Appendix J.3

Drainage Master Plan
Q3, 2021

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Technical Appendices

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Travertine Development

DRAINAGE MASTER PLAN

City of La Quinta and County of Riverside, California

Regional Hydrology, Hydraulics, and Proposed Project Flood Risk Mitigation

June 10, 2021

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Electronic Technical Appendix

Provided electronically

HEC-HMS Models

Excel spreadsheet – slope analysis

Excel spreadsheet – determination of precipitation losses

Excel spreadsheet – determination of debris yield A-T factors

Rating Table Hydraulics

ArcGIS files

FLO-2D PRO input and output files

Selected References

1 INTRODUCTION

1.1 Project overview

The proposed Travertine Specific Plan Amendment is an 855.4-acre mixed-use community in the City of La Quinta and parts of unincorporated Riverside County, California. The Specific Plan Area is tucked into one of the eastern piedmonts along the base of the Santa Rosa Mountains, above the flood impoundment zone of West Dike System Dike No. 4 (BOR, 1947; Bechtel, 1991), and adjacent to the Guadalupe Creek Diversion Dikes (CVCWD, 1968; Bechtel, 1991) to the north. Dike No. 4 and the Guadalupe Creek Diversion Dikes were constructed circa 1967.

The purpose of the Drainage Master Plan is to determine the project-related impacts to existing hydrology, floodplains, and drainage/flood control features, and identify appropriate flood control and local drainage improvements necessary for the planned development. The Drainage Master Plan addresses both regional and local impacts, flood hazard mitigation requirements, and design constraints/features. The Drainage Master Plan is based on the requirements of the Coachella Valley Water District (CVWD), County of Riverside, and the City of La Quinta. Travertine is shown in Figure 1-1 (location map), Figure 1-2 (regional vicinity map), Figure 1-3 (local vicinity map), and Figure 1-4 (concept grading plan).

1.2 Project description

The Travertine Specific Plan Amendment encompasses approximately 855 acres in the City of La Quinta at the northeastern base of the Santa Rosa Mountains in the Coachella Valley. The property is generally bounded by Avenue 60 to the north; Avenue 64 and Bureau of Land Management land to the south; Madison Street to the east; and Jefferson Street to the west. The property is located roughly one mile south of PGA West and Lake Cahuilla, situated in Section 33, Township 6 south, Range 7 east; and in Sections 3, 4 and 5 of Township 7 south, Range 7 east, San Bernardino Base and Meridian, County of Riverside, California.

The Travertine Specific Plan proposes a variety of complementary land uses, including up to 1,200 residential units, proposed 100 villa resort, and associated commercial and recreational facilities. Recreational opportunities include a 4-hole golf practice facility, open-space areas, and private recreational facilities provided in the individual residential developments.

1.3 Goals and objectives

The purpose of this document is to provide a detailed watershed assessment, including regional and local hydrology, flood hazard analysis, hydraulics, and sedimentation in order to develop a drainage master plan for the Travertine development. The overall goal of this study is to provide the appropriate level of flood protection for the public, non-CVWD storm water facilities, and impacted CVWD storm water facilities that are consistent with the requirements and guidelines instituted by the City of La Quinta, CVWD, and the U.S. Bureau of Reclamation (Dike No. 4).

The primary objectives of this study include the following:

- Develop baseline ("without" project) and project conditions hydrology to establish peak flow rates and flood volumes for use in the conceptual design of combined onsite/offsite flood conveyances and temporary impoundments along the outer project edge of the planned development (Travertine)
- Develop project conditions local hydrology for use in the conceptual design of on-site storm water facilities

- Identify and propose mitigation for any potentially significant development-related adverse flood hazard impacts, including Dike No. 4 and the Guadalupe Creek Diversion Dikes
- Identify hydraulic, sedimentation, and erosion issues/design constraints associated with the major flood conveyances, which extend through or around the proposed development, including the Guadalupe Creek Diversion Dikes along the north edge of the Travertine boundary
- Formulate the conceptual design of regional and local storm water facilities

The drainage master plan includes the preparation of detailed technical studies for the regional watershed and local onsite drainage areas leading to the identification of flood hazards, which may impact site development. The technical studies included:

- Geomorphic assessment of the regional watershed, project site, and surrounding vicinity
- Regional hydrology, hydraulics, and sedimentation analysis for the Dike No. 4 watershed
- Dike No. 4 flood routing/impoundment and impact analysis
- Guadalupe Creek Diversion Dikes flood routing and impact analysis
- Local hydrology analysis and preliminary pipe sizing

The intended use of the drainage master plan is to (1) identify flood hazards within and in the vicinity of the Travertine Specific Planning Area, (2) develop a regional approach to mitigate the flood hazards, (3) identify local drainage facility requirements, and (4) evaluate development-related impacts to existing facilities, including Dike No. 4, the Guadalupe Creek Diversion Dikes, and the CVWD deep aquifer recharge basins.

1.4 CVWD standards and policies

The standards and policies described in the CVWD Development Design Manual (last revised February 3, 2020) were applied herein. The relevant sections considered are as follows:

- Section 8 – Design Criteria Stormwater Facilities, last revised January 2020
- Guideline K-3 – Scour Calculation Guidance, last revised December 11, 2019
- Guideline K-6 – Framework for Hydrologic Modeling, last revised January 14, 2020
- Guideline K-7 – Ordinance 1234.1, last revised February 20, 2020

The standard requirements for hydrologic studies applicable to the Travertine Development are as follows:

- NOAA Atlas 14 (NWS, 2014) precipitation depth spatial datasets will be used in the computation of the 1-percent annual chance (100-year) storm and flood
- CVWD recommends constructing a synthetic storm from 100-year precipitation depths using selected durations ranging from 5 minutes to 6 hours
- The Standard Project Storm (SPS) is based on the September 24, 1939 Indio Storm; and the Standard Project Flood (SPF) is determined from hydrologic analysis based on the SPS
- For the analysis of the SPS/SPF and 1-percent annual chance storm event, the precipitation depth-area curve developed for the September 24, 1939 Indio Storm (Plate 4; USACE, 1980), will be applied to watersheds with areas greater than 10 square miles
- CVWD recommends HEC-HMS for rainfall-runoff modeling
- CVWD recommends the Whitewater River dimensionless S-graph (USACE, 1980; RCFCWCD, 1978) for determining basin-specific unit hydrographs
- General advice on hydrologic loss rates is provided in Appendix K-5 of the Development Design Manual

Ordinance 1234.1 states that levees shall be designed with a minimum of 4 feet of freeboard from the levee crest elevation to the 1-percent annual chance maximum flood stage and a minimum of one foot of freeboard as measured from the levee crest elevation to the Standard Project Flood maximum flood stage.

1.5 Document format

The document sections are set out to complete the primary objectives of this drainage master plan and include the detailed discussion and technical analysis used for the study. Methodologies, technical approaches, assumptions, design parameters, and result summaries used for the development of the analyses, and identification of flood protection requirements as well as mitigation measures are included. The detailed technical calculations, including spreadsheets and computer input/output files, are provided electronically.

Submittal and Approval Process

The report is being submitted in three phases to better facilitate the review and approval of the document. Each succeeding phase will expand on the previous submittal. The three phases are as follows:

1. Regional hydrology, baseline conditions
2. Regional and local hydrology, Project conditions
3. Final report including impact analysis and mitigation

This document is the complete submittal of Phases 1 through 3.

Figure 1-1. Location map



Figure 1-2. Regional vicinity map

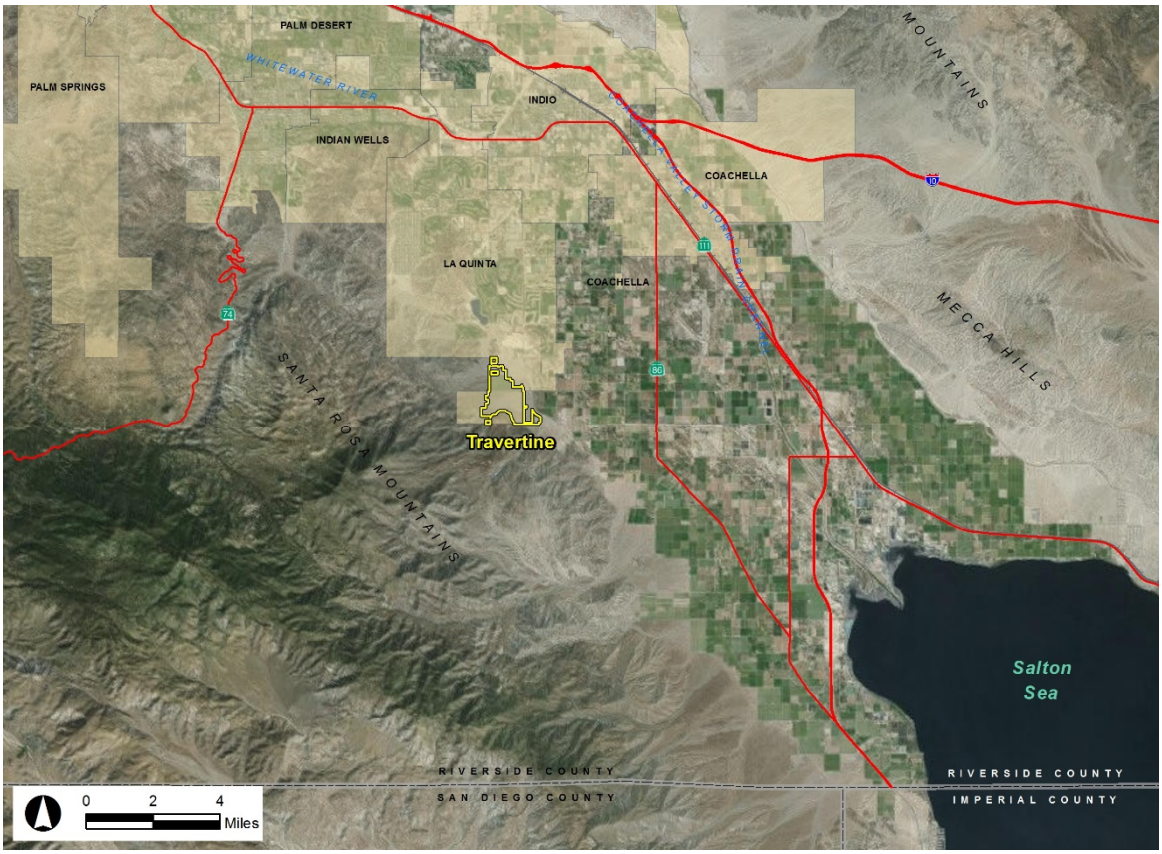


Figure 1-3. Local vicinity map

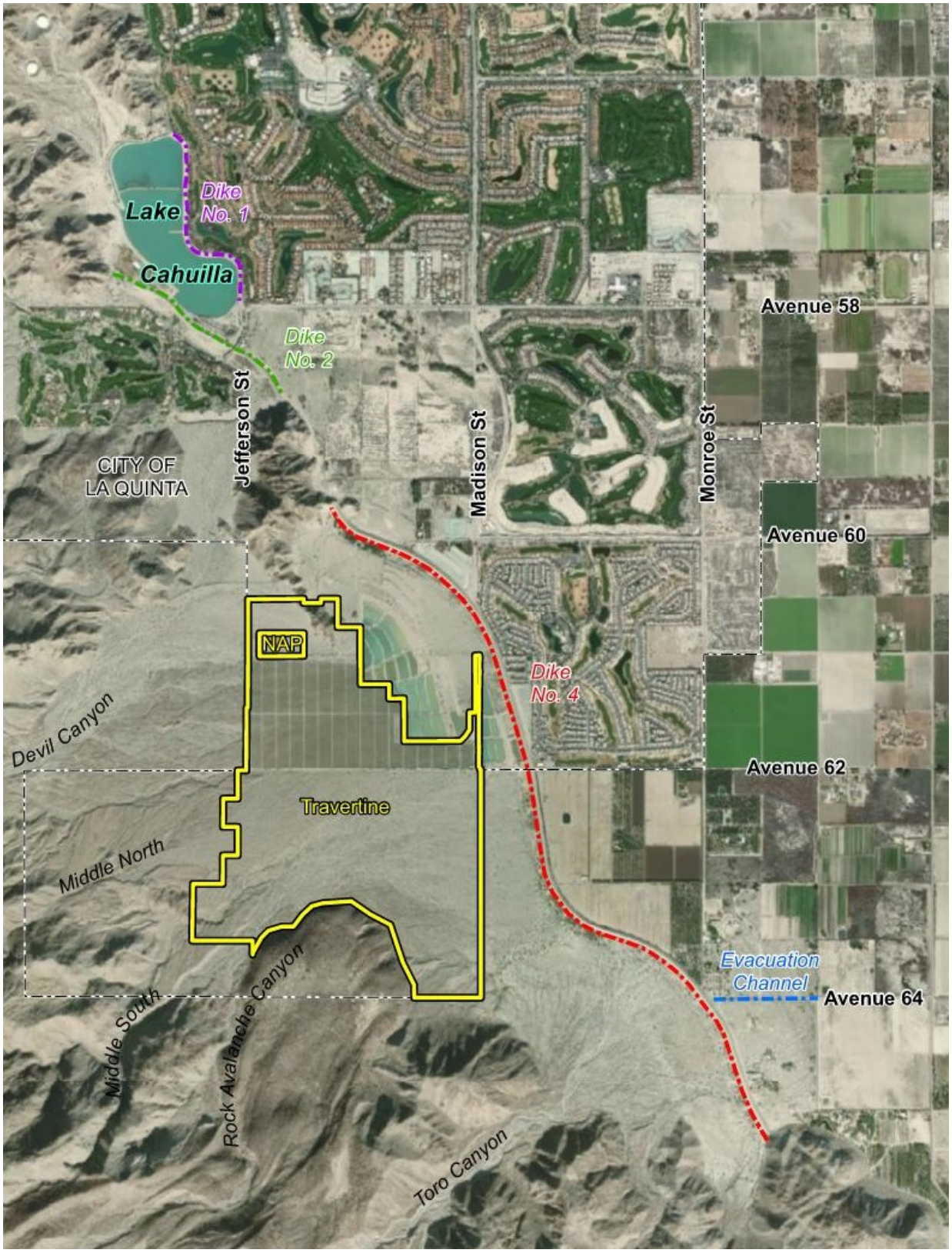
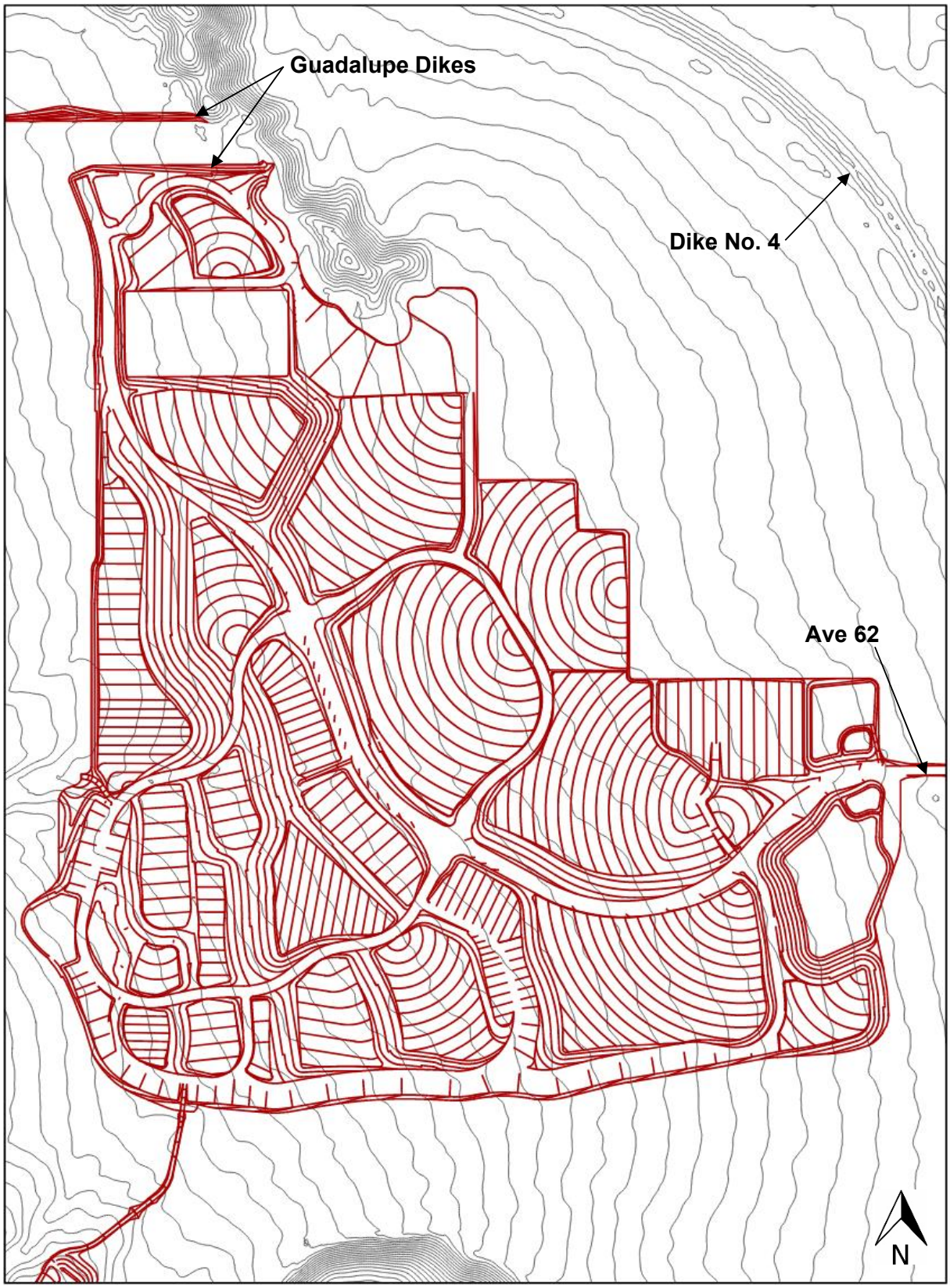


Figure 1-4. Travertine grading concept plan



2 WEST DIKE SYSTEM

2.1 Dike No. 4

Dike No. 4 is part of West Dike System, which also includes Dike No. 1 (Lake Cahuilla) and Dike No. 2 immediately to the north. This facility was originally constructed in the late 1960's by the Bureau of Reclamation (BOR). This facility is currently owned by the BOR and maintained by CVWD.

There have been a number of studies in the past, which have reevaluated the hydrology and/or function of the outlet works and interconnectivity of the West Dike System. Most recently, a Federal Emergency Management Agency (FEMA) Letter of Map Revision (LOMR) application for the certification of Dike No. 4 was prepared and last revised in 2005 by Pacific Advanced Civil Engineering (PACE) for CVWD, the FEMA community responsible for this facility, on behalf of the Enclave at La Quinta (formerly known as the Trilogy development).

