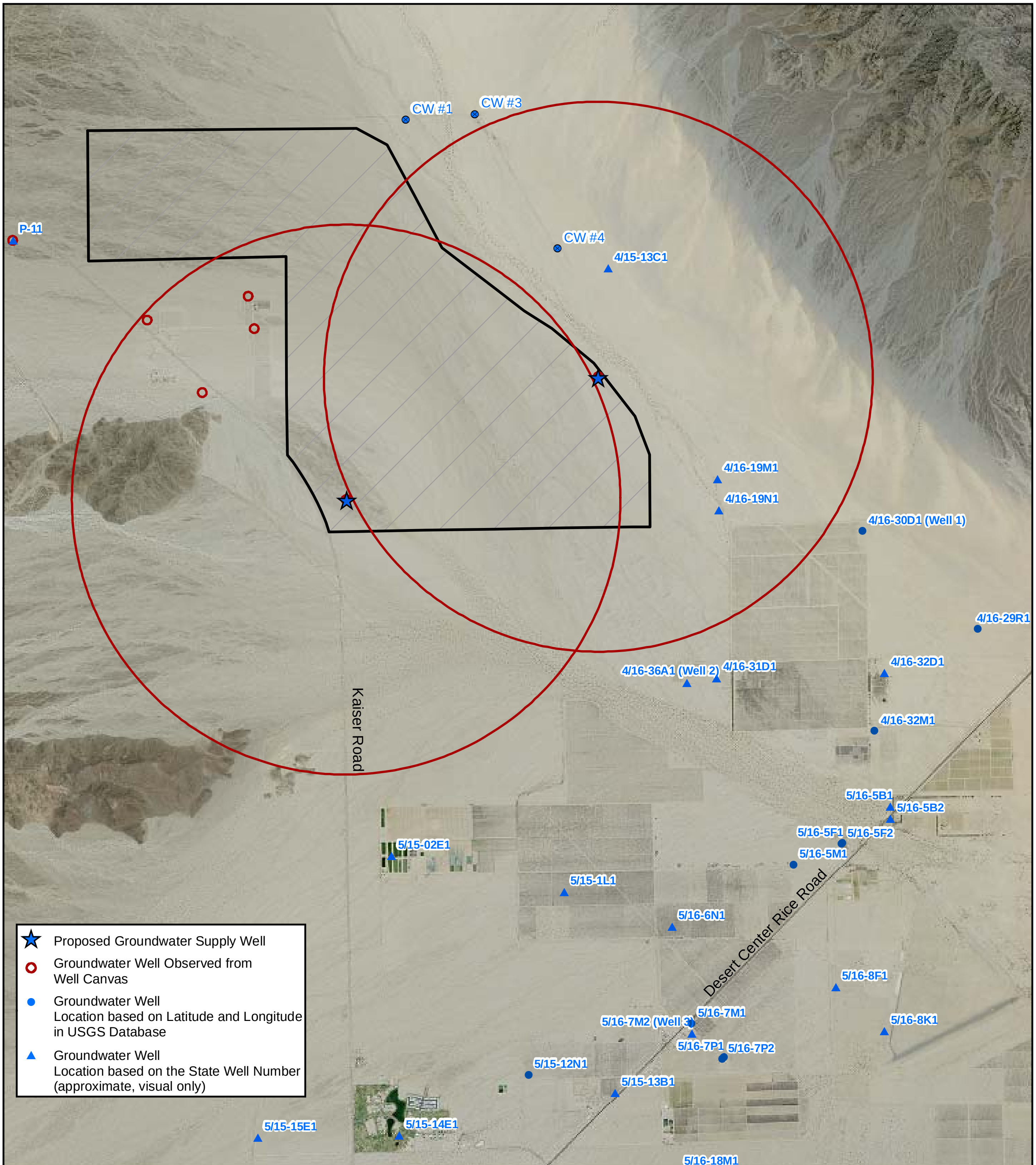
Figure 1
Project Vicinity Map

First Solar.

AECOM

Project: 60139386.014
Date: March 2010

J:\GIS\Projects\12414-First_Solar\011-Desert Sunlight\mxd\Feb10Bnd\Figure 1_Project Vicinity Map.mxd



Map Location

Legend

Model Predicted Drawdown (feet)

Approximate 2-Mile Radius Around Proposed Wells

Proposed Solar Farm Site

1 in = 1 miles

0 1 2 Miles

Desert Sunlight Solar Farm Project

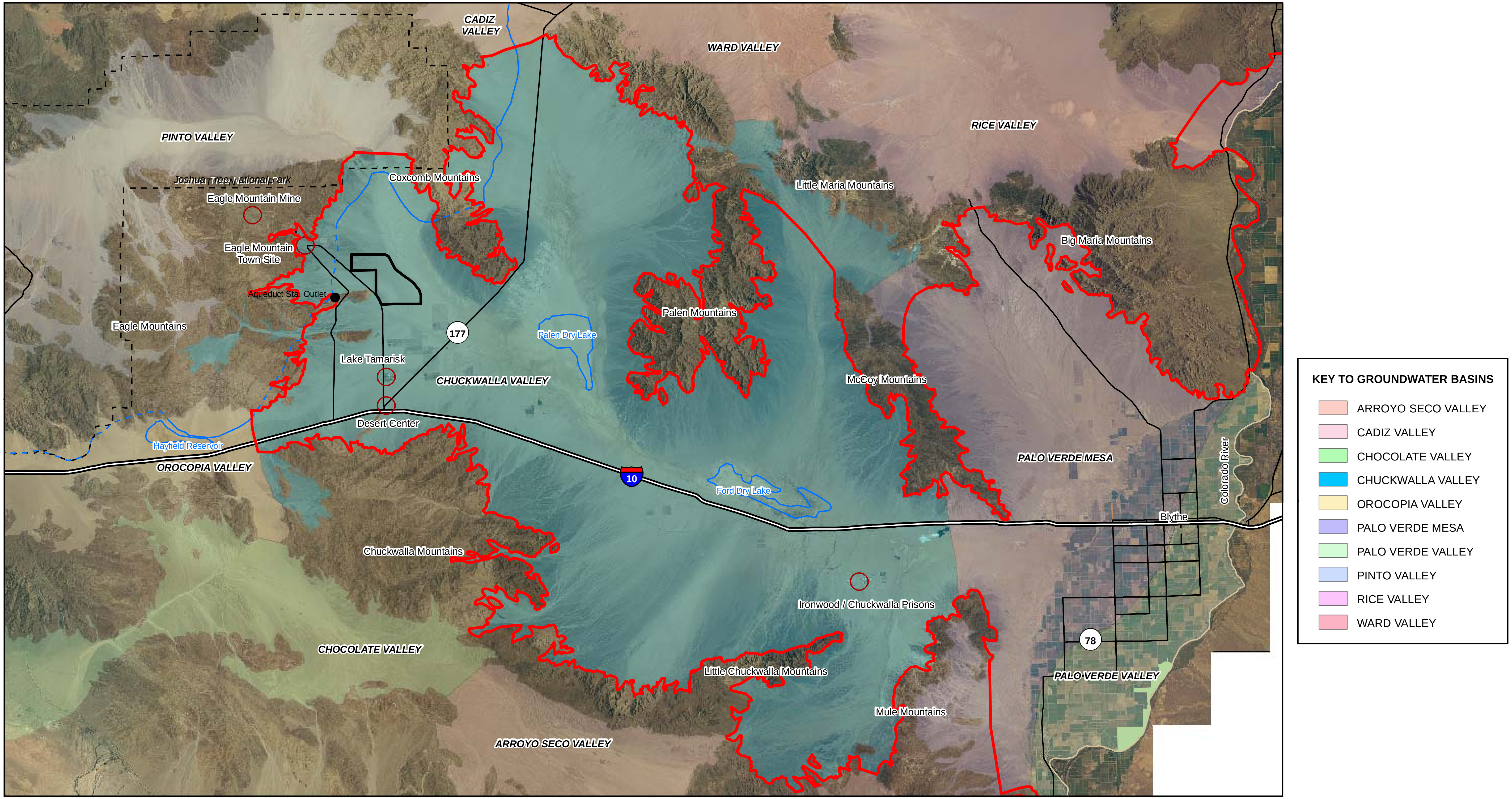
Figure 2
Site Map Showing Proposed Wells

First Solar.

AECOM

Project: 60139386.014
Date: June 2010

J:\GIS\Projects\12414-First_Solar\011-Desert Sunlight\mxd\Feb10Bnd\Figure 2_Study Area and Site.mxd



Map Location

Legend

- USGS (2008) Parker-Palo Verde-Cibola Model Domain
- Proposed Solar Farm Site
- Colorado River Aqueduct
- Colorado River Aqueduct (Dash showing underground interval)
- Freeway
- Highway / Major Road
- Geographic/Cultural Area of Interest

Data Sources:
Air Photo, California Spatial Information Library, NAIP, 2005 Riverside County

Water Basins, Department of Water Resources Website groundwater basin map file B118v3NAD27UTM10.zip

N W E S

0 6 12 Miles
1 in = 6 miles

Desert Sunlight Solar Farm Project

Figure 3
USGS (2008) Model Area

First Solar.