As part of this FEMA LOMR application, the following documents were prepared:

- PACE, 2005, *FEMA Application for Physical Map Revision: The Enclave at La Quinta – Dike No. 4*, prepared for the Enclave at La Quinta, LLC, April
- Sladden Engineering, 2001, *Geotechnical Investigation of CVWD Dike No. 4 Flood Control Levee*, October

The analytical methods used were as follows:

- Flood hydrographs were developed based on the synthetic unit hydrograph method prescribed in the Riverside County Hydrology Manual (RCFCWCD, 1978) and implemented using the HEC-1 v4.0 (USACE, 1990) computer model
- Debris/sediment yield was determined using the U.S. Army Corps of Engineers Los Angeles District Method for Prediction of Debris Yield (2000).
- The Dike No. 4 storage analysis was performed using the Haested Methods PondPak v7.5 basin routing model.

Flood hydrographs were developed for the 100-year and Standard Project Flood events. The assumptions applied in their development include the following:

- Dike No. 4 has a total drainage area of 27.7 square miles, divided among three subbasins (Devil, Middle, and Toro Canyon areas)
- The Standard Project Flood 6-hour storm pattern is based on the Indio Storm of September 24, 1939
- The 100-year storm 6-hour pattern was synthesized from 100-year precipitation depths using selected durations ranging from 5 minutes to 6 hours
- The 100-year 6-hour watershed-average maximum point precipitation depth is 2.90 inches, estimated from NOAA Atlas 2; precipitation depths for shorter durations were determined from the Cathedral City precipitation gauge and adjusted based on the 6-hour depth ratio between both sources (NOAA Atlas 2 and Cathedral City)
- The Standard Project Flood watershed-average maximum point precipitation depth is 6.45 inches, based on the Indio Storm of September 1939
- A depth-area-reduction factor of 0.94, determined from the NOAA Atlas 2 depth-areal reduction curves, was applied to the 100-year 6-hour storm, reducing the watershed-average maximum point precipitation depth from 2.90 inches to 2.74 inches
- A depth-area-reduction factor of 0.82, determined from USACE (1980), was applied to the Standard Project Storm, reducing the average-watershed maximum point precipitation depth from 6.45 inches to 5.29 inches

- A constant loss rate of 0.20 inches per hour was applied to all subbasins to account for precipitation losses
- A hydraulic roughness value of 0.035 and the Whitewater S-graph were applied in the determination of the lag and unit hydrograph

The debris yield was computed based on the following assumptions:

- Equation 2 was used compute the debris yield for each individual subbasin and Equation 4 was used to compute the debris yield for the combined drainage
- A fire factor of 3.0 was used since fuel for wildfires was considered minimal
- No Area-Transposition factor was applied

The adopted storage analysis was conducted using the following assumptions:

- A basin infiltration rate of 2.5 inches per hour was assumed for the SPF and no infiltration for the 100-year storm;
- No pipe outflow;
- 50 percent of debris/sediment yield is transported to the basin for the Standard Project Flood and 100 percent for the 100-year storm;
- Volume rating curve derived from aerial topographic mapping with minimum 2-ft contour intervals and spot elevations based on NGVD29, flown June 27, 2001 by R.J. Lung and Associates and processed by Mainiero, Smith and Associates

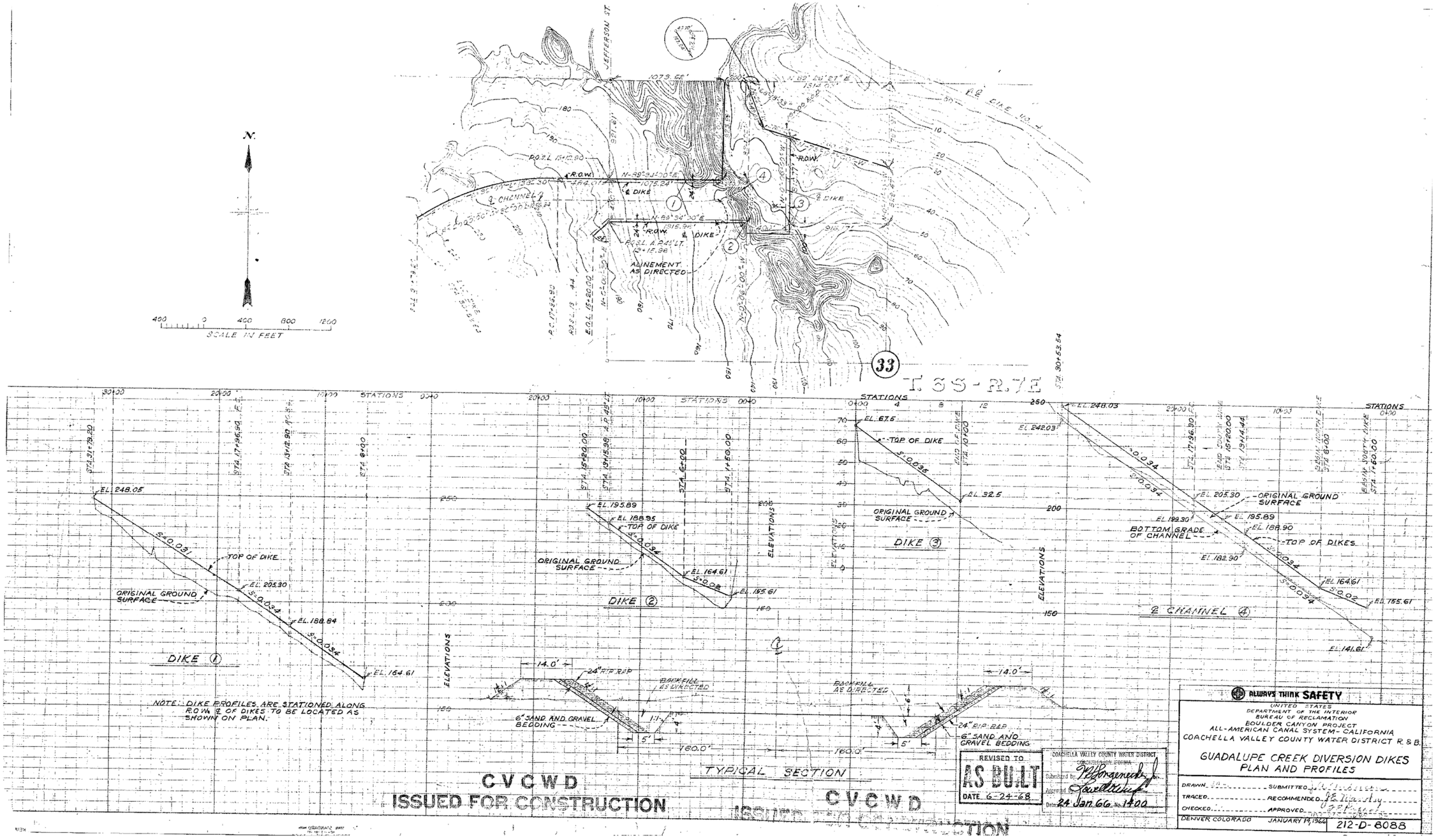
Storage analysis conclusions:

- The Levee crest is to be maintained at elevation 25.0 feet NGVD29; the elevations were reported to vary from 23.6 feet to 26 feet; an estimated 1,500 cubic yards of fill were required to raise the crest to a minimum of 25.0 feet NGVD29
- The computed 100-year maximum water surface elevation was 6.9 feet NGVD29 (18.1 feet of freeboard)
- The computed SPF maximum water surface elevation was 20.3 feet NGVD29 (4.7 feet of freeboard)
- Wave runup calculations resulted in 2 feet for 40 mph winds and 3 feet for 60 mph winds

2.2 Guadalupe Creek Diversion Dikes

As noted previously, the Guadalupe Creek Diversion Dikes are aligned parallel to northern Travertine property boundary and direct flow to the east. Information related to the Guadalupe Creek Diversion Dikes is sparse, only briefly being mentioned in previous studies (Bechtel, 1991) and presented in a single-sheet as-built drawing of the plan and profiles (CVCWD, 1968) as shown in Figure 2-1.

Figure 2-1. Guadalupe Creek Diversion Dikes – as-built plan and profiles (CVCWD, 1968)

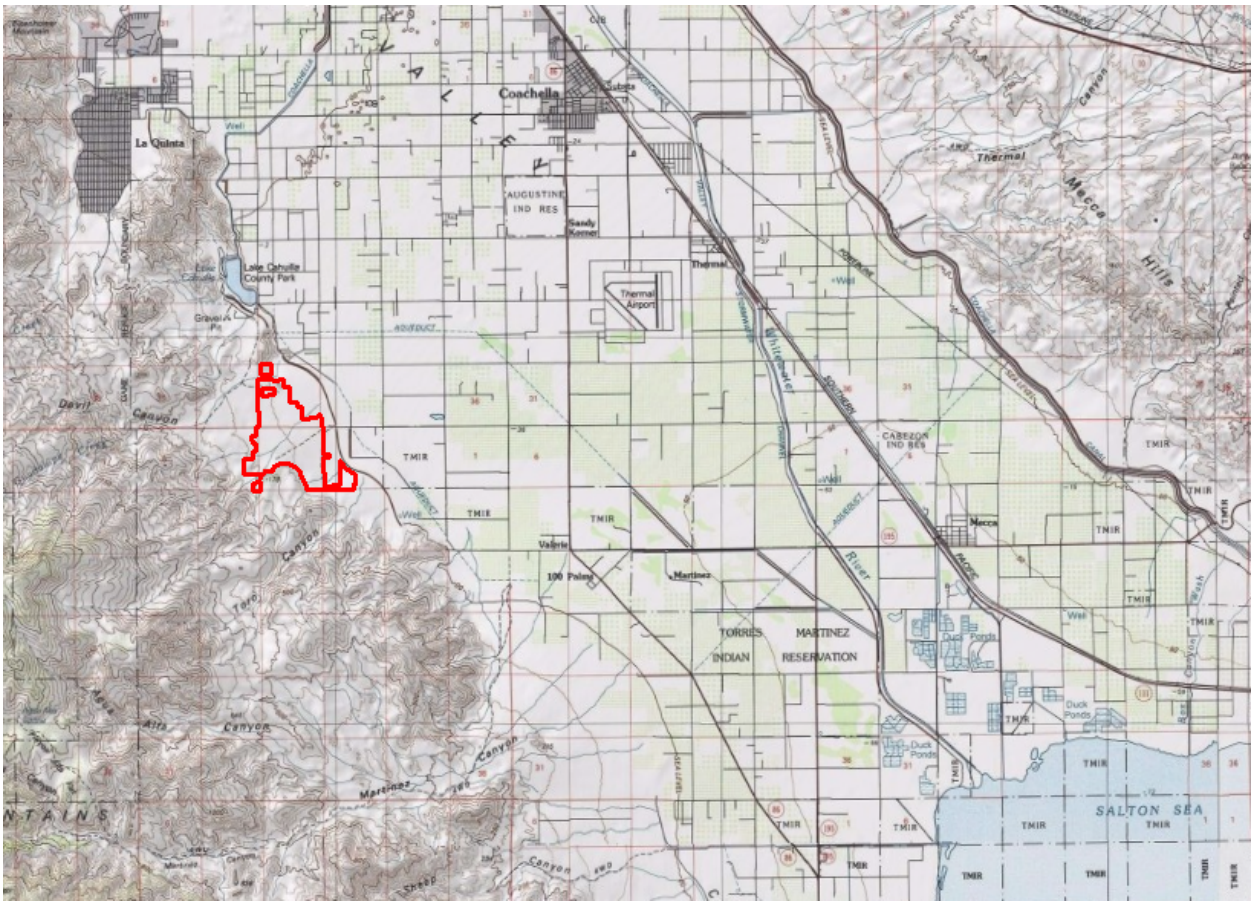


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3 GEOMORPHIC WATERSHED ASSESSMENT

Geomorphic analysis was conducted to identify landform features and evidence of flooding and sedimentation processes that would impact site development alternatives for the proposed Travertine Development (Figure 3-1). This reconnaissance-level analysis focused on alluvial fans, as well as on watercourses with severe lateral erosion potential. Areas subject to active (and inactive) alluvial fan flooding, distributary flow path flooding, shallow sheet flooding, debris and mud flows, and ponding were identified. Without adequate consideration of geomorphic processes operating in the study area, future drainage improvements could be at risk of performance failure.

Figure 3-1. Location of Travertine on the eastern piedmont of the Santa Rosa Mountains



3.1 Site description

The Travertine Development is located on a piedmont bajada composed of moderately steep-sloped relict (inactive) and active alluvial fans. This bajada extends eastward within an embayment of the Santa Rosa Mountain front, and slopes toward the floor of the Coachella Valley and the Salton Sea. The portions of the piedmont that have active alluvial fan areas do not have a strongly defined fan shape (Figure 3-2), but the distributary channel pattern and surficial geology (Figure 3-3) suggests some potential for flow-path uncertainty as well as relatively high rates of sediment transport. The lack of a defined radial fan shape often is consistent with low rates of aggradation or even net degradation and dissection in places.

Figure 3-2. USGS topography showing a lack of radial fan contours on the upper piedmont

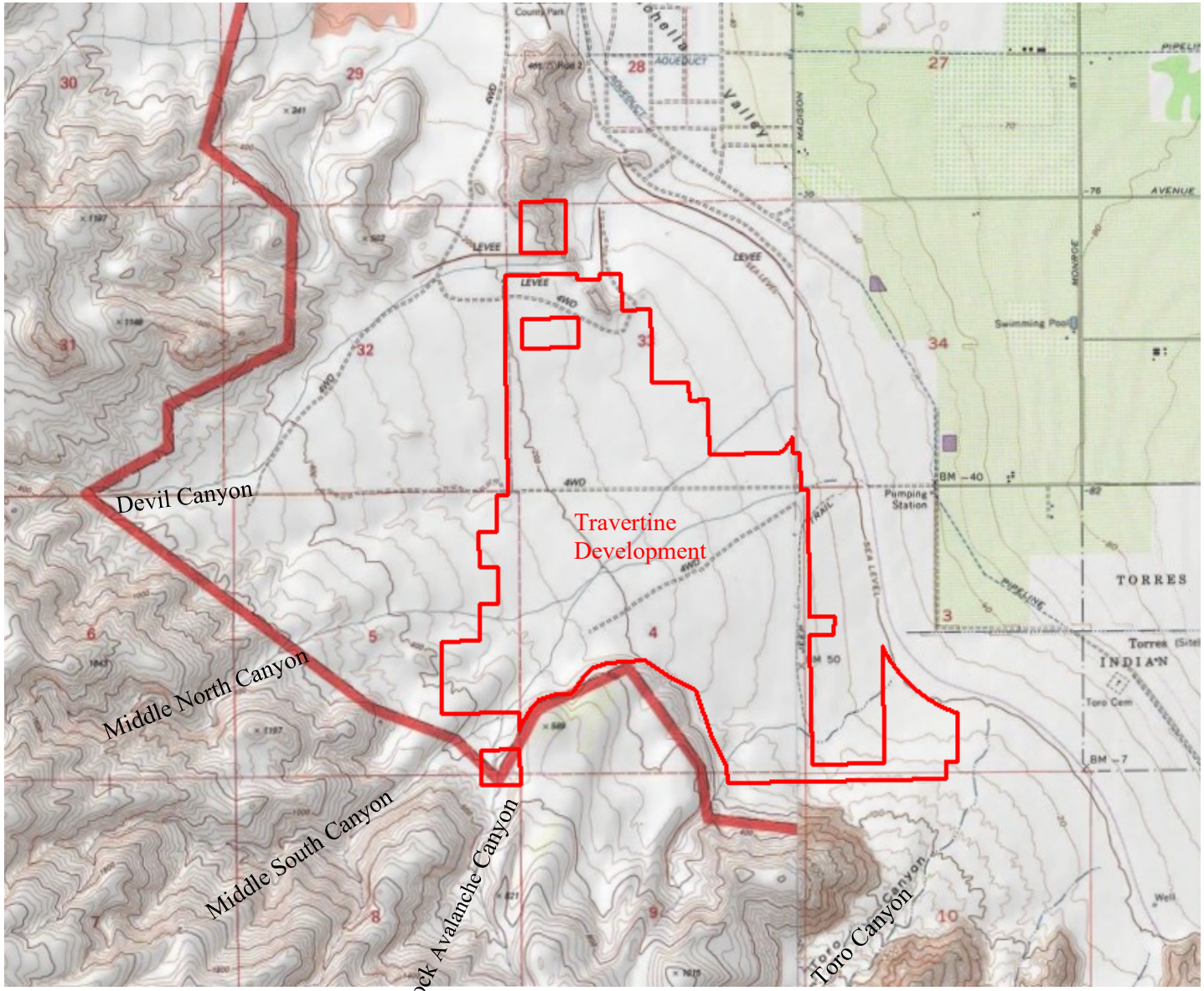
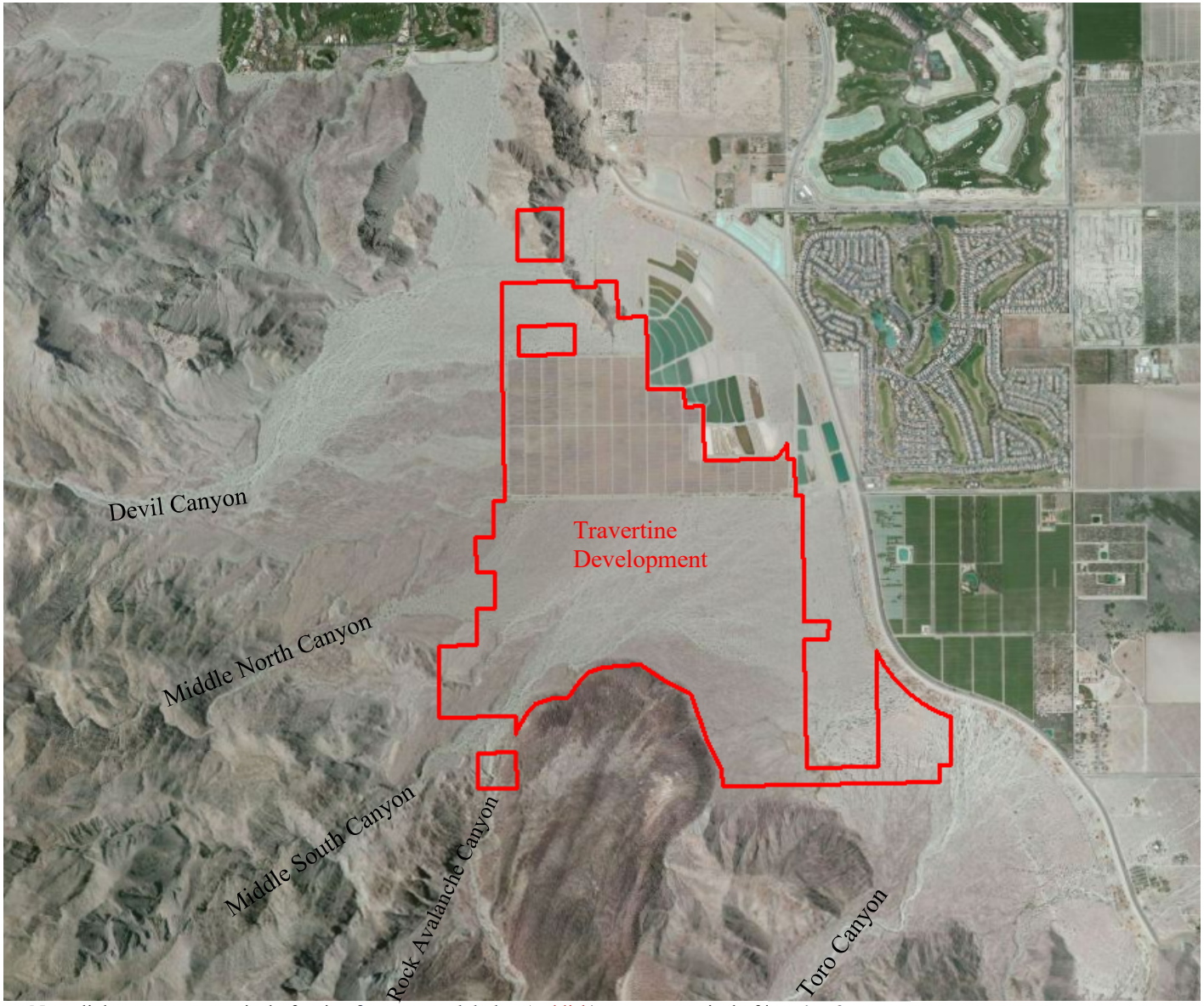


Figure 3-3. Recent aerial shows active/inactive fan areas and evidence of development activities



Note: lighter tones are typical of active fan areas and darker (reddish) tones are typical of inactive fan areas

Eight canyon drainages (subbasins) contribute runoff to the piedmont above Dike No. 4, which have been identified as follows (from north to south):

- Devil Canyon
- unnamed canyon
- Middle North Canyon
- unnamed canyon
- Middle South Canyon
- Rock Avalanche Canyon
- unnamed canyon
- Toro Canyon
- unnamed canyon
- unnamed canyon

The major contributors are Devil Canyon, Middle North Canyon, Middle South Canyon, Rock Avalanche Canyon, and Toro Canyon, which are identified as well in Figure 3-3. The unnamed canyons (not shown in Figure 3-3) are generally shallow canyons along the mountain front. Devil Canyon has the largest drainage of those tributary to the Travertine Development, with an area that extends to the ridgelines of the Santa Rosa Mountains at elevations near 2,000 feet above sea level. Middle North, Middle South and Rock Avalanche Canyons (collectively part of what is referred to as the Middle Canyons Area) are smaller, but coalesce on the piedmont within the Travertine property boundaries. Toro Canyon is a moderate-sized subbasin, which impacts only the southern edge of the Travertine property. There are active alluvial fans associated with each of the ten subbasins, all varying in size and degree of fan activity.

3.2 Methodology

The geomorphic analysis was based on field observations, the interpretation of aerial photographs, the evaluation of topographic maps, and a review of general soils and geologic information. Surficial characteristics such as development of desert varnish, desert pavement, weathering of surface rock, color, channel pattern, drainage network development, channel incision, topographic relief, and vegetative suites were examined to identify active and relict fluvial processes. These surficial characteristics are indicative of surface age, which in turn, is tied to the flood and erosional history of the surface. That is, old surfaces become "old" by not being subject to flood inundation or to widespread erosion and sediment deposition over very long time periods, typically hundreds or thousands of years. Using this methodology, active and inactive areas on the piedmont were readily distinguished. Active areas are subject to potential flow-path uncertainty. For inactive alluvial fan areas, the risk of flow-path uncertainty is very low or nonexistent and can be set aside in the evaluation of flood hazards.

3.3 General findings

The study area for this geomorphic assessment consists of the area within and near the Travertine property limits, and is focused on the watercourses and floodplains that drain to the development boundaries, as indicated by the extents of Figure 3-4 and Figure 3-5. The following general findings and conclusions apply to the entire study area:

- **Debris flow potential**. There is low potential for debris flows to impact the lands within the development property limits. Debris flows are a type of flooding that consists of thick slurries of mud, rock, sediment, water and debris that originate as mass movement on steep slopes or in steep canyons. While debris flows can occur in the Santa Rosa Mountains, because the mountains are low in elevation with generally thin soil mantles, debris flows tend to be limited to the slopes of steep canyons, and tend not to run out beyond the canyon bottoms, with even less potential to reach the mountain front. Also, the watershed is poorly vegetated, making it less vulnerable to catastrophic wild-fire, which is a causative factor for many debris flows in Southern

California. Finally, the Travertine Development boundaries are located nearly a mile from the mountain front, making the possibility of debris flows reaching the project limits even more remote. Therefore, whatever alluvial fan flooding impacts exist for the Travertine Development, they are limited to fluvial processes (water floods).

- **Debris/sediment yield**. All of the subbasins within the study area are likely to deliver relatively high sediment loads, which is typical for floods on arid region streams. Sediment concentrations are likely to be in excess of 10 to 20 percent, with a high proportion of coarse sand and gravel conveyed as bed load. Flood control structures built for the Travertine Development should be designed to accommodate, convey, or store sediment delivered from the off-site drainages. Much of the sediment delivered from the mountainous portions of the upper drainages is deposited and stored on the active alluvial fan surfaces upstream of the Travertine property limits.
- **Transmission losses**. The surficial geology and channel morphology on the fan surfaces upstream of the Travertine Development suggests that transmission losses are an important process on the inundated portions of the active and inactive fan surface. The beds of the broad, distributary channel systems that convey runoff over the fan surfaces are comprised of coarse sand and gravel, which have high transmissivity and are capable of absorbing and storing significant volumes of flow. This results in significant flood attenuation, particularly during small floods. As floods spread out in the distributary channels on the fan surfaces, they not only lose volume to the substrate, they lose stream power because of the massive flow expansion. As they lose stream power, they drop their sediment load, which enhances the distributary flow pattern as well as the potential for further flow attenuation and subsequent transmission losses. Transmission losses should be accounted for in the routing of floods across the piedmont surfaces prior to reaching the boundaries of the Travertine Development.
- **On-fan flooding sources**. The piedmont draining to the Travertine Development is large enough that rainfall on the piedmont can generate significant flooding on its own, without contributions from the mountain drainages. Runoff from on-fan precipitation should be accounted for in the design of flood control and drainage facilities at the Travertine Development.
- **Active/inactive alluvial fans**. As described below, a large portion of the piedmont surface in the vicinity of Travertine consists of readily identified inactive alluvial fans. It is likely that many of the areas currently shown as active in Figure 3-4 and Figure 3-5 have not been active for hundreds of years or are subject only to shallow sheet flooding. The most active alluvial fan areas are limited in extent and located close to the mountain front upstream of the Travertine Development.
- **Past development impacts**. There are development activities on the piedmont surface that impact the natural, pre-development flood hazards to some degree. The piedmont is far from pristine. Development impacts observed during the field reconnaissance and from inspection of aerial photographs include mass grading and construction of flood control berms and recharge ponds, channelization associated with agricultural development, sand and gravel mining excavations, road construction, utility lines, a major aqueduct, residential development, and construction of engineered and non-engineered diversion dikes and levees.
- **Geologic history**. While the piedmont surface may been formed by alluvial fan flooding and debris flows in the distant geologic past, field evidence suggests that the geologic history over the past 10,000 years has been dominated by erosion of the lower and mid-range of the piedmont, with low rates of periodic aggradation due to fluvial processes in the upper piedmont. Field evidence includes: (1) declining base level due to retreat of the Salton Sea during the late Holocene, (2) exposed (unburied) shorelines of Lake Cahuilla, (3) drastic reduction in sediment size (transport capacity and yield) between older and younger fan surfaces, and (4) overall reduction of active portions of the fan surface relative to the size of the fan landform. Within typical engineering time scales, net aggradation will be minimal, as will the effect of sedimentation aggradation on drainage boundaries, and flow path uncertainty.

- **Fan toe**. The toe of the fan landform (piedmont), or downstream end of the piedmont was defined based on topographic contour pattern shown on the USGS topographic maps. The toe is located east of Dike No. 4, and represents the geographic boundary between the piedmont slope and valley floor.

3.4 Landform mapping

A reconnaissance-level map of geomorphic landforms in the study area was created to identify the types of expected flood hazards in the Travertine Development watershed. The following geomorphic map units are depicted on Figure 3-4 and Figure 3-5:

- **Mountains/bedrock**. The mountain/bedrock map unit consists steep mountain slopes, exposed bedrock, and deep canyons. These areas are generally sources for runoff and sediment supply and are located in the upper drainages well outside the limits of the Travertine Development property lines.
- **Active alluvial fans**. Active alluvial fans are landforms that a composed of alluvial sediment deposits (loosely compacted sand, gravel, cobbles, and boulders, with minor amounts of fine-grained sediments). Active alluvial fans are subject to periodic flood inundation, active sediment transport (erosion and deposition). However, the defining characteristics for active alluvial fans is the potential for channel avulsions (sudden relocations of flood channels) and/or net aggradation (progressive sediment deposition) that can lead to flood flow path uncertainty.

  Design of flood control structures on or downstream of active alluvial fan should account for the potential for flow path uncertainty and sedimentation. Included in this unit are weakly or marginally active alluvial fans that may have low rates of aggradation and more stable channel patterns.

  Within the study area, active alluvial fans are located at and near the mountain front. The largest subbasins have active alluvial fans that extend farther down the piedmont. However, the more distal portions of the active alluvial fans are less subject to sediment deposition and are better classified as transport rather than depositional (aggrading) surfaces, and typically are subject to sheet flooding rather than channelized stream flow. In the case of the Travertine study area, the lowest portions of the active alluvial fans have been completely altered by development to the degree that the natural flood processes may no longer exist.

  The following sub-units were also defined within the active alluvial fan category:

  1. **Active**. These areas consist of the least disturbed, most active surfaces closest the mountain front. Flood hazards in this map unit are the most severe, with the highest flow rates, depths, and velocities and the highest rates of sediment deposition, erosion and avulsion.
  2. **Active – sheet flooding area**. These areas consist of the mid- to distal portions of active alluvial fans where flow has transitioned from single and distributary stream channels into widely inundated areas of shallow sheet flow. Where there is some degree of flow path uncertainty in sheet flooding areas, the floods tend to be relatively passive due to low flow depths and velocities. Concentration of sheet flooding by development can lead to significant erosion, channelization, headcutting, and downstream sedimentation.
  3. **Active – agricultural**. These areas consist of probable active alluvial fans that have been overlain by agricultural development that obscures and/or alters the natural flood regime. Note that the geomorphic mapping is for the pre-development surface. Modern agricultural development may alter the pre-development flood type and/or risk, which is discussed in more detail later in this report.
  4. **Active – ponding**. These areas consist of ponding areas upstream of the large aqueduct that transects the piedmont downstream of the Travertine Development.

5. **Active – urbanized**. These areas consist of the highly developed portions of the study area located downstream and east of the aqueduct.

- **Inactive alluvial fans**. Inactive alluvial fans are landforms that a composed of alluvial sediment deposits (loosely compacted sand, gravel, cobbles, and boulders, with minor amounts of fine-grained sediments). Inactive alluvial fans are essentially former active alluvial fans that have been abandoned and are no longer subject to widespread flood inundation and are no longer receiving sediment deposition from upper watershed sources and are not subject to channel avulsions or flow path uncertainty. Inactive fans generally have tributary drainage patterns that have developed over time from rainfall and erosion of the fan surface.

  Design of flood control structures on inactive alluvial fans is similar to design of such structures in riverine watersheds. On the piedmont surface below Devil Canyon, the mapped inactive area includes erosional surfaces that may receive minor amounts of breakout flows from the active alluvial fan area during the largest floods. It is unlikely that such breakouts have the flow volume, frequency or sediment load to create avulsive flow paths, i.e., warrant inclusion in the active alluvial fan flooding surface unit. Field observations indicate that such overflows are rare and are primarily erosional in nature.

  Within the study area, inactive alluvial fans are located in the upper piedmont away from the largest mountain watershed sources, and along the margins of the active alluvial fans. Inactive fans can be identified by their tributary drainage patterns, greater internal topographic relief, and darker surface color due to surficial aging processes.

  The following sub-units were also defined with the inactive alluvial fan category:

  1. **Inactive**. These areas consist of primary undisturbed piedmont surfaces. Some of the inactive alluvial fan surfaces, e.g. portions of the piedmont downstream of Devil Canyon, are cut by stable channels that may convey overflows from the active portion of the piedmont (DC1 and DC2 in Figure 3-5). These overflows, in conjunction with on-piedmont runoff, contribute to downstream flood hazards which should be considered in the design of flood control measures for the proposed project. The geomorphic information collected for this reconnaissance-level assessment suggests that the risk of avulsion is low along these corridors.
  2. **Inactive – agricultural**. These areas consist of probable inactive surfaces that have been overlain by agricultural development, obscuring and/or altering natural flood processes to some degree. Note that the geomorphic mapping is for the pre-development surface. Modern agricultural development may alter the pre-development flood type and/or risk, which is discussed in more detail later in this report.
  3. **Valley floor**. The valley floor unit consists of area below the piedmont slope and is comprised of former lake bed sediments. The valley floor unit is located entirely east and downstream of the Travertine Development and has no impact on the project.

3.5 Other Considerations

Devil Canyon active alluvial fan. There are two areas where it appears the floodwaters within the active portion of the Devil Canyon fan have overflowed onto areas considered inactive. The hydraulic and hydrologic effects of the overflow flow splits or breakout flows are described in more detail in Sections 4 and 5 of this report. The breakout points are mapped as "DC1" and "DC2" in Figure 3-5; however, despite the appearance of these breakouts, the areas were not included in the active alluvial fan mapping unit for the following reasons:

- **Mapping scale**. At a reconnaissance level of mapping, the potential breakouts did not warrant their own geomorphic map unit, particularly given the other factors identified below.

- **Field observations**. Observations made during the field visit indicate that the potential breakouts are only utilized during the largest discharges and are topographically separated from the low flow conveyance areas within the active fan areas.
- **Geologic context**. The overall geologic context appears to be one of abandonment of the "breakout" flow paths in favor of the active areas defined in Figure 3-4 and Figure 3-5.
- Surficial age. Field observations indicate that the "breakout" areas are significant older and less active than the Devil Canyon active alluvial fan, even though they are not as old as the adjacent inactive surfaces.
- **Flood processes**. The characteristics of the flood processes that occur during breakouts from the active alluvial fan area (overflow at the peak of a hydrograph, minimal bedload transport) are substantively different from flood and geomorphic processes that occur within an active alluvial fan area (avulsion, aggradation, deposition). The "breakouts" are better classified as stable distributary flow points rather than active alluvial fan flooding.
- **High erosion hazard areas**. The potential for lateral erosion exists along all of the flood corridors in the study area due to the presence of high velocities and sand/gravel sediments. The active alluvial fan surfaces are most vulnerable to lateral erosion due to the composition of the sediment, as well as the general lack of compaction, cohesion, vegetative cover, and/or cementation. In contrast, older inactive fan surfaces tend to be comprised of soils that are compacted, cohesive, or armored by deflation and thus, are more resistant to lateral erosion. Sheet flooding surfaces, if left undisturbed, are not subject to significant erosion hazards. However, where sheet flooding is concentrated or channelized it can often cause head-cutting, incision, and lateral erosion.

Figure 3-4. Geomorphic surfaces with flood hazard types overlain on USGS topography