AECOM

Project: 60139386.014
Date: June 2010

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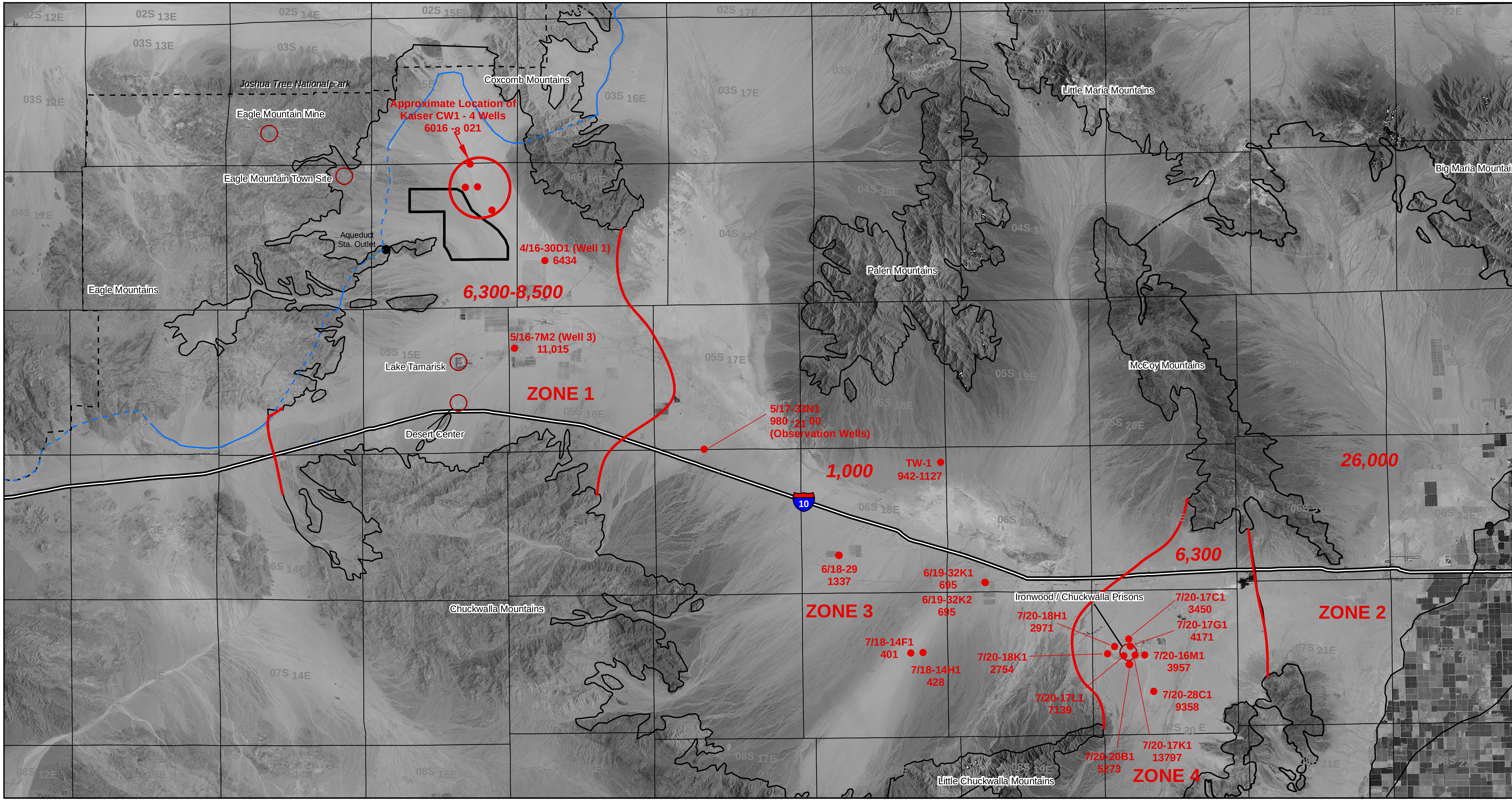

Map Location

Legend

- Project Study Area
- Colorado River Aqueduct
- Colorado River Aqueduct (Dash showing underground interval)
- Chuckwalla Valley and Palo Verde Mesa Groundwater Basin Boundaries
- Freeway
- Geographic/Cultural Area of Interest
- 1337 — Groundwater Well with Reported Transmissivity Data (feet squared per day (ft²/d)
- 6,300 — Transmissivity (ft²/d
- Boundary between Transmissivity Zones

Data Sources:
Air Photo, California Spatial Information Library, NAIP, 2005 Riverside County

Water Basins, Department of Water Resources Website groundwater basin map file B118v3NAD27UTM10.zip

AECOM 2009, 2010
GEI 2009
WorleyParsons 2009

N W E S

0 4 8 Miles
1 in = 4 miles

Desert Sunlight Solar Farm Project

Figure 4
Transmissivity Distribution

First Solar.