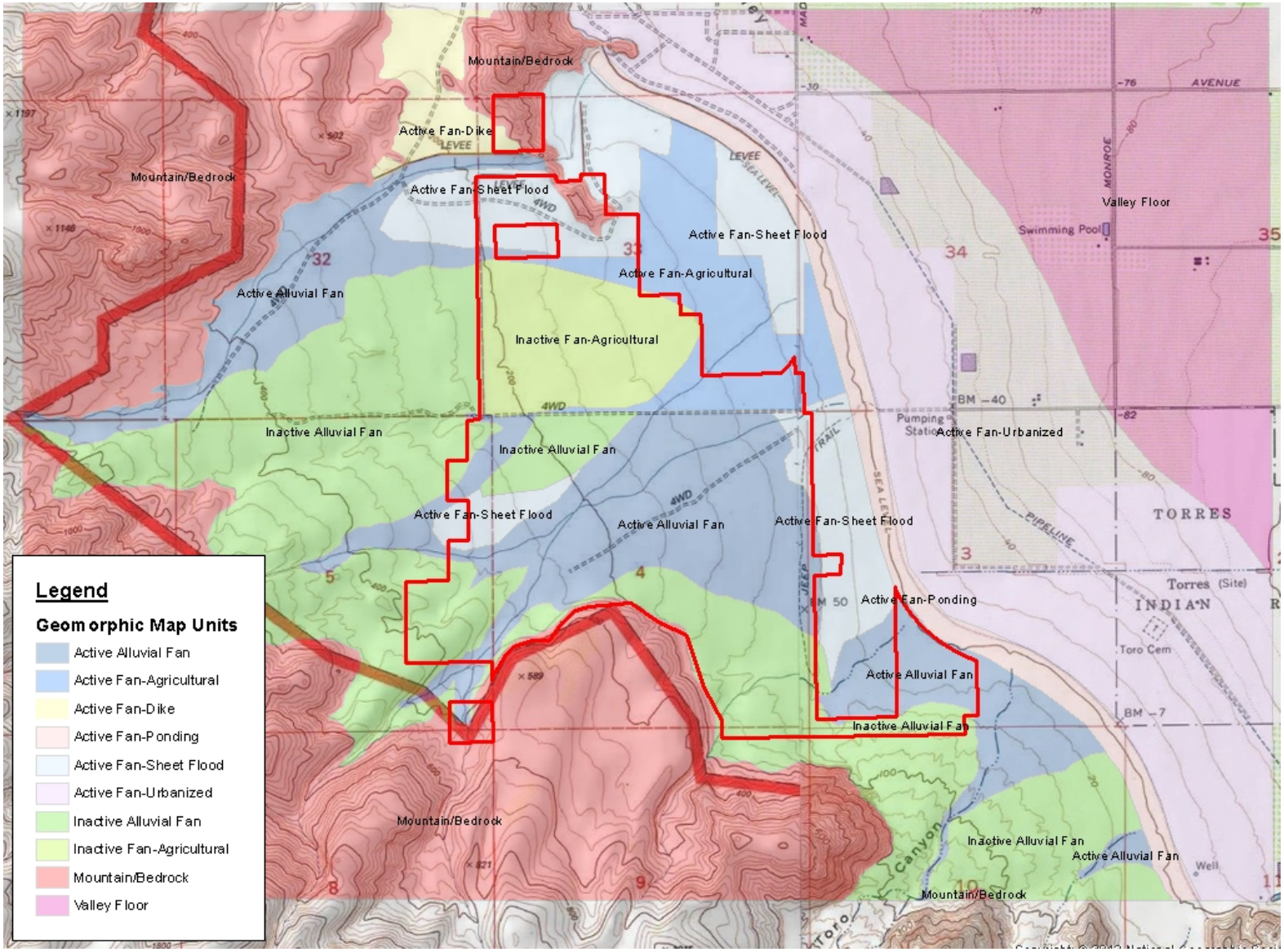
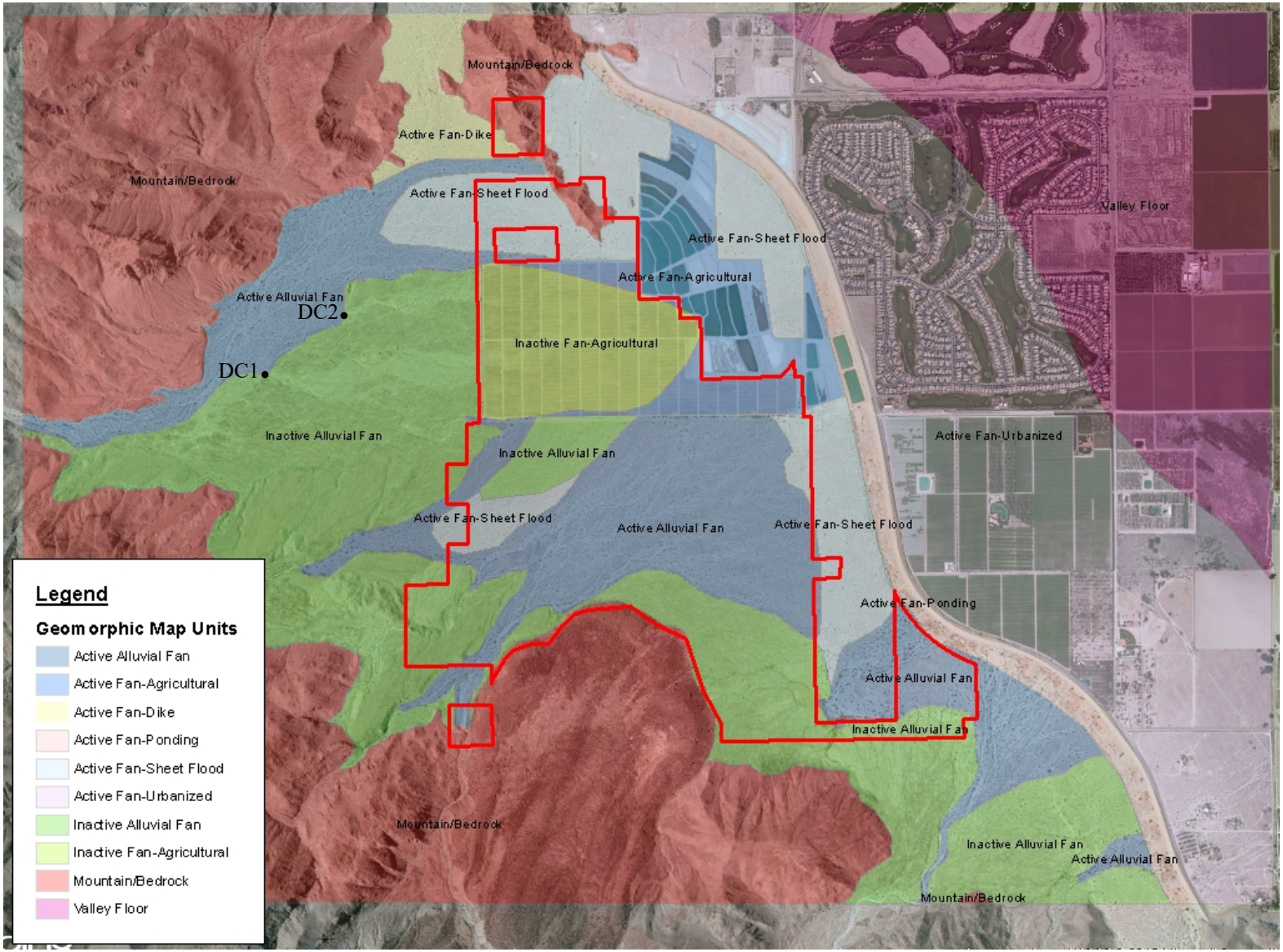


Figure 3-5. Geomorphic surfaces with flood hazard types overlain on recent aerials



4 REGIONAL HYDROLOGY

4.1 Hydrologic goals and objectives

The development of the regional hydrology for the planned development (Travertine) and affected facilities, which includes the Guadalupe Creek Diversion Dikes, Dike No. 4, and the CVWD deep-aquifer recharge basins, is intended to serve as the hydrologic basis for debris analysis, hydraulic modeling, and sediment transport/scour computations conducted for site planning and design as well as for the determination of impacts, mitigation requirements, and engineering constraints associated with the following:

- Conveyance of floodwaters along the edge conditions and near vicinity of Travertine as it relates to stream stability, flood and erosion protection, and consequences to adjacent properties and existing infrastructure, including the integrity and performance of the Guadalupe Creek Diversion Dikes
- Increased runoff volume and flow redistribution attributed to Travertine, which may impact the integrity and performance of Dike No 4 as it relates to freeboard and embankment stability when subjected to the temporary impoundment of floodwaters
- Displacement of impoundment storage and the disruption of impoundment connectivity that will be incurred due to the proposed transportation infrastructure required to provide access to Travertine

Flood hydrographs were developed for contributing drainages consisting of one or more subbasins, based on the 1-percent annual chance storm, for scenarios related to flood conveyance and/or the temporary impoundment of floodwaters along project edge and near vicinity, including the Guadalupe Creek Diversion Dikes, but excluding Dike No. 4. Precipitation areal effects were not considered since the contributing drainage for any of these scenarios is less than 10 square miles.

For matters related to the impoundment of floodwaters along Dike No. 4, flood hydrographs were developed for each subbasin, based on the 1-percent annual chance storm and the Standard Project Storm (SPS). Precipitation areal effects are expected to be significant since the entire watershed, which far exceeds 10 square miles, will be contributing to the development of runoff. The SPS, which is based on the September 24, 1939 Indio Storm, is the event required by the CVWD Development Design Manual to develop the Standard Project Flood (SPF).

Hydrology was developed for the baseline ("without project") and project conditions to support the determination of specific impacts, mitigation requirements, and design constraint related to Travertine.

4.2 Hydrologic analysis – assumptions and general approach

The HEC-HMS Hydrologic Modeling System Version 4.2 (USACE, 2016) was used for all hydrologic model development and simulations performed herein.

The Riverside County Hydrology Manual (RCHM; RCFCWCD, 1978) Synthetic Unit Hydrograph Method (SUHM) was used as the framework methodology for developing flood hydrographs.

Parameter development was performed using a combination of GIS and spreadsheet applications.

Concentration points were designated at or near canyon outfalls (above the Travertine property boundary) and along the interior depression at the base of Dike No. 4.

The hydrologic parameters were determined using the following criteria, methods, and data resources:

- The 1-percent annual chance storm was implemented using a hypothetical (synthetic) pattern accomplished by nesting precipitation depths for duration ranging from 5 minutes to 6 hours

- Frequency-duration point precipitation depths were extracted from the NOAA Atlas 14 spatial datasets
- The SPS is based on September 24, 1939 Indio Storm, which produced a maximum total point precipitation of 6.45 inches over 6.25 hours
- Areal effects were estimated for the SPS and the 1-percent annual chance storm using the depth-area relationship derived from the September 24, 1939 Indio Storm (Plate 4; USACE 1980) for contributing drainages exceeding 10 square miles; precipitation depths for durations less than one hour were not adjusted
- Constant loss rates were determined based on the method prescribed in the RCHM; however, the low loss fraction (low loss rate) was not applied in conjunction with the constant loss rate, as prescribed in the RCHM, which is consistent with CVWD development guidelines
- An initial abstraction was applied in the determination of precipitation losses based on typical values used in similar environments (NHC, 2014; Bechtel, 1997)
- Topographic-based physical parameters, including drainage boundaries, lengths, and slopes were based on 5-meter digital terrain model developed from interferometric synthetic aperture radar (IFSAR) topographic datasets (Intermap Technologies, 2005)
- The lag formula for southern California (USACE, 1962; RCFCWCD, 1978) was used in conjunction with the Whitewater S-graph (RCFCWCD, 1978; USACE, 1980) to transform unit hydrographs; a representative slope analysis was used to estimate the "effective" slope along the longest watercourse; the basin factor (N) was computed based on the area weighting of contributing terrain surfaces of varying hydraulic roughness (e.g., mountain/ hillslopes, active/relic alluvial surfaces/piedmont, Travertine/developed areas)

The following outlines the general procedure used to develop the hydrologic models for simulation:

- Delineate the watershed, subbasins, and define the stream network to support the concentration points required to satisfy the hydrologic objectives
- Identify the storm and storm pattern; estimate the applicable frequency-duration precipitation depth(s) and areal adjustment to each contributing subbasin
- Determine the initial abstraction and constant loss rate for each subbasin for all conditions/scenarios
- Determine the lag parameters and S-graph for each subbasin
- Determine the channel routing parameters
- Configure the watershed model, including basins, processes, and there ordered connectivity; assign the required parameters as well as time-series and paired datasets

4.3 Synthetic Unit Hydrograph Method

The Synthetic Unit Hydrograph Method (SUHM; RCFCWCD, 1978) was used to develop flood hydrographs for each delineated subbasin within the watershed tributary to Dike No. 4. The SUHM is statistically based, assuming the watershed discharge is related to the total volume of runoff. The time factors affecting the shape of the SUHM are dominant. The watershed storm rainfall-runoff relationships are characterized by watershed area, slope, and shape factors. The SUHM is used to estimate the time distribution of watershed runoff in drainage basins where stream gauge information is not available. In Riverside County, the SUHM is normally used to evaluate individual drainage areas in excess of 300 to 500 acres.

4.3.1 Watershed delineation, existing (baseline) conditions

Travertine is located within the lower extent of the watershed tributary to Dike No. 4, on the eastern piedmont skirting the base of the Santa Rosa Mountains, generally situated on the southern terrace of the active floodplain formed below the Devil Canyon outfall. The watershed tributary to Dike No. 4, associated herein with the hydrologic node prefix of D4, encompasses just over 27 square miles as

determined from the delineated watershed presented in Figure 4-1 (baseline conditions) and Figure 4-2 (project conditions) based on IFSAR topographic data (Intermap Technologies, 2005).

The Dike No. 4 watershed was subdivided into 10 canyon subbasins and three (3) piedmont subbasins. Canyon subbasins have a corresponding hydrologic node (concentration point) located at or near each of their respective outfalls and outside and above the Travertine property boundary. The piedmont subbasins each have a concentration point located in the depression along the base of Dike No. 4. The delineated subbasins are listed as follows with their respective downstream hydrologic nodes identified in parentheses:

- Devil Canyon (D411)
- piedmont below Devil Canyon (D410)
- unnamed canyon (D422)
- Middle Canyon North(D423)
- unnamed canyon (D424)
- Middle Canyon South (D421A)
- Rock Avalanche Canyon (D421B)
- piedmont below the Middle Canyons (D420)
- unnamed canyon (D432)
- Toro Canyon (D431)
- piedmont below Toro Canyon (D430)
- unnamed canyon (D44)
- unnamed canyon (D45)

Middle Canyon South (D421A), Rock Avalanche Canyon (D421B), Middle Canyon North (D423), and unnamed canyons (D422 and D424 collectively form the Middle Canyons Area. Toro Canyon (D431) and the unnamed canyon (D432) collectively form the Toro Canyon Area.

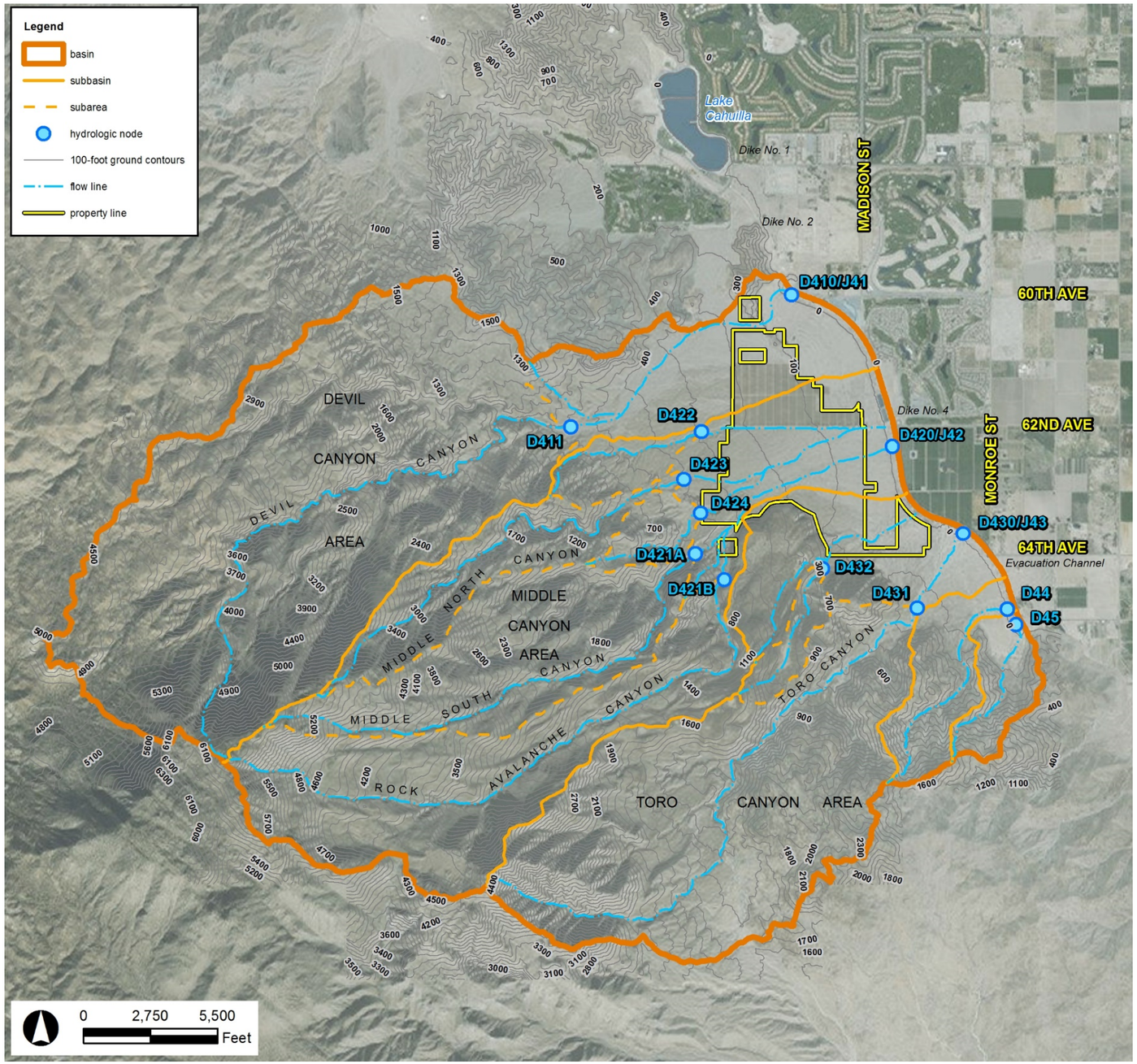
4.3.2 Watershed delineation, project conditions

The planned development causes the following changes to the following interior drainage divides:

- The drainage divide separating subbasins D410 and D420 was adjusted to conform to the concept grading proposed for the planned development shown in Figure 1-4
- The runoff produced by subbasin D422 is diverted from J42 to J41 as a result of the graded footprint of the planned development

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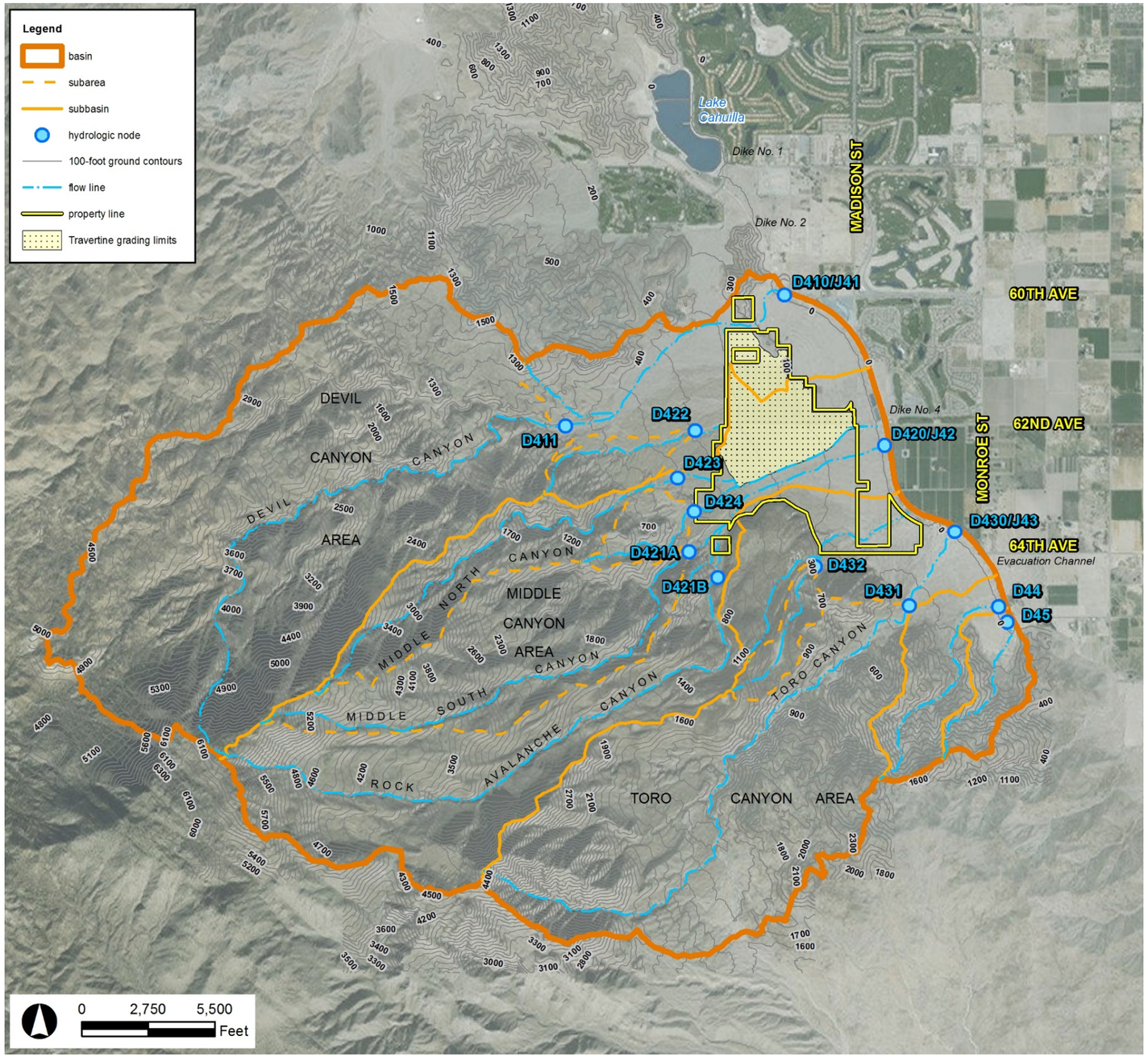
Figure 4-1. Baseline conditions regional hydrology map



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Figure 4-2. Project conditions regional hydrology map



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4.3.3 Precipitation

The storm scenarios that were analyzed herein include the following:

- 1-percent annual chance storm local analysis (local contributing drainages less than 10 square miles; no areal effects)
- 1-percent annual chance storm regional analysis (areal effects based on the entire watershed contributing to the development of runoff)
- SPS/SPF regional analysis (areal effects based on entire watershed contributing to the development of runoff)

4.3.3.1 Standard Project Storm

The SPF is based on the Indio Storm of September 24, 1939 (SPS), which produced a total precipitation depth of 6.45 inches in approximately 6 hours. The SPS precipitation depth was adjusted using the depth-area curve presented in Figure 4-3 (Plate 4; USACE, 1980) assuming the storm covers the entire watershed at the same intensity, producing an adjusted value of 6 inches based on a watershed size of 27.07 square miles. The SPS was applied to the 6-hour storm pattern presented in Plate E-5.9 (RCFCWCD, 1978), which is also based on the September 24, 1939 Indio Storm.

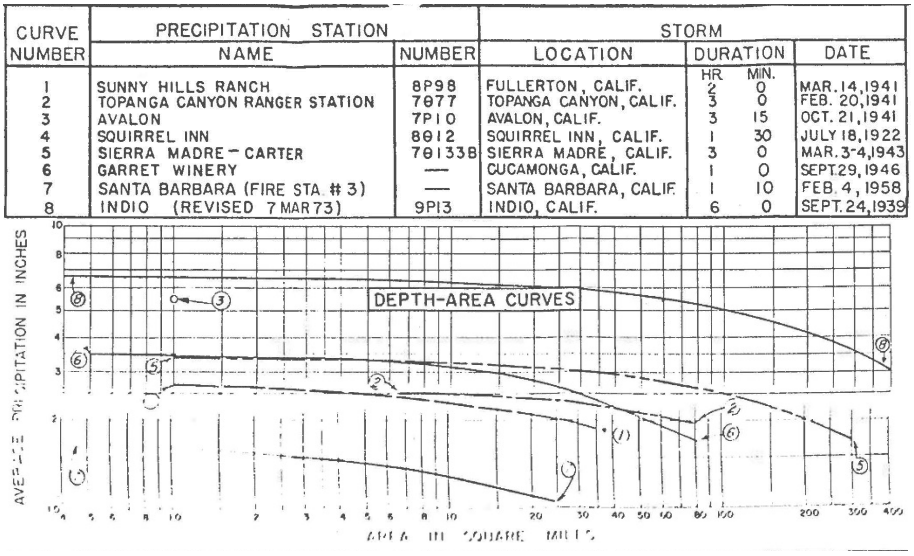
4.3.3.2 1-percent annual chance storm

The 1-percent annual chance storm was synthesized over a 6-hour duration based on the hypothetical storm, which is constructed from nested area-weighted average maximum precipitation depths for 5-minute, 15- minute, 60-minute, 2-hour, 3-hour, and 6-hour durations. These precipitation depths were estimated from the NOAA Atlas 14 spatial datasets (NWS, 2014) and are presented in Table 4-1.

Although the depth-area relationship for the September 24, 1939 Indio Storm shown in Figure 4-3 is specific to the areal nature and duration for this particular storm, CVWD guidance dictates that it be applied to all storms and all durations; therefore, it was applied to each duration precipitation depth of the 1-percent annual chance event. Depth-area-duration adjustments were not applied to individual or combined drainage areas of less than 10 square miles.

The hypothetic storm pattern is balanced around the peak, with the maximum precipitation occurring two thirds of the way through the 6-hour storm period.

Figure 4-3. Southern California local storm depth-area relationships (Plate 4; USACE, 1980)



CURVE NUMBER	PRECIPITATION STATION NAME	NUMBER	STORM LOCATION	DURATION HR	DURATION MIN.	DATE
1	SUNNY HILLS RANCH	8P98	FULLERTON, CALIF.	2	0	MAR. 14, 1941
2	TOPANGA CANYON RANGER STATION	7077	TOPANGA CANYON, CALIF.	3	0	FEB. 20, 1941
3	AVALON	7P10	AVALON, CALIF.	3	15	OCT. 21, 1941
4	SQUIRREL INN	8012	SQUIRREL INN, CALIF.	1	30	JULY 18, 1922
5	SIERRA MADRE – CARTER	701338	SIERRA MADRE, CALIF.	3	0	MAR. 3-4, 1943
6	GARRET WINERY	—	CUCAMONGA, CALIF.	1	0	SEPT. 29, 1946
7	SANTA BARBARA (FIRE STA. # 3)	—	SANTA BARBARA, CALIF.	1	10	FEB. 4, 1958
8	INDIO (REVISED 7 MAR 73)	9P13	INDIO, CALIF.	6	0	SEPT. 24, 1939

4.3.3.3 Storm centering

There is no guidance provided in the RCHM or by CVWD as it relates to the application of storm centering and when it should be required. Current guidance assumes a storm covers the entire contributing area at the same intensity, with a single depth-areal-reduction factor representing the entire contributing area. This approach, referred to as "whole-basin" centering, is suitable for small basins or very large-scale, frontal-type storms. The Dike No. 4 watershed is considered small enough where storm centering is not expected to significantly change the hydrologic outcomes; therefore, additional storm centering scenarios were not evaluated.

Table 4-1. 1-percent annual chance precipitation depths for selected durations

conditions	subbasin	precipitation depths, in *inches*, for selected durations					
		5m	15m	60m	2h	3h	6h
depth-areal reduction* =		0.93	0.93	0.93	0.93	0.93	0.93
baseline	410	0.47	0.81	1.67	2.08	2.39	3.01
		0.44	0.76	1.55	1.93	2.22	2.80
	411	0.72	1.25	2.69	3.18	3.49	4.23
		0.67	1.16	2.50	2.95	3.25	3.93
	420	0.48	0.84	1.72	2.15	2.47	3.11
		0.45	0.78	1.60	2.00	2.29	2.89
	421A	0.67	1.16	2.48	2.98	3.32	4.06
		0.62	1.08	2.30	2.77	3.08	3.78
	421B	0.71	1.23	2.64	3.17	3.52	4.31
		0.66	1.14	2.46	2.95	3.27	4.00
	422	0.56	0.97	2.04	2.50	2.81	3.47
		0.52	0.90	1.90	2.32	2.62	3.23
	423	0.72	1.25	2.69	3.22	3.55	4.33
		0.67	1.16	2.50	2.99	3.30	4.03
	424	0.53	0.92	1.93	2.38	2.69	3.34
		0.49	0.86	1.80	2.21	2.50	3.11
	430	0.47	0.81	1.66	2.10	2.43	3.07
		0.44	0.76	1.55	1.95	2.26	2.85
	431	0.58	1.00	2.12	2.61	2.95	3.65
		0.54	0.93	1.97	2.43	2.74	3.39
	432	0.51	1.06	1.85	2.40	2.63	3.29
		0.47	0.99	1.72	2.23	2.45	3.06
	44	0.49	0.85	1.76	2.22	2.55	3.20
		0.45	0.79	1.63	2.06	2.37	2.98
	45	0.47	0.82	1.68	2.13	2.46	3.12
		0.44	0.76	1.56	1.98	2.29	2.90
project	410	0.48	0.83	1.71	2.12	2.43	3.06
		0.44	0.77	1.59	1.97	2.26	2.84
	420	0.47	0.82	1.68	2.11	2.43	3.07
		0.44	0.76	1.56	1.96	2.26	2.85

*Depth-areal reduction factors were based on a total drainage area of 27.07 sq. mi.;
values in red represent depth-areal reduced precipitation depths

4.3.4 Precipitation losses (constant loss rate determination)

The following process was implemented to determine the constant loss rate for each delineated subbasin within the Dike No. 4 watershed:

- The land use and hydrologic soils spatial datasets were intersected to determine the composition of landuse/ hydrologic-soil-group combinations within each subbasin.
- The dimensionless runoff index value (RI) for pervious areas was determined for each land-use/hydrologic-soil-group combination
- The pervious area constant loss rate (F_P), in *inches per hour*, was determined for each land-use/hydrologic soil-group combination from the relationship between the pervious infiltration rate for pervious areas versus runoff index values (Plate E-6.2; RCFC&WCD, 1978)
- The impervious fraction (A_i) for each land-use/hydrologic-soil group combination was assigned based on the land-use percent imperviousness value (RTIMP)
- The adjusted constant loss rate (F) for each land-use/hydrologic-soil-group combination was computed using the following equation:

$$F = F_p\,(1 - 0.9A_i)$$

- The adjusted constant loss rate computed for each land-use/hydrologic-soil-group combination within each delineated subbasin was area weighted and averaged to determine the average adjusted constant loss rate for each subbasin
- A low loss fraction (low loss rate) was not applied, which is consistent with CVWD guidance

In Riverside County, the NRCS detailed soil survey maps are typically used to estimate the spatial variation of hydrologic soil groups within the drainage basin of interest. The detailed soil maps, which provide coverage within the Dike No. 4 watershed are the Coachella Valley Area Soil Survey (CA680; NRCS, 2008) and the San Bernardino National Forest Soil Survey Area (CA777; NRCS, 2008), roughly encompassing a combined 12 percent of the total watershed. The U.S Generalized Soils Map (NRCS, 2004) was used to supplement soil information for the remainder of the watershed.

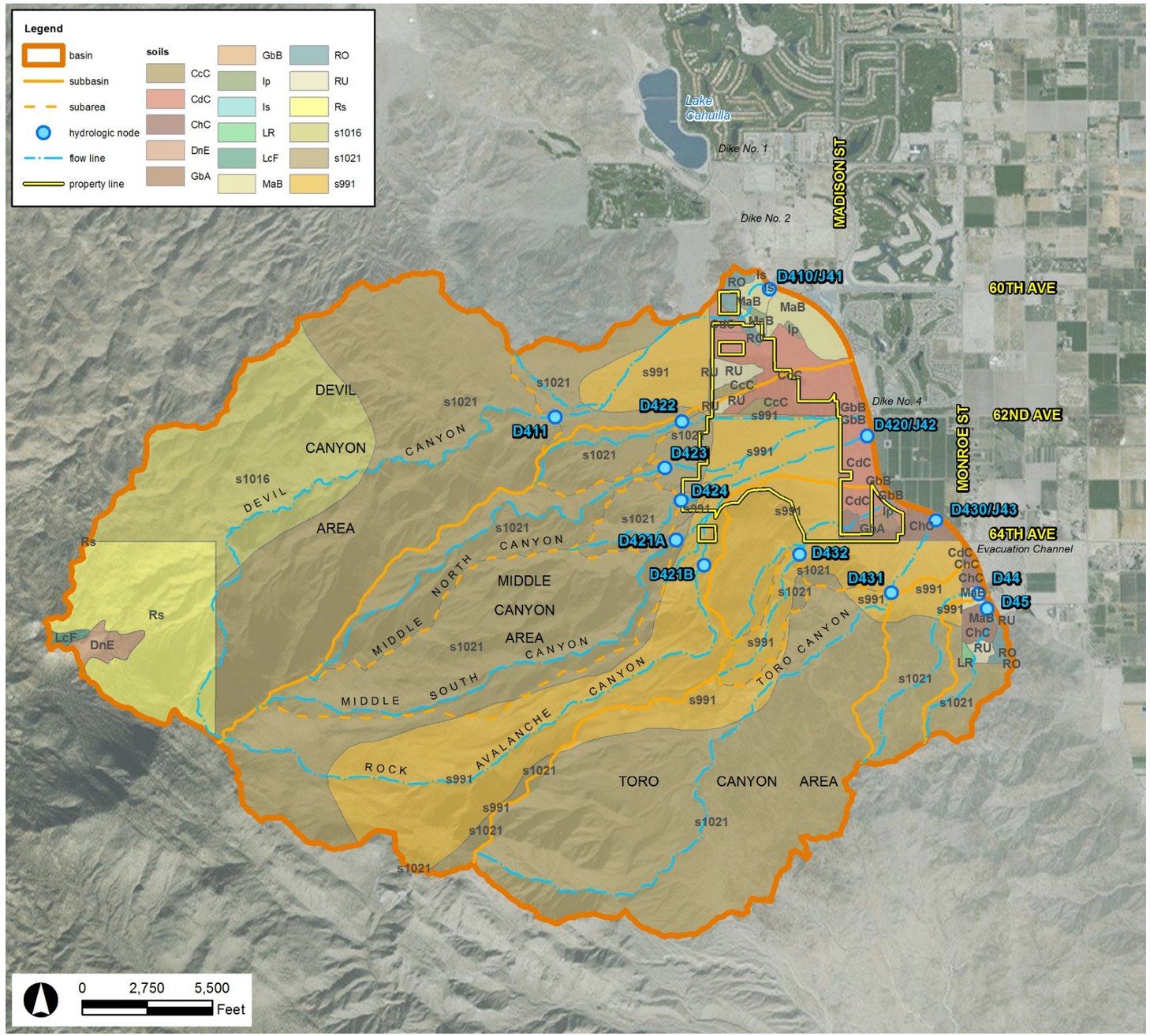
The NRCS soils datasets were used to prepare a composite map of detailed and generalized soil map units for the Dike No. 4 watershed as presented in Figure 4-4 (baseline conditions) and Figure 4-5 (project conditions); the percent distribution of soil map units for each subbasin and the watershed as a whole is listed in Table 4-2. The percent distribution of hydrologic soil groups for each soil map unit are listed in Table 4-3.

The land use dataset published by the Southern California Area Governments (SCAG, 2008) was used to approximate the composition of land uses in the watershed tributary to Dike No. 4 as presented in Figure 4-6 (baseline conditions) and Figure 4-7 (project conditions). The percent distribution of land use map units for each subbasin is depicted in Table 4-4. The land use categories and their respective percent imperviousness are listed in Table 4-5. An average percent imperviousness for the Travertine developed area was estimated from assumed land-use densities based on the current grading concept plan (Figure 1-4).

Land cover was classified as poor-quality desert shrub (NRCS, 2004) for the entire watershed. The loss rate parameterization for each subbasin is summarized in Table 4-6. The detailed loss rate calculation worksheets for each subbasin are included as part of the Electronic Technical Appendix.

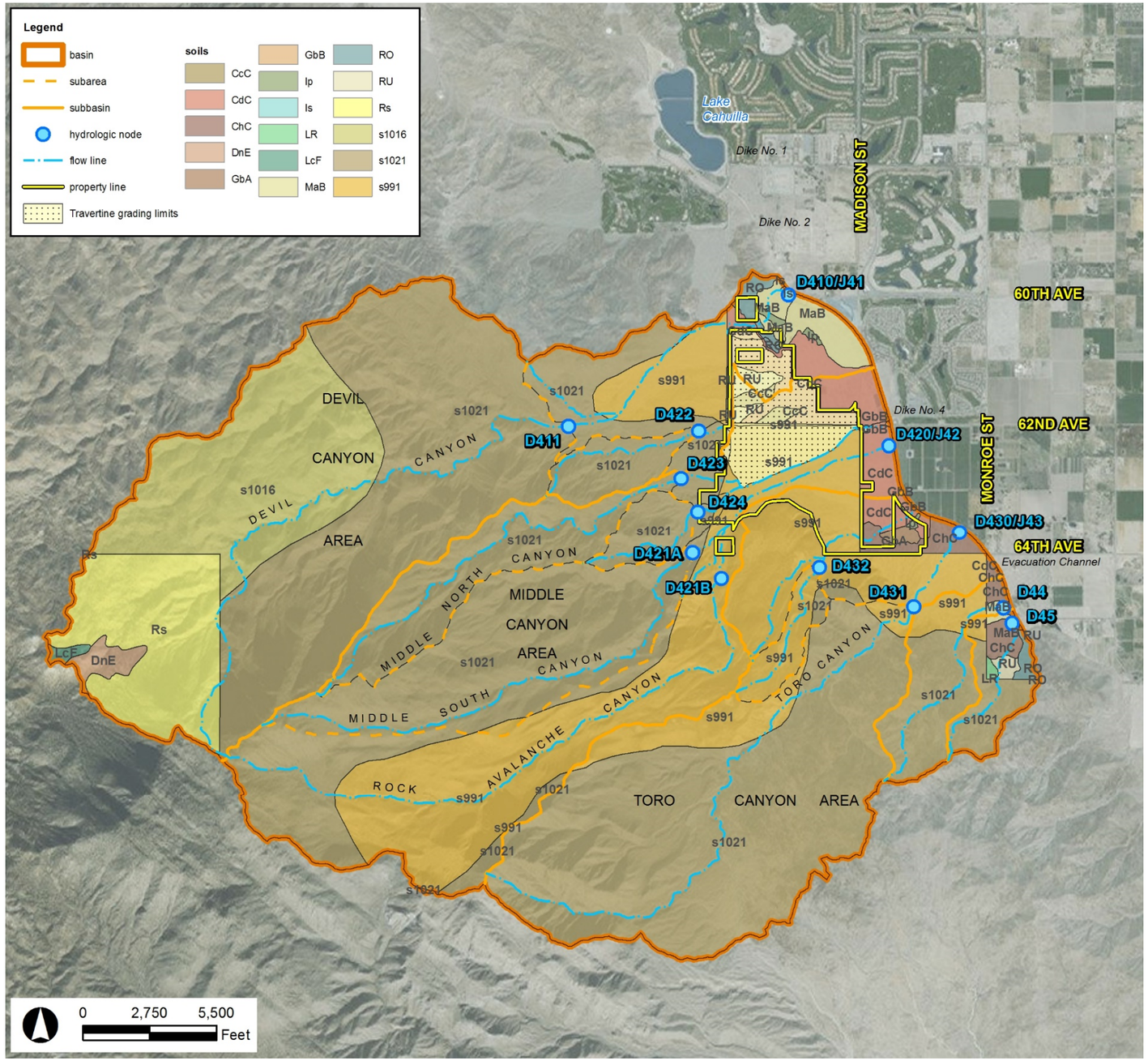
A watershed-average initial abstraction (IA) of 0.25 inches was used, derived from typical values assumed for deserts and rangeland (0.15 – 0.35 inches) and vegetated mountains and hillslopes (0.25), which is consistent with other studies conducted in the Coachella Valley region (NHC, 2014; Bechtel, 1997).