AECOM

Project: 60139386.014
Date: June 2010

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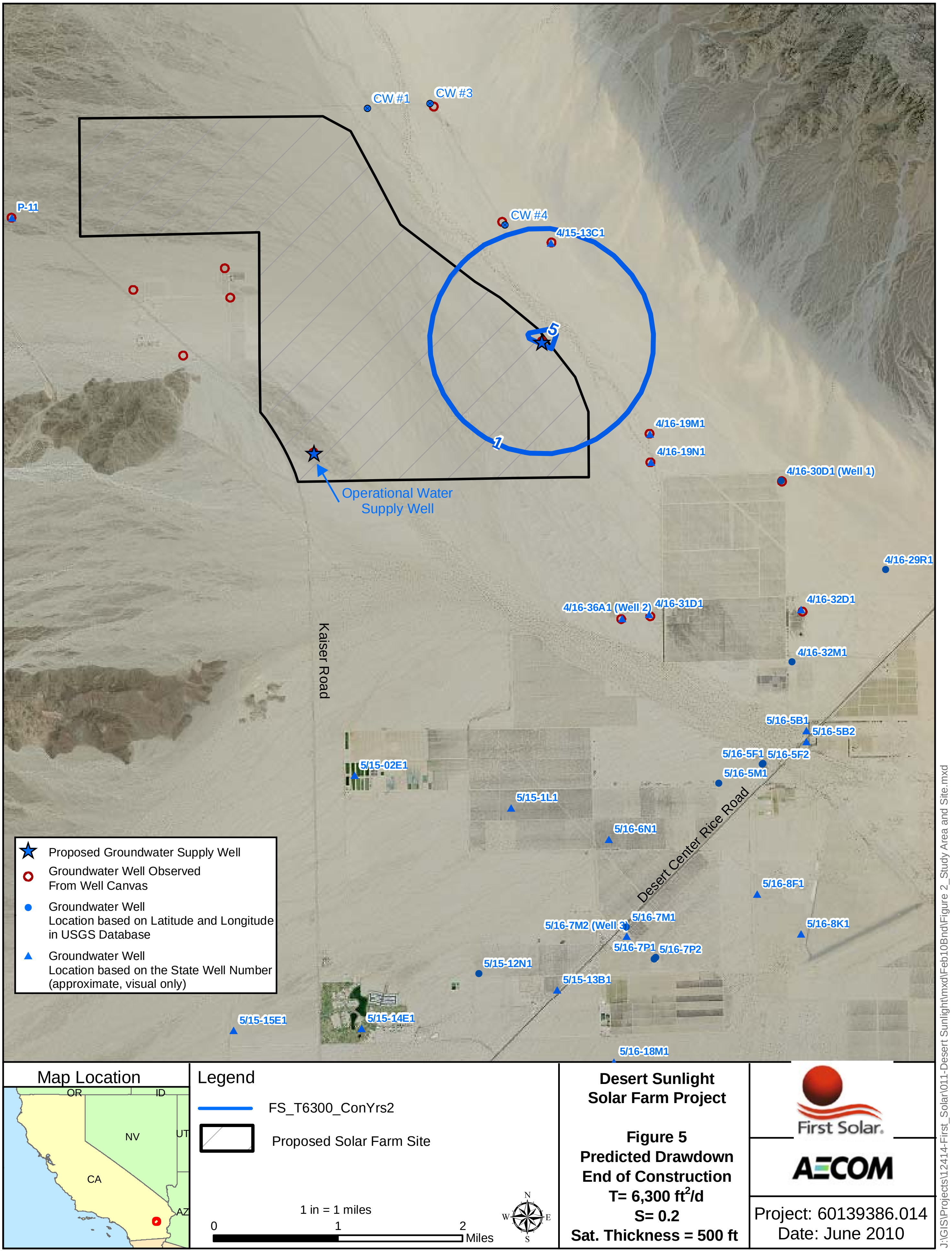
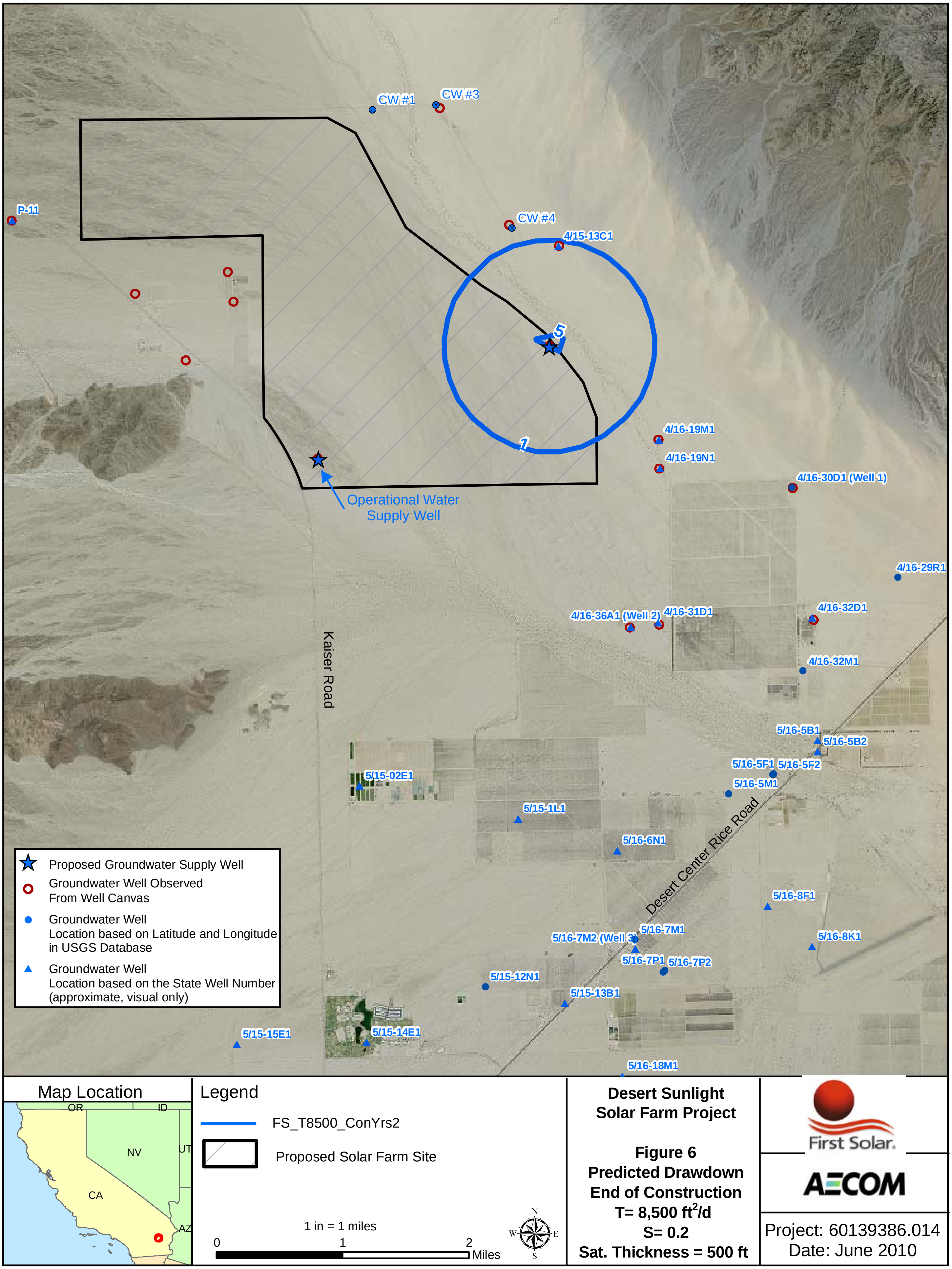