Figure 4-4. Baseline conditions composite soil map



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Figure 4-5. Project conditions composite soil map



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Table 4-2. Distribution of soil map units

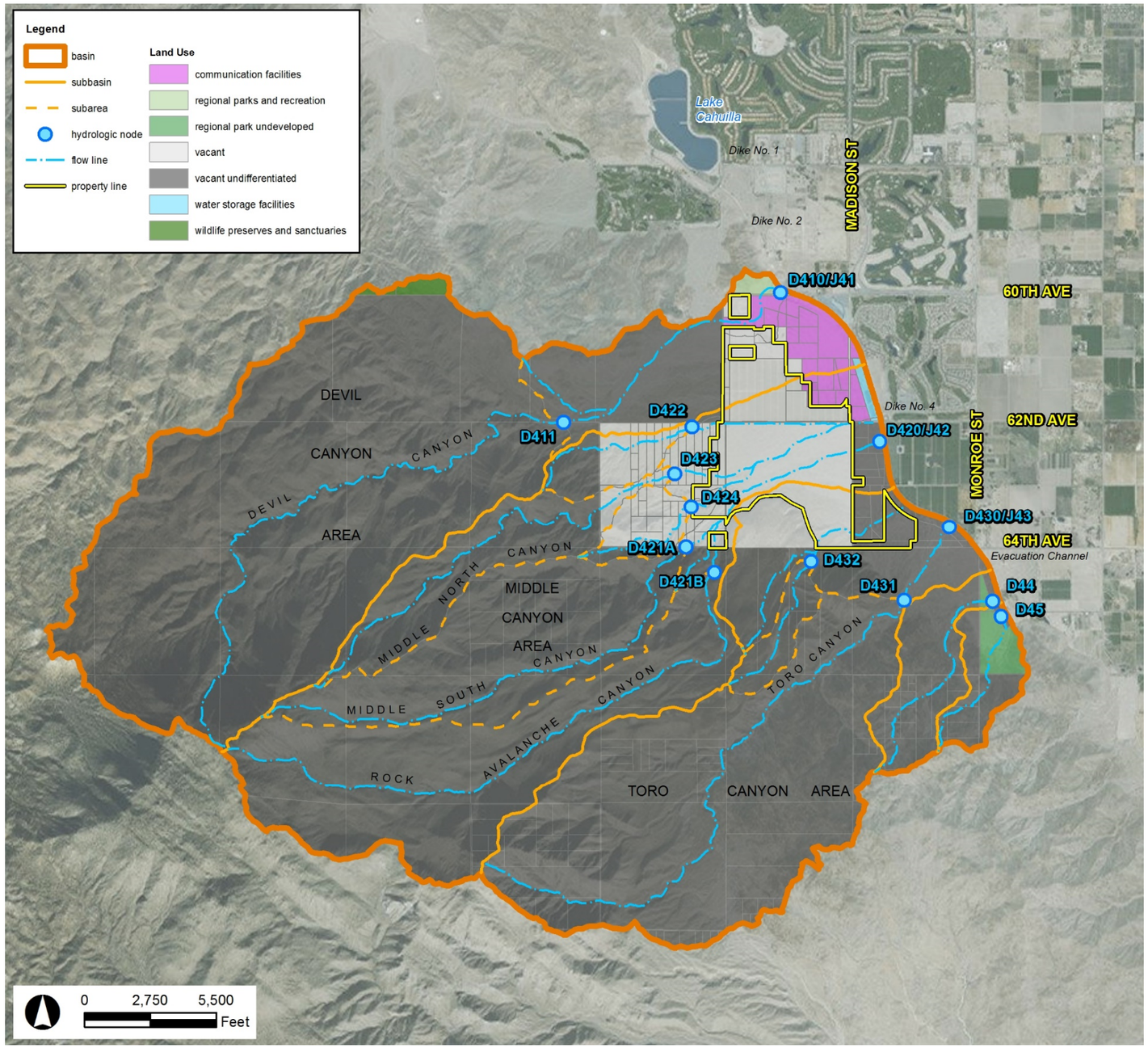
NRCS soil identifier		watershed drainage area {acres}		subbasin drainage area {acres}															
				baseline conditions														project conditions	
survey area	MUSYM	baseline conditions	project conditions	D411	D410	D421A	D421B	D422	D423	D424	D420	D431	D432	D430	D44	D45	D410	D420
CA680	CcC	39	39	0	20	0	0	0	0	0	19	0	0	0	0	0	8	31
	CdC	550	550	0	183	0	0	0	0	0	294	0	0	73	0	0	178	299
	ChC	177	177	0	0	0	0	0	0	0	0	0	0	109	19	50	0	0
	GbA	19	19	0	0	0	0	0	0	0	0	0	0	19	0	0	0	0
	GbB	2	2	0	0	0	0	0	0	0	1	0	0	0	0	0	0	1
	Ip	17	17	0	13	0	0	0	0	0	0	0	0	4	0	0	13	0
	Is	4	4	0	4	0	0	0	0	0	0	0	0	0	0	0	4	0
	LR	7	7	0	0	0	0	0	0	0	0	0	0	0	0	7	0	0
	MaB	184	184	0	168	0	0	0	0	0	0	0	0	0	9	6	168	0
	RO	78	78	0	52	0	0	0	0	0	0	0	0	0	0	25	52	0
	RU	79	79	0	48	0	0	0	0	0	13	0	0	0	0	19	35	25
	W	0.5	0.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
CA777	DnE	74	74	74	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	LcF	19	19	19	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	Rs	905	905	905	0	0	0	0	0	0	0	0	0	0	0	0	0	0
US	s991	3,661	3,661	0	341	16	1,445	0	0	9	481	323	165	771	94	16	346	476
	s1016	1,458	1,458	1,458	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	s1021	10,053	10,053	2,628	361	1,463	723	250	756	223	165	2,901	19	22	365	176	431	95
total:	{acres}	17,328	17,328	5,085	1,190	1,479	2,168	250	756	232	974	3,224	184	999	487	300	1,235	928
	{sq mi}	27.075	27.075	7.946	1.859	2.311	3.387	0.390	1.182	0.362	1.522	5.038	0.287	1.561	0.761	0.468	1.930	1.450

Table 4-3. Hydrologic soil group composition for soil map units

NRCS soil identifier		hydrologic soil composition			
survey area	MUSYM	A	B	C	D
CA680	CcC	100	0	0	0
	CdC	100	0	0	0
	ChC	100	0	0	0
	GbA	100	0	0	0
	GbB	100	0	0	0
	Ip	100	0	0	0
	Is	100	0	0	0
	LR	0	0	0	100
	MaB	100	0	0	0
	RO	0	0	0	100
	RU	100	0	0	0
	W	0	0	0	100
CA777	DnE	100	0	0	0
	LcF	100	0	0	0
	Rs	0	0	0	100
US	s991	100	0	0	0
	s1016	67	0	0	33
	s1021	25	0	0	75

.

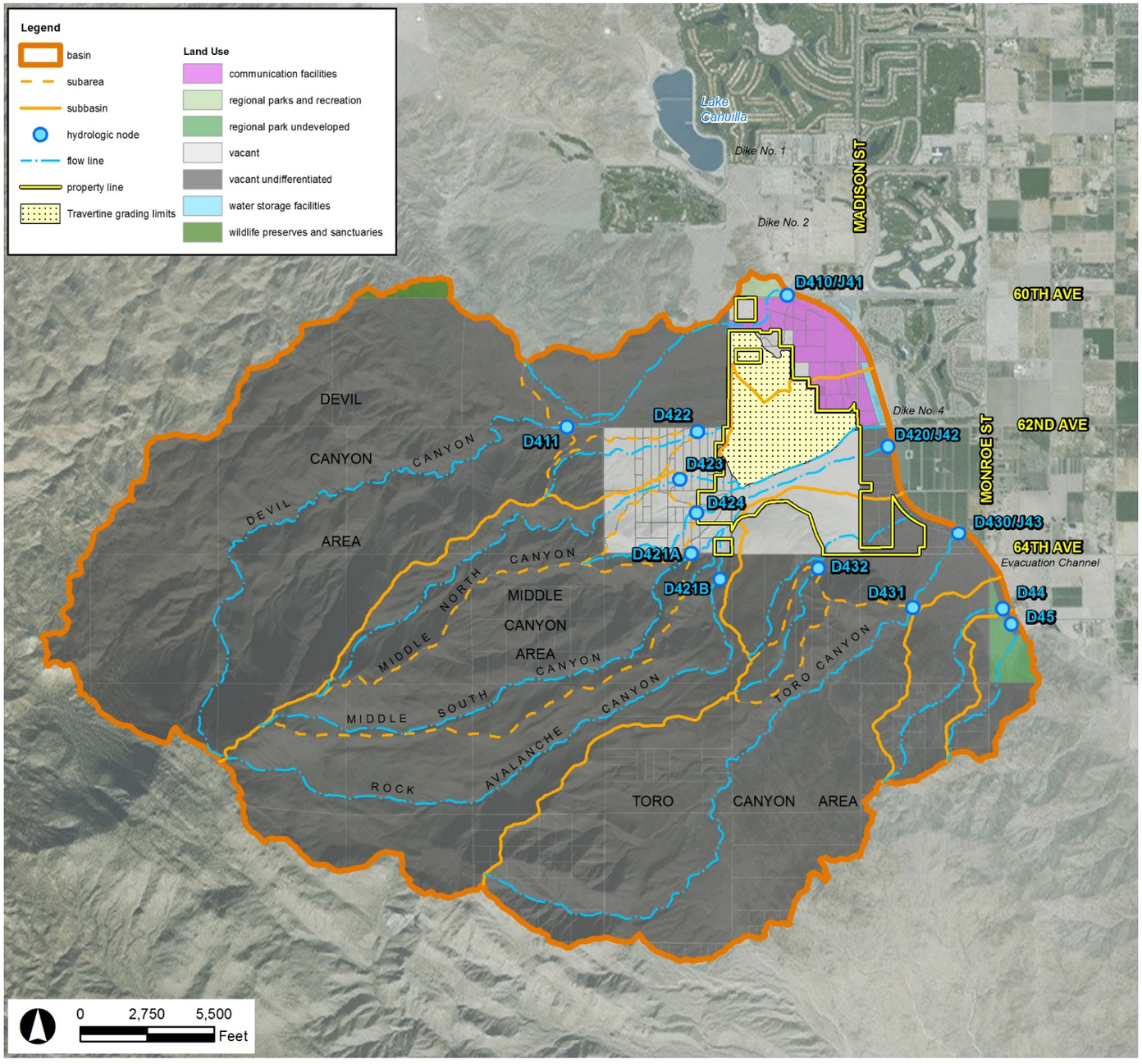
Figure 4-6. Baseline conditions land use map*



*Based on SCAG (2008)

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Figure 4-7. Project conditions land use map*



*Based on SCAG (2008)

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Table 4-4. Distribution of land-use units

land-use identifier		watershed drainage area {acres}		subbasin drainage area {acres}														
				baseline conditions													project conditions	
		baseline conditions	project conditions	D411	D410	D421A	D421B	D422	D423	D424	D420	D431	D432	D430	D44	D45	D410	D420
communication facilities		307	301	0	220	0	0	0	0	0	86	0	0	0	0	0	220	81
golf courses		0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
multi-family residential		0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
regional park undeveloped		102	102	0	0	0	0	0	0	0	0	0	0	0	10	92	0	0
regional parks and recreation		39	39	0	39	0	0	0	0	0	0	0	0	0	0	0	39	0
single family residential		0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Travertine		0	445	0	0	0	0	0	0	0	0	0	0	0	0	0	109	337
under construction		0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
vacant		1,613	1,173	0	244	1	0	138	58	193	701	0	0	278	0	0	172	334
vacant undifferentiated		15,195	15,195	5,040	682	1,479	2,168	112	699	38	164	3,224	184	721	477	208	692	154
water		0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
water storage facilities		26.3	26.3	0	3	0	0	0	0	0	23	0	0	0	0	0	3	23
wildlife preserves and sanctuaries		45	45	45	0	0	0	0	0	0	0	0	0	0	0	0	0	0
total:	{acres}	17,328	17,328	5,085	1,190	1,479	2,168	250	756	232	974	3,224	184	999	487	300	1,235	928
	{sq mi}	27.075	27.075	7.946	1.859	2.311	3.387	0.390	1.182	0.362	1.522	5.038	0.287	1.561	0.761	0.468	1.930	1.450

Table 4-5. Assigned percent imperviousness values for land use categories

land-use identifier	RTIMP {%}
communication facilities	1
golf courses	10
multi-family residential	75
regional park undeveloped	0
regional parks and recreation	5
single family residential	50
Travertine	42
under construction	50
vacant	0
vacant undifferentiated	0
water	95
water storage facilities	50
wildlife preserves and sanctuaries	0

Table 4-6. Summary of subbasin constant loss rates

conditions	subbasin	DA {acres}	RTIMP {%}	hydrologic soil composition				F* {in/h}
				A	B	C	D	
baseline	410	1,190	0	69	1	3	27	0.350
	411	5,085	0	32	0	0	68	0.243
	420	974	1	86	0	1	13	0.395
	421A	1,479	0	26	0	0	74	0.225
	421B	2,168	0	75	0	0	25	0.367
	422	250	0	25	0	0	75	0.223
	423	756	0	25	0	0	75	0.223
	424	232	0	28	0	0	72	0.231
	430	999	0	96	2	0	2	0.431
	431	3,224	0	33	0	0	67	0.244
	432	184	0	92	0	0	8	0.418
	44	487	0	44	0	0	56	0.277
	45	300	0	41	0	4	55	0.270
project	410	1,235	4	66	1	2	30	0.328
	420	928	17	90	0	2	8	0.342

*maximum loss rate based on AMC II and sparse {poor quality} desert shrub pervious

4.3.5 Unit hydrograph transform

The transformation of unit hydrographs is a process that is integrated into the HEC-HMS model definition. The lag formula used for Southern California watersheds (USACE, 1963; RCFCWCD, 1978) is as follows:

$$\text{lag}\ (hours) = C\left[\frac{LL_{CA}}{S^{0.5}}\right]^{0.38}$$

where

C = $24\overline{n}$ = basin factor or correlation coefficient

$\overline{n}$ = "n-bar" = mean hydraulic roughness of all collection streams and channels within a watershed (dimensionless)

L = length of longest watercourse, in *miles*

L_{CA} = length along longest watercourse, measured upstream to a point opposite the centroid of the area, in *miles*

S = overall slope of the longest watercourse between headwaters and the collection point, in *feet per mile*

The unit hydrograph lag parameters required for this transform were determined as follows:

Watercourse lengths. The length of the longest watercourse (L), in *miles*, and the length along the longest watercourse from downstream to a line that intersects the area centroid and longest watercourse

and is perpendicular to the longest watercourse (LCA), in *miles*, were computed for each delineated subbasin based on IFSAR topographic data (Intermap Technologies, 2005).

Representative slope. The representative slope of the longest watercourse (S), in *feet per mile*, was determined for each subbasin by balancing the area above and below a constant slope (representative slope) formed between the longitudinal profile and the constant slope; in addition, representative slopes were determined between each concentration point along the watercourse to support channel flood routing calculations. All slope determinations were based on IFSAR topographic data (Intermap Technologies, 2005)

Basin factor. The basin factor (*C*) was determine for each subbasin based on the landform composition consisting of some combination of Travertine/developed areas ($\overline{n}$ = 0.02), piedmont/alluvial surfaces ($\overline{n}$ = 0.03), and mountain/hillslope areas ($\overline{n}$ = 0.05). This is a conservative assumption given that more than 90 percent of the watershed will likely experience shallow flooding less than 0.5 feet in depth. Shallow flooding n-values typically range from 0.05 to 0.3 (USACE, 1997), influenced by gradient, uniformity of the terrain, soil texture, and vegetation. The derived lag hydraulic roughness values for each subbasin are listed in Table 4-7.

S-graph. The Whitewater S-graph was assumed to represent the runoff response within the regional watershed. The Whitewater S-graph is recommended by CVWD for use in the Coachella Valley and is also the adopted Desert S-graph for Riverside County (RCFCWCD, 1978). The Whitewater S-graph was developed by the USACE Los Angeles District by averaging the S-graphs constructed for nine gauged watersheds located in southern California.

The lag parameters and computation is summarized in Table 4-8. The computed lag times were used in conjunction with the Whitewater S-graph in the unit hydrograph transform

Table 4-7. Lag formula hydraulic roughness $\overline{n}$ ("*n*-bar")

conditions	subbasin	DA {acres}	hydaulic rougness "*n*-bar" distribution by percent			composite *n*-bar
			Travertine/ developed areas {0.02}	piedmont/ alluvial surfaces {0.03}	mountain/ hillslope areas {0.05}	
baseline	D411	5,085	0.0	32.1	67.9	0.044
	D410	1,190	0.5	72.5	27.0	0.035
	D421A	1,479	0.0	25.8	74.2	0.045
	D421B	2,168	0.0	75.0	25.0	0.035
	D422	250	0.0	25.0	75.0	0.045
	D423	756	0.0	25.0	75.0	0.045
	D424	232	0.0	27.9	72.1	0.044
	D431	3,224	0.0	32.5	67.5	0.043
	D432	184	0.0	92.4	7.6	0.032
	D430	999	0.0	98.4	1.6	0.030
	D44	487	0.0	43.8	56.2	0.041
	D45	300	0.0	45.0	55.0	0.041
project	D410	1,235	4.2	66.7	29.1	0.036
	D420	928	16.6	77.0	6.5	0.030

Table 4-8. Unit hydrograph lag parameter summary

conditions	subbasin	drainage area		USACE lag equation parameters					lag {hours}
		{acres}	{sq mi}	L {miles}	LCA {miles}	S {ft/mi}	"*n*-bar"	basin factor C	
baseline	D411	5,085	7.945	5.49	2.86	800.7	0.044	1.06	0.845
	D410	1,190	1.859	2.79	1.13	289.6	0.035	0.84	0.442
	D421A	1,479	2.311	4.27	1.82	1,056.0	0.045	1.07	0.6194
	D421B	2,168	3.388	5.23	3.61	843.8	0.035	0.84	0.713
	D422	250	0.391	1.49	0.78	431.0	0.045	1.08	0.361
	D423	756	1.181	4.02	2.63	870.8	0.045	1.08	0.731
	D424	232	0.363	1.50	0.82	529.9	0.044	1.06	0.346
	D420	974	1.522	2.23	0.84	298.1	0.032	0.77	0.330
	D431	3,224	5.038	5.18	2.25	503.2	0.043	1.03	0.805
	D432	184	0.288	1.31	0.63	799.4	0.032	0.77	0.200
	D430	999	1.561	2.03	0.28	373.2	0.030	0.72	0.189
	D44	487	0.761	2.01	0.97	518.6	0.041	0.98	0.387
	D45	300	0.469	1.37	0.57	680.7	0.041	0.98	0.260
project	D410	1,235	1.930	2.79	1.55	289.6	0.036	0.86	0.513
	D420	928	1.450	2.23	0.83	298.1	0.030	0.72	0.308

4.4 Channel routing

Reaches were defined for conveyances collecting flow from more than one subbasin, as shown in Figure 4-8 (baseline conditions) and Figure 4-9 (project conditions). Beyond the length and slope of the channel, there is little information available to parameterize in-channel flow. The Muskingum routing approach was selected because it is widely used, requires only two parameters that can be reasonably estimated with limited data, and has been used in previous studies in the Coachella Valley (NHC, 2014; Bechtel, 1997; SLA, 2000).