CW #1
CW #3
P-11
CW #4
4/15-13C1
5
1
4/16-19M1
4/16-19N1
4/16-30D1 (Well 1)
Operational Water
Supply Well
4/16-29R1
4/16-36A1 (Well 2)
4/16-31D1
4/16-32D1
4/16-32M1
Kaiser Road
5/16-5B1
5/16-5B2
5/16-5F1
5/16-5F2
5/16-5M1
5/15-02E1
5/15-1L1
5/16-6N1
Desert Center Rice Road
5/16-8F1
5/16-7M1
5/16-7M2 (Well 3)
5/16-8K1
5/16-7P1
5/16-7P2
5/15-12N1
5/15-13B1
5/15-15E1
5/15-14E1
5/16-18M1
Proposed Groundwater Supply Well
Groundwater Well Observed
From Well Canvas
Groundwater Well
Location based on Latitude and Longitude
in USGS Database
Groundwater Well
Location based on the State Well Number
(approximate, visual only)
Map Location
OR
ID
NV
UT
CA
AZ
Legend
FS_T6300_ConYrs2
Proposed Solar Farm Site
1 in = 1 miles
0
1
2
Miles
N
W
E
S
Desert Sunlight
Solar Farm Project
Figure 5
Predicted Drawdown
End of Construction
T= 6,300 ft2/d
S= 0.2
Sat. Thickness = 500 ft
First Solar.
AECOM
Project: 60139386.014
Date: June 2010
J:\GIS\Projects\12414-First_Solar\011-Desert Sunlight\mxd\Feb10Bnd\Figure 2_Study Area and Site.mxd