The Muskingum *K*-parameter was estimated from the channel length divided by the flow velocity, estimated using the Manning's formula; the width and depth of flow was approximated using stable alluvial channel relationships; the longitudinal slope was estimated from IFSAR and the hydraulic roughness from aerial photographic imagery.

A value of 0.15 was used for the Muskingum *x*-parameter, given the predominant non-riverine and uncertain nature of the flood processes, which are expected on the piedmont

The Muskingum routing parameters are tabulated in Table 4-9. The hydrologic routing processes are not expected to influence peak flow rates or runoff volumes resulting from the computation of the flood hydrographs.

Figure 4-8. Baseline conditions HEC-HMS model schematic*

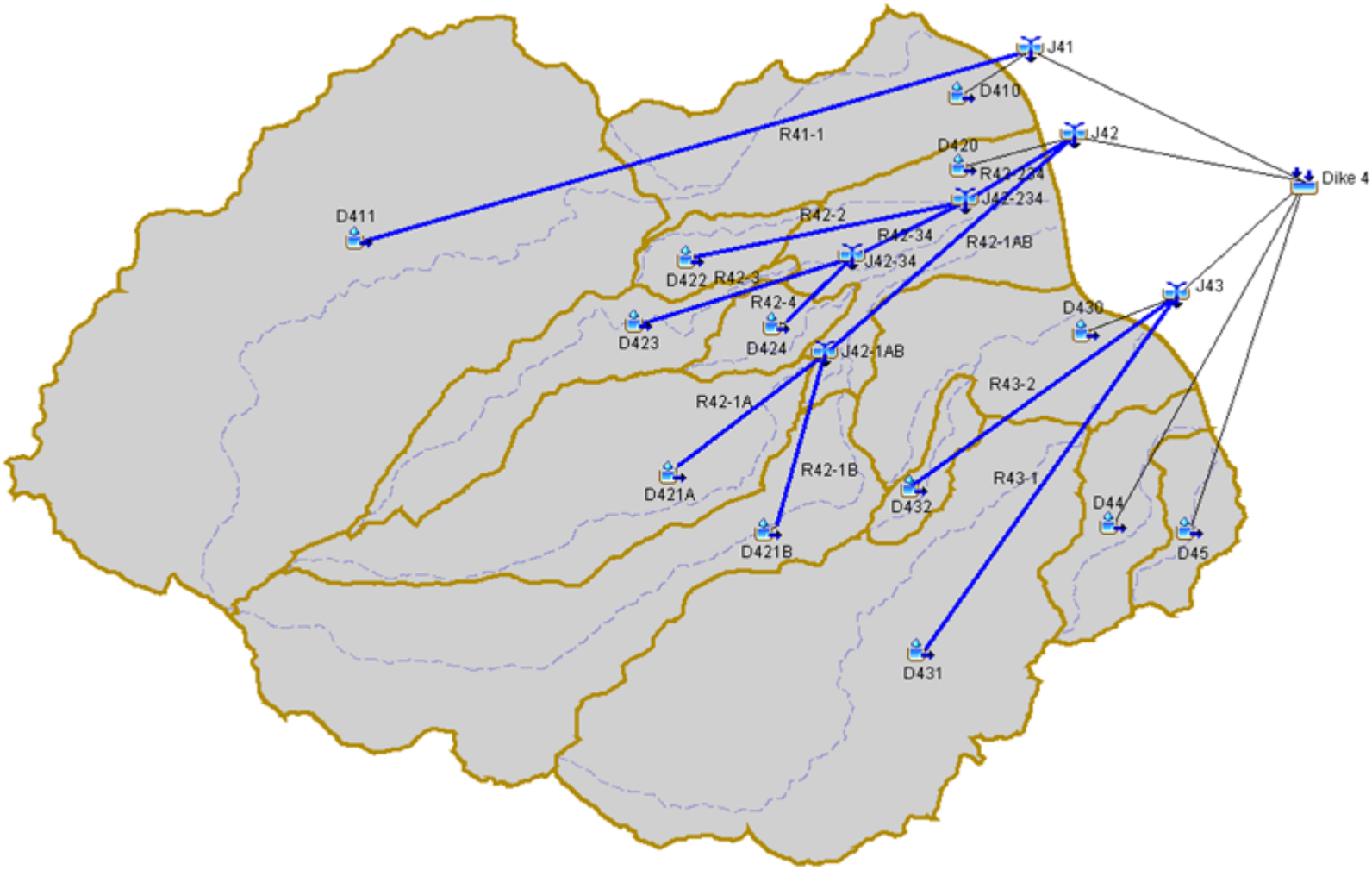


Figure 4-9. Project conditions HEC-HMS model schematic*

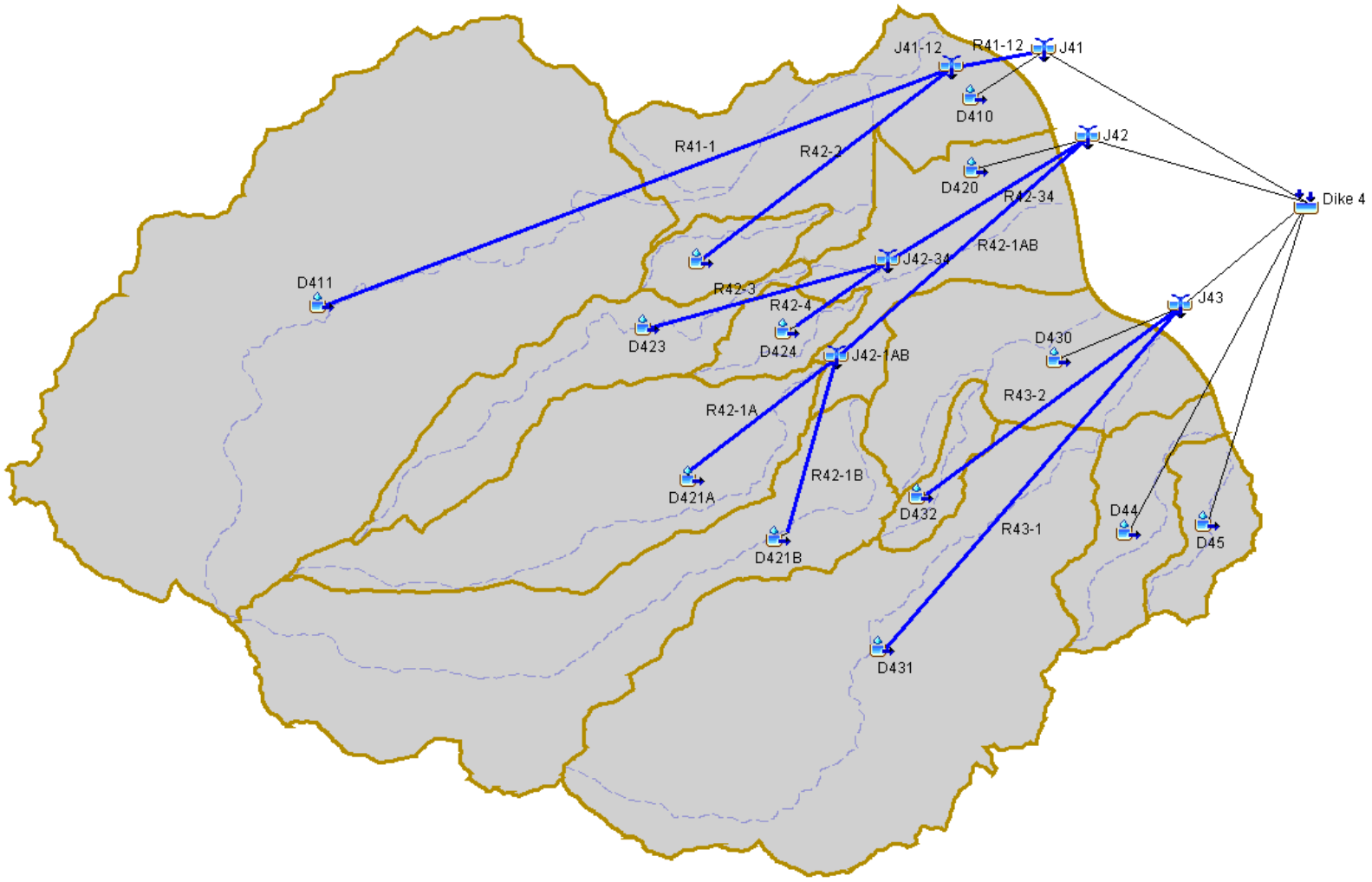


Table 4-9. Muskingum routing parameter development summary

conditions	routing ID	stable alluvial channel relationships			length {feet}	n	S {ft/ft}	A {sq ft}	P {feet}	R {feet}	V {fps}	Muskingum parameters		
		Q_{100} {cfs}	width {feet}	depth {feet}								K {hours}	x	no. of subreaches
baseline	R41-1	10,512	386	2.84	12,013	0.045	0.047	1,097	392	2.80	14.27	0.23	0.15	2.8
	R42-1A	3,418	246	1.81	1,971	0.045	0.060	447	250	1.79	11.93	0.05	0.15	0.6
	R42-1B	4,577	277	2.04	2,438	0.045	0.079	564	281	2.01	14.85	0.05	0.15	0.5
	R42-1AB	7,795	342	2.52	7,727	0.045	0.044	864	347	2.49	12.67	0.17	0.15	2.0
	R42-2	613	124	0.91	6,436	0.045	0.042	113	126	0.90	6.31	0.28	0.15	3.4
	R42-3	1,728	187	1.38	3,627	0.045	0.048	259	190	1.36	8.86	0.11	0.15	1.4
	R42-4	534	117	0.86	1,924	0.045	0.041	101	119	0.85	6.03	0.09	0.15	1.1
	R42-34	1,957	197	1.45	4,100	0.045	0.039	286	200	1.43	8.33	0.14	0.15	1.6
	R42-234	2,361	212	1.56	1,278	0.045	0.056	332	215	1.54	10.43	0.03	0.15	0.4
	R43-1	5,349	294	2.17	3,776	0.045	0.036	639	299	2.14	10.37	0.10	0.15	1.2
	R43-2	552	119	0.87	5,115	0.045	0.029	104	120	0.86	5.12	0.28	0.15	3.3
project	R41-1	10,512	386	2.84	8,787	0.045	0.041	1,097	392	2.80	13.36	0.18	0.15	2.2
	R41-12	10,512	386	2.84	3,226	0.045	0.047	1,097	392	2.80	14.22	0.06	0.15	0.8
	R42-2	613	124	0.91	5,862	0.045	0.015	113	126	0.90	3.74	0.44	0.15	5.2
	R42-3	1,728	187	1.38	2,747	0.045	0.039	259	190	1.36	8.02	0.10	0.15	1.1
	R42-4	534	117	0.86	638	0.045	0.038	101	119	0.85	5.80	0.03	0.15	0.4
	R42-34	1,957	197	1.45	6,450	0.045	0.042	286	200	1.43	8.58	0.21	0.15	2.5

4.5 HEC-HMS model development summary

A summary of model development using HEC-HMS V4.2 (USACE, 2016) is as follows:

- The watershed, subbasins, streams, and subbasin centroids were determined from 5-meter IFSAR topographic data (Intermap Technologies, 2005).
- Subbasins and routing were defined and linked to accommodate selected concentration points at major canyon outlets and along the depression at the base of Dike No. 4
- The model schematic constructed for the Dike No. 4 watershed is presented in Figure 4-8 (baseline conditions) and Figure 4-9 (project conditions).
- Precipitation for the 1-percent annual chance storm was represented as a hypothetical storm using the frequency storm method. Depth-areal reduction factors were applied externally to precipitation depths prior to their implementation in HEC-HMS. The drainage area for the frequency storm method was assigned a value of 0.1 to prevent HEC-HMS from applying an additional depth-areal reduction factor. Also, the peak of the storm synthesized storm pattern was defined to occur two thirds into the storm period.
- The Standard Project Storm was defined as a user-specified hyetograph linked to the 6-hour storm pattern based on the Indio Storm of September 24, 1939 (Plate E-5.9; RCFC&WCD, 1978) defined as an incremental time series.
- The Whitewater S-graph was defined as a paired-data percentage curve and linked as a user-specified S-graph for the unit hydrograph transformation in conjunction with the lag parameters and coefficients
- Channel routing was performed using the Muskingum method based on hydraulic parameters generalized from topographic mapping and aerial photographic imagery.

Data and worksheets used to parameterize the hydrologic models, including the models themselves, are presented in the Electronic Technical Appendix.

4.6 Debris yield

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

As defined in this method, the "total debris yield" is the total debris outflow from a watershed measurable at a specific concentration point for a specified event. It may include clay, silt, sand, gravel, boulders, tree stumps, and other organic materials. The "debris production" is the gross erosion within a watershed while the "debris yield" is the quantity of debris actually delivered to a concentration point of interest. The entire debris production of the watershed may not necessarily reach its outlet because it is stored temporarily within the watershed due to the lack of transporting capacity of the conveyance system.

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

Equation 2 is usually applied to drainages 3 to 10 square miles in area. Equation 1, which is a function of precipitation rather than runoff, is used for basins 0.1 to 3; however, if frequency discharge information is available, Equation 2 may be used for areas less than 3 square miles (USACE, 2002).

Equation 2 was applied herein to the drainages of interest less than 3 square miles in size since frequency discharge information was available; thus, Equation 2 was applied to every subbasin:

$$\text{Equation 2: } \log DY = 0.85\log Q + 0.53\log RR + 0.04\log A + 0.22FF$$

where

DY = unit debris yield (yd^3/mi^2),

RR = relief ratio (feet/mile),

A = drainage area (acres),

FF = non-dimensional fire factor, and

Q = unit peak flow (cfs/mi^2)

Limitations. The general limitations related to the applications of the USACE Los Angeles District Debris Method in the prediction of debris yield are as follows: (1) geographic constraints, (2) drainage area constraints, (3) topographic constraints, (4) frequency constraints, and (5) input constraints. The frequency and input constraints pertain to small events less than 20-percent annual chance and low runoff or precipitation. Since the recurrence interval in this study is 100-year, only the geographic, drainage area and topographic constraints remain. This method is intended to be used for the estimation of debris yield mainly from coastal-draining mountainous watersheds located in southern California. Since the predictive equations were derived from data observed in the San Gabriel Range, the use of these equations for watershed conditions different from those of the San Gabriel Range must be specifically addressed. The method is applicable only to watersheds with areas ranging from 0.1 to 200 square miles and with a high proportion of their total area in steep, mountainous terrain. The use of this method to compute debris yields for watersheds in mild-sloped valley areas with a high percentage of piedmonts and alluvial fans or valley fill areas may result in estimates that are higher than actual yield. If the sediment transport capacity is less than the statistical debris method results, the sediment transport capacity governs the debris yield.

Adjustment-Transposition (A-T) factor. The use of predictive equations developed from data pertaining to watersheds, which historically demonstrate extremely high unit yields will result in overestimates of debris yields when applied to areas with less erosional activity. Recognizing this limitation, and the importance of uncertain geomorphic and geologic parameters, the USACE Los Angeles District developed an Adjustment-Transposition (A-T) factor.

Since there are no debris or sediment records available for the Dike No. 4 watershed or nearby watersheds, the USACE Los Angeles District suggests using Technique 4 (USACE, 2002) to estimate the A-T factor. Technique 4, describes a method to determine the Adjustment-Transposition factor based on four basin parameters: (1) parent material or surficial geology, (2) soils, (3) channel morphology, and (4) hillslope geomorphology. A numerical factor ranging from 0.05 to 0.25 is assigned to each of these parameters according to the characteristics of each of these parameters.

Guidelines were developed (Table D-1; USACE, 2002) to aid in the selection of these values. The guidelines are also shown in Table 4-10. The A-T factor is equal to the sum of the individually assigned numerical values for the four the A-T subfactor groups.

Observations that contributed to the basis for the A-T factor selection are summarized below:

- **Parent material**. The influence of folding, faulting, and fracturing on sediment production and delivery was considered most severe on the steeper slopes of the Santa Rosa Mountains and insignificant on the milder sloped alluvial surfaces. Weathering is sporadic, primarily a function of chemical, thermal, and wind processes, and the infrequent and highly episodic nature of high intensity rainfall. Overall, the contribution from weathering is minor relative to other parental material factors.
- **Soils**. The influence of cohesion and clay colloids is considered increasingly more significant on more developed portions of the piedmont and less so on the steep rocky canyon slopes of the Santa Rosa Mountains. Active alluvial surfaces are not expected to be affected. The soil profile was viewed as being most developed on the older surfaces of the piedmont, moderately developed in areas on the slopes of the Santa Rosa Mountains, and minimally developed on the

more active alluvial surfaces. Exposed rock can be seen as emulating the same behavior in that it is resistant to erosion, but more due to its physical continuity.

- **Channel morphology**. Bedrock exposures and bank erosion are expected to have some contribution along the edges of confined canyon beds and the more developed portions of the piedmont. Vegetation is generally limited throughout and there is no significant evidence of headcutting observed in the watershed; bed and bank materials are generally non-cohesive on active portions of the piedmont and partially cohesive on the steeper slopes of the Santa Rosa Mountains, and most significant along the edges of the more developed portions of the piedmont.
- **Hillslope morphology**. This subfactor group has little influence on the production and delivery of sediment and debris within the watershed. There is no significant evidence of active rilling, gulling, and mass movement. There are limited eroding deposits in the confined channel reaches.

A breakdown of A-T factors by soil map unit is shown in Table 4-11. The A-T factor computation for each subbasin is presented in Table 4-12. The resultant A-T factors estimated for the each subbasin ranges from 0.50 to 0.57.

Due to the low risk of wildfires occurring in this region due to sparse vegetation, the Fire Factor (FF) used in the analysis of each subbasin was assigned a minimum value of 3.0 based on fire factor (Tables A-1 and A-2; USACE, 2000).