Desert Sunlight Solar Farm Project

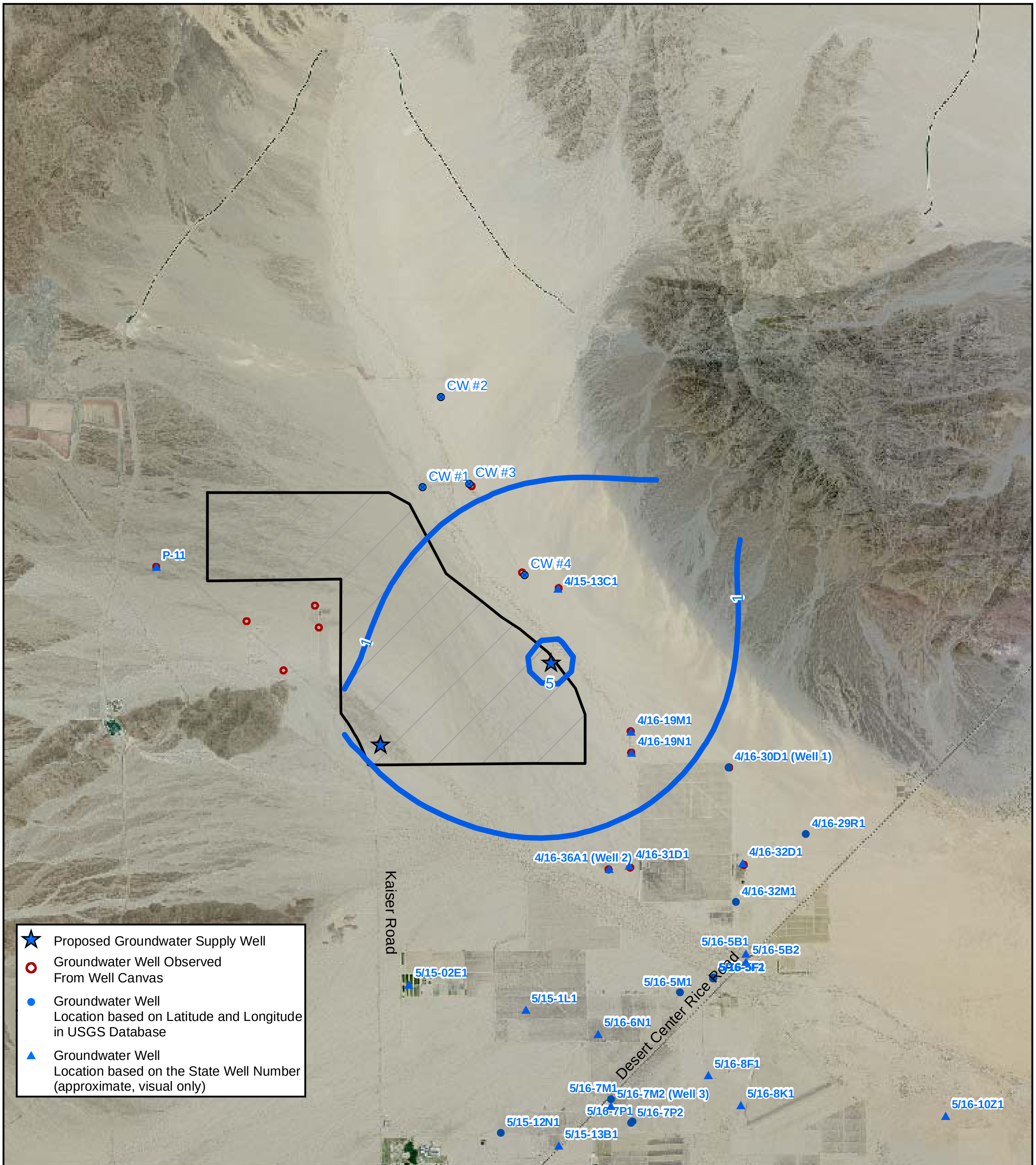
Figure 6
Predicted Drawdown
End of Construction
T= 8,500 ft^2/d
S= 0.2
Sat. Thickness = 500 ft

First Solar.

AECOM

Project: 60139386.014
Date: June 2010

J:\GIS\Projects\12414-First_Solar\011-Desert Sunlight\mxd\Feb10Bnd\Figure 2_Study Area and Site.mxd



Map Location

Legend

FS_T6300_2w_add1_ConYrs2

Proposed Solar Farm Site

1 in = 1 miles

0 1 2 Miles

Desert Sunlight Solar Farm Project

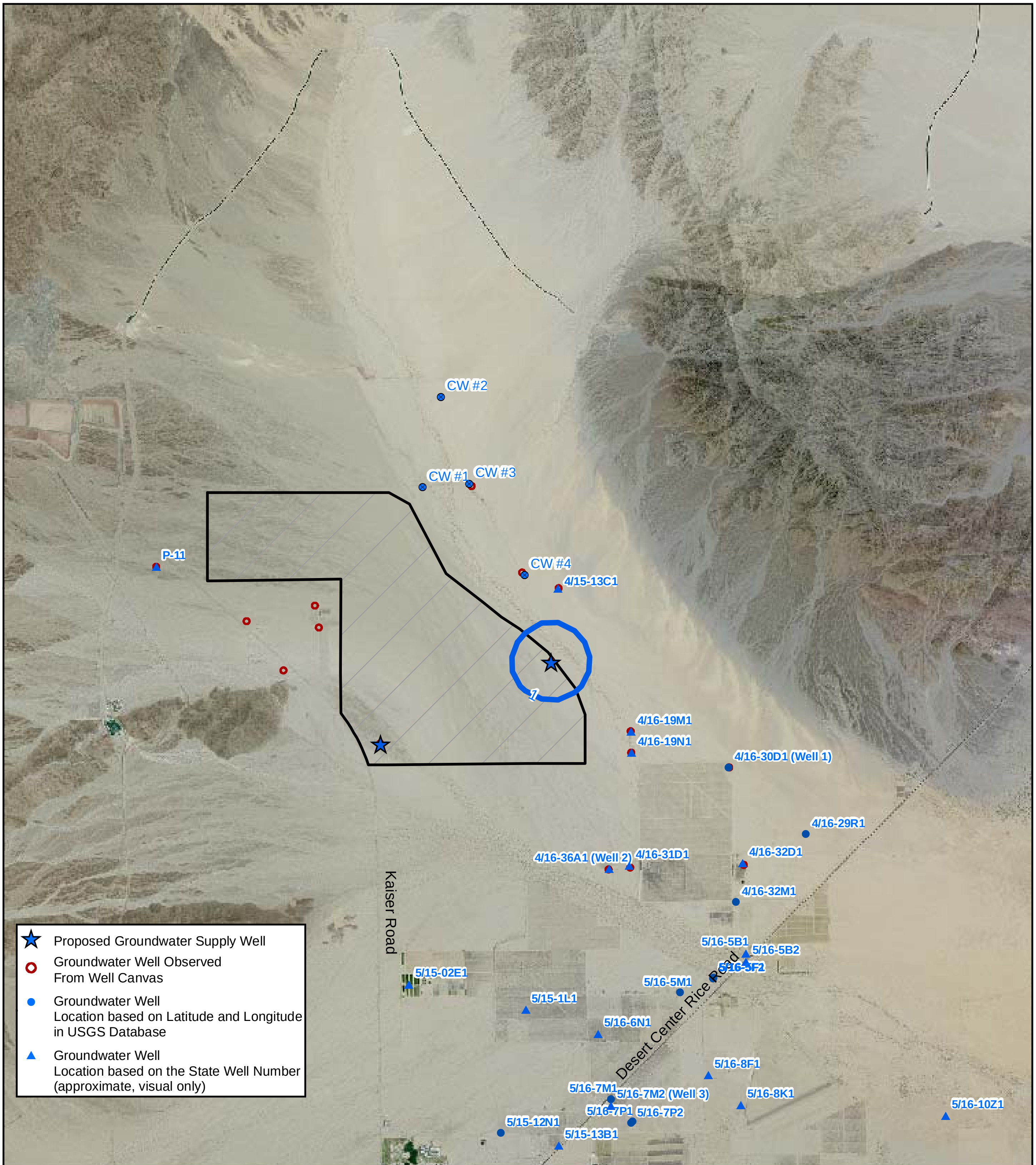
Figure 7
Predicted Drawdown
End of Construction
T= 6,300 ft^2/d
S= 0.05
Sat. Thickness = 500 ft

First Solar.

AECOM

Project: 60139386.014
Date: June 2010

J:\GIS\Projects\12414-First_Solar\011-Desert Sunlight\mxd\Feb10Bnd\Figure 2_Study Area and Site.mxd



Map Location

Legend

FS_T6300_2w_add3_ConYrs2

Proposed Solar Farm Site

1 in = 1 miles

0 1 2 Miles

N W E S

Desert Sunlight Solar Farm Project

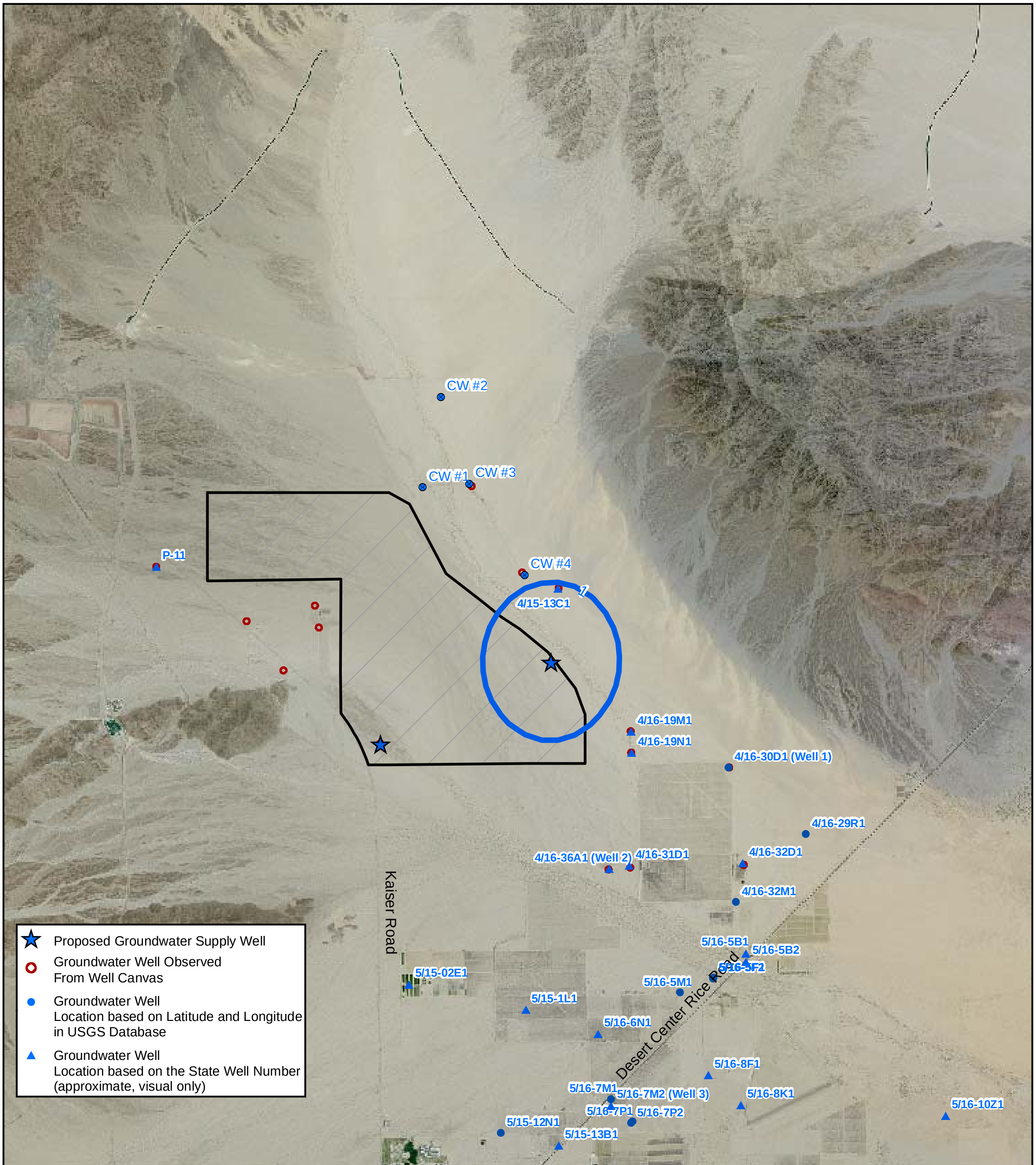
Figure 8
Predicted Drawdown
End of Construction
T= 6,300 ft^2/d
S= 0.2
Sat. Thick = 150 ft

First Solar.

AECOM

Project: 60139386.014
Date: June 2010

J:\GIS\Projects\12414-First_Solar\011-Desert Sunlight\mxd\Feb10Bnd\Figure 2_Study Area and Site.mxd



Map Location

Legend

FS_T6300_2w_add5_ConYrs2

Proposed Solar Farm Site

1 in = 1 miles

0 1 2 Miles

Desert Sunlight Solar Farm Project

Figure 9
Predicted Drawdown
End of Construction
T= 6,300 ft^2/d
S= 0.05
Sat. Thick = 150 ft

First Solar.

AECOM

Project: 60139386.014
Date: June 2010

J:\GIS\Projects\12414-First_Solar\011-Desert Sunlight\mxd\Feb10Bnd\Figure 2_Study Area and Site.mxd