Table 4-10. Los Angeles District Debris Method (Table D-1; USACE, 2000) A-T factor guidelines

subfactor group	group parameter	A-T subfactor				
		0.25	0.20	0.15	0.10	0.05
parent material	folding	severe		moderate		minor
	faulting	severe		moderate		minor
	fracturing	severe		moderate		minor
	weathering	severe		moderate		minor
soils	cohesion	non-cohesive		partly cohesive		highly cohesive
	profile	minimum soil profile		some soil profile		well-developed soil profile
	cover	much bare soil in evidence		some bare soil in evidence		little bare soil in evidence
	clay colloids	few clay colloids		some clay colloids		many clay colloids
channel morphology	bedrock exposures	few segments in bedrock		some segments in bedrock		many segments in bedrock
	bank erosion	> 30% of banks eroding		10 - 30% of banks eroding		< 10% of banks erodings
	bed and bank materials	non-cohesive bed and banks		partly cohesive bed and banks		mildy cohesive bed and banks
	vegetation	poorly vegetated		some vegetation		much vegetation
	headcutting	many headcuts		few headcuts		no headcutting
hillslope erosion	rills and gullies	many and active		some signs		few signs
	mass movement	many scars evident		few signs evident		no signs evident
	debris deposits	many eroding deposits		some eroding deposits		few eroding deposits

Table 4-11. A-T factors estimated for each soil map unit

NRCS soil identifier				parent material					soils					channel morphology						hillslope erosion				
survey area	MUSYM	DA {acres}	soil RTIMP {%}	folding	faulting	fracturing	weathering	A-T subfactor	cohesion	profile	cover	clay colloids	A-T subfactor	bedrock exposures	bank erosion	bed and bank materials	vegetation	headcutting	A-T subfactor	rills and gullies	mass movement	debris deposits	A-T subfactor	A-T factor
CA680	CcC	39	0	0.05	0.04	0.05	0.05	0.05	0.25	0.15	0.25	0.25	0.23	0.25	0.05	0.25	0.25	0.05	0.17	0.10	0.05	0.25	0.13	0.58
	CdC	550	0	0.05	0.04	0.05	0.05	0.05	0.25	0.15	0.25	0.25	0.23	0.25	0.05	0.25	0.25	0.05	0.17	0.10	0.05	0.25	0.13	0.58
	ChC	177	0	0.05	0.04	0.05	0.05	0.05	0.25	0.15	0.25	0.25	0.23	0.25	0.05	0.25	0.25	0.05	0.17	0.10	0.05	0.25	0.13	0.58
	GbA	19	0	0.05	0.04	0.05	0.05	0.05	0.17	0.15	0.25	0.17	0.19	0.25	0.05	0.17	0.25	0.05	0.15	0.10	0.05	0.25	0.13	0.52
	GbB	2	0	0.05	0.04	0.05	0.05	0.05	0.17	0.15	0.25	0.17	0.19	0.25	0.05	0.17	0.25	0.05	0.15	0.10	0.05	0.25	0.13	0.52
	GP	0	0	0.05	0.04	0.05	0.05	0.05	0.05	0.15	0.00	0.05	0.06	0.25	0.00	0.05	0.00	0.00	0.06	0.10	0.00	0.25	0.12	0.29
	Ip	17	0	0.05	0.04	0.05	0.05	0.05	0.24	0.15	0.25	0.24	0.22	0.25	0.05	0.24	0.25	0.05	0.17	0.10	0.05	0.25	0.13	0.57
	Is	4	0	0.05	0.04	0.05	0.05	0.05	0.24	0.15	0.25	0.24	0.22	0.25	0.05	0.24	0.25	0.05	0.17	0.10	0.05	0.25	0.13	0.57
	LR	7	20	0.09	0.07	0.09	0.09	0.08	0.20	0.13	0.25	0.20	0.20	0.21	0.05	0.20	0.25	0.05	0.15	0.09	0.05	0.21	0.12	0.55
	MaB	184	0	0.05	0.04	0.05	0.05	0.05	0.25	0.15	0.25	0.25	0.23	0.25	0.05	0.25	0.25	0.05	0.17	0.10	0.05	0.25	0.13	0.58
	RA	0	0	0.05	0.04	0.05	0.05	0.05	0.25	0.15	0.25	0.25	0.23	0.25	0.05	0.25	0.25	0.05	0.17	0.10	0.05	0.25	0.13	0.58
	RO	78	90	0.23	0.17	0.23	0.23	0.22	0.07	0.06	0.25	0.07	0.11	0.07	0.05	0.07	0.25	0.05	0.10	0.06	0.05	0.07	0.06	0.48
	RU	79	0	0.05	0.04	0.05	0.05	0.05	0.25	0.15	0.25	0.25	0.23	0.25	0.05	0.25	0.25	0.05	0.17	0.10	0.05	0.25	0.13	0.58
	W	0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CA777	DnE	74	0	0.05	0.04	0.05	0.05	0.05	0.25	0.15	0.25	0.25	0.23	0.25	0.05	0.25	0.25	0.05	0.17	0.10	0.05	0.25	0.13	0.58
	LcF	19	25	0.10	0.08	0.10	0.10	0.09	0.20	0.13	0.25	0.20	0.19	0.20	0.05	0.20	0.25	0.05	0.15	0.09	0.05	0.20	0.11	0.55
	Rs	905	85	0.22	0.17	0.22	0.22	0.21	0.08	0.07	0.25	0.08	0.12	0.08	0.05	0.08	0.25	0.05	0.10	0.06	0.05	0.08	0.06	0.49
US	s991	3,661	0	0.05	0.04	0.05	0.05	0.05	0.25	0.15	0.25	0.25	0.23	0.25	0.05	0.25	0.25	0.05	0.17	0.10	0.05	0.25	0.13	0.58
	s1016	1,458	19	0.09	0.07	0.09	0.09	0.08	0.19	0.13	0.25	0.19	0.19	0.21	0.05	0.19	0.25	0.05	0.15	0.09	0.05	0.21	0.12	0.54
	s1021	10,053	75	0.20	0.15	0.20	0.20	0.19	0.10	0.08	0.25	0.10	0.13	0.10	0.05	0.10	0.25	0.05	0.11	0.06	0.05	0.10	0.07	0.50

Table 4-12. Summary of A-T factors estimated for each subbasin

subfactor group	group parameter	subbasin														
		baseline conditions													project conditions	
		D411	D410	D421A	D421B	D422	D423	D424	D420	D431	D432	D430	D44	D45	D410	D420
parent material	folding	0.17	0.10	0.20	0.10	0.20	0.20	0.19	0.08	0.18	0.07	0.05	0.16	0.15	0.11	0.07
	faulting	0.13	0.08	0.15	0.08	0.15	0.15	0.15	0.06	0.14	0.05	0.04	0.12	0.12	0.08	0.05
	fracturing	0.17	0.10	0.20	0.10	0.20	0.20	0.19	0.08	0.18	0.07	0.05	0.16	0.15	0.11	0.07
	weathering	0.17	0.10	0.20	0.10	0.20	0.20	0.19	0.08	0.18	0.07	0.05	0.16	0.15	0.11	0.07
	A-T subfactor	0.16	0.10	0.19	0.09	0.19	0.19	0.18	0.07	0.17	0.06	0.05	0.15	0.14	0.11	0.06
soils	cohesion	0.13	0.20	0.10	0.20	0.10	0.10	0.11	0.22	0.12	0.23	0.25	0.14	0.15	0.19	0.23
	profile	0.09	0.12	0.08	0.12	0.08	0.08	0.08	0.14	0.08	0.14	0.15	0.09	0.10	0.12	0.14
	cover	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25
	clay colloids	0.13	0.20	0.10	0.20	0.10	0.10	0.11	0.22	0.12	0.23	0.25	0.14	0.15	0.19	0.23
	A-T subfactor	0.15	0.19	0.13	0.19	0.13	0.13	0.13	0.21	0.14	0.22	0.22	0.15	0.16	0.19	0.21
channel morphology	bedrock exposures	0.13	0.20	0.10	0.20	0.10	0.10	0.11	0.22	0.12	0.23	0.25	0.14	0.15	0.19	0.23
	bank erosion	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05
	bed and bank materials	0.13	0.20	0.10	0.20	0.10	0.10	0.11	0.22	0.12	0.23	0.25	0.14	0.15	0.19	0.23
	vegetation	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25
	headcutting	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.30	0.05	0.05	0.05
	A-T subfactor	0.12	0.15	0.11	0.15	0.11	0.11	0.11	0.16	0.12	0.16	0.17	0.18	0.13	0.15	0.16
hillslope erosion	rills and gullies	0.07	0.09	0.06	0.09	0.06	0.06	0.06	0.09	0.07	0.10	0.10	0.07	0.07	0.08	0.10
	mass movement	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05
	debris deposits	0.13	0.20	0.10	0.20	0.10	0.10	0.11	0.22	0.12	0.23	0.25	0.14	0.15	0.19	0.23
	A-T subfactor	0.08	0.11	0.07	0.11	0.07	0.07	0.07	0.12	0.08	0.13	0.13	0.09	0.09	0.11	0.13
A-T factor		0.51	0.55	0.50	0.55	0.50	0.50	0.50	0.56	0.51	0.57	0.57	0.57	0.52	0.54	0.57

4.7 Hydrologic analysis results

All hydrologic models were analyzed using a 24-hour simulation time to ensure the flood hydrograph recessions and total runoff volumes for the 6-hour storm are reported.

The 1-percent annual chance hydrologic model and debris analysis results focused on conveyance and temporary impoundment along the project edge and near vicinity, including the Guadalupe Creek Dike Diversions are summarized in Table 4-13. The contributing drainages for each area of interest is less than 10 square miles; therefore, areal effects were not applied. Flood hydrographs were developed and debris analysis performed for each individual subbasin for each set of conditions (baseline and project). The subbasins associated with hydrologic nodes D410 and D420 are the only drainages, which change characteristics going from baseline to project conditions.

The 1-percent annual chance and SPS/SPF hydrology and debris analysis results focused on the impoundment of floodwaters along the base of Dike No. 4 are summarized in Table 4-14 and Table 4-15, respectively. Areal effects were based on the entire watershed tributary to Dike No. 4 (27.07 square miles) contributing. Flood hydrographs were developed and debris analysis performed for each individual subbasin for each set of conditions (baseline and project).

Table 4-13. 1-percent annual chance hydrology and debris analysis results for the evaluation of the project edge conditions

conditions	subbasin	drainage area {acres}	relief ratio {ft/mi}	fire factor	A-T factor	Q_p {cfs}	flood volume {ac-ft}	q_p {cfs/mi^2}	debris yield {ac-ft}	bulked volume {ac-ft}	bulking factor	$Q_{p,bulked}$ {cfs}
baseline	D411 {Devil Canyon}	5,085	800.7	3.0	0.51	10,505	1,192	1,322	251	1,443	1.21	12,720
	D410	1,190	289.6	3.0	0.55	2,032	137	1,093	30	167	1.22	2,473
	J41	6,275	-	-	-	10,413	1,329	1,062	281	1,610	1.21	12,615
	D421A {Middle Canyon South}	1,479	1117.9	3.0	0.50	3,420	336	1,480	90	425	1.27	4,332
	D421B {Rock Avalanche Canyon}	2,168	843.8	3.0	0.55	4,577	441	1,351	117	558	1.27	5,791
	D422	250	431.0	3.0	0.50	613	45	1,573	9	54	1.20	734
	D423 {Middle Canyon North}	756	870.8	3.0	0.50	1,730	189	1,463	39	227	1.21	2,084
	D424	232	529.9	3.0	0.50	535	39	1,477	9	48	1.22	655
	D420	974	298.1	3.0	0.56	1,958	111	1,286	29	140	1.26	2,462
	J42	5,859	-	-	-	9,955	1,161	1,087	292	1,453	1.25	12,454
	D431 {Toro Canyon}	3,224	503.2	3.0	0.51	5,349	604	1,062	102	705	1.17	6,249
	D432	184	799.4	3.0	0.57	554	24	1,932	12	36	1.51	839
	D430	999	373.2	3.0	0.57	2,574	105	1,649	42	146	1.40	3,597
	J43	4,407	-	-	-	5893	732	856	155	888	1.21	7,143
	D44	487	518.6	3.0	0.57	1,006	70	1,321	19	89	1.28	1,286
	D45	300	680.7	3.0	0.52	719	41	1,536	14	55	1.34	964
	total	17,327	-	-	-	-	3,333	-	761	4,094	-	-
project {changes only}	D410	1,235	289.6	3.0	0.54	2,054	151	1,064	27	178	1.18	2,422
	D422 {moved from J42}	250	431.0	3.0	0.50	613	45	1,573	9	54	1.20	734
	J41	6,570	-	-	-	10,934	1,388	1,065	287	1,675	1.21	13,196
	D420	928	298.1	3.0	0.57	1,997	111	1,377	19	129	1.17	2,335
	J42	5,563	-	-	-	9,269	1,115	1,066	273	1,388	1.24	11,536
	total	17,327	-	-	-	-	3,347	-	749	4,095	-	-

Note: Contributing drainages are less than 10 square miles; therefore, hydrology was not subjected to areal effects

Table 4-14. 1-percent annual chance debris analysis results for the evaluation of flood impoundment adjacent to Dike No. 4

conditions	subbasin	drainage area {acres}	relief ratio {ft/mi}	fire factor	A-T factor	Q_p {cfs}	flood volume {ac-ft}	q_p {cfs/mi^2}	debris yield {ac-ft}	bulked volume {ac-ft}	bulking factor	$Q_{p,bulked}$ {cfs}
baseline	D411 {Devil Canyon}	5,085	800.7	3.0	0.51	9,680	1,078	1,218	234	1,313	1.22	11,784
	D410	1,190	289.6	3.0	0.55	1,872	123	1,007	28	150	1.23	2,293
	J41	6,275	-	-	-	9,564	1,201	975	262	1,463	1.22	11,651
	D421A {Middle Canyon South}	1,479	1117.9	3.0	0.50	3,152	303	1,364	84	387	1.28	4,022
	D421B {Rock Avalanche Canyon}	2,168	843.8	3.0	0.55	4,211	400	1,243	109	509	1.27	5,356
	D422	250	431.0	3.0	0.50	566	41	1,452	8	49	1.20	682
	D423 {Middle Canyon North}	756	870.8	3.0	0.50	1,596	171	1,350	36	207	1.21	1,933
	D424	232	529.9	3.0	0.50	495	35	1,367	8	43	1.23	610
	D420	974	298.1	3.0	0.56	1,798	100	1,182	27	127	1.27	2,278
	J42	5,859	-	-	-	9,141	1,050	999	272	1,322	1.26	11,508
	D431 {Toro Canyon}	3,224	503.2	3.0	0.51	4,923	543	977	95	637	1.17	5,781
	D432	184	799.4	3.0	0.57	510	21	1,777	11	33	1.54	783
	D430	999	373.2	3.0	0.57	2,383	94	1,527	39	133	1.41	3,371
	J43	4,407	-	-	-	5390	658	783	145	803	1.22	6,578
	D44	487	518.6	3.0	0.57	922	63	1,211	18	81	1.29	1,188
	D45	300	680.7	3.0	0.52	663	37	1,417	13	50	1.36	899
	total	17,327	-	-	-	-	3,009	-	710	3,719	-	-
project {changes only}	D410	1,235	289.6	3.0	0.54	1,879	136	974	25	161	1.18	2,226
	D422 {moved from J42}	250	431.0	3.0	0.50	566	41	1,452	8	49	1.20	682
	J41	6,570	-	-	-	10,048	1,255	979	268	1,522	1.21	12,193
	D420	928	298.1	3.0	0.57	1,836	99	1,266	17	116	1.18	2,159
	J42	5,563	-	-	-	8,510	1,008	979	254	1,262	1.25	10,657
	total	17,327	-	-	-	-	3,020	-	698	3,718	-	-

Note: Hydrology subjected to areal effects assuming the entire watershed (27.07 square miles) is contributing

Table 4-15. SPF debris analysis results for the evaluation of flood impoundment adjacent to Dike No. 4

conditions	subbasin	drainage area {acres}	relief ratio {ft/mi}	fire factor	A-T factor	Q_p {cfs}	flood volume {ac-ft}	q_p {cfs/mi^2}	debris yield {ac-ft}	bulked volume {ac-ft}	bulking factor	$Q_{p,bulked}$ {cfs}
baseline	D411 {Devil Canyon}	5,085	800.7	3.0	0.51	9,031	1,882	1,137	221	2,103	1.12	10,092
	D410	1,190	289.6	3.0	0.55	2,550	384	1,372	36	420	1.09	2,789
	J41	6,275	-	-	-	9,766	2,266	996	257	2,523	1.11	10,873
	D421A {Middle Canyon South}	1,479	1117.9	3.0	0.50	3,020	559	1,307	81	640	1.14	3,456
	D421B {Rock Avalanche Canyon}	2,168	843.8	3.0	0.55	3,844	684	1,135	101	785	1.15	4,410
	D422	250	431.0	3.0	0.50	609	95	1,562	9	103	1.09	667
	D423 {Middle Canyon North}	756	870.8	3.0	0.50	1,439	287	1,217	33	320	1.12	1,605
	D424	232	529.9	3.0	0.50	571	87	1,578	9	96	1.11	632
	D420	974	298.1	3.0	0.56	2,277	296	1,496	33	328	1.11	2,527
	J42	5,859	-	-	-	9588	2,007	1,047	265	2,272	1.13	10,855
	D431 {Toro Canyon}	3,224	503.2	3.0	0.51	5,869	1,192	1,165	110	1,302	1.09	6,410
	D432	184	799.4	3.0	0.57	491	54	1,710	11	65	1.20	591
	D430	999	373.2	3.0	0.57	2,685	288	1,720	43	331	1.15	3,088
	J43	4,407	-	-	-	7528	1,533	1,093	164	1,697	1.11	8,334
	D44	487	518.6	3.0	0.57	1,137	173	1,493	22	195	1.12	1,278
	D45	300	680.7	3.0	0.52	783	107	1,673	15	122	1.14	893
	total	17,327	-	-	-	-	6,087	-	723	6,810	-	-
project {changes only}	D410	1,235	289.6	3.0	0.54	2,563	411	1,328	33	444	1.08	2,766
	D422 {moved from J42}	250	431.0	3.0	0.50	609	95	1,562	9	103	1.09	667
	J41	6,570	-	-	-	10,366	2,387	1,010	262	2,650	1.11	11,506
	D420	928	298.1	3.0	0.57	2,248	303	1,551	21	324	1.07	2,402
	J42	5,563	-	-	-	8,776	1,920	1,010	244	2,164	1.13	9,943
	total	17,327	-	-	-	-	6,121	-	708	6,829	-	-

Note: Hydrology subjected to areal effects assuming the entire watershed (27.07 square miles) is contributing

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5 FLOOD CONVEYANCE AND STORAGE ANALYSIS

The proposed Travertine Development is situated on the piedmont, which skirts along the eastern base of the Santa Rosa Mountains. The upper piedmont is comprised of a number of active fan areas stemming from the canyons above. The active fan areas associated with the larger collective of canyons referred to as Devil, Middle, and Toro Canyon Areas, are physically segregated by areas of significant soil development, identified as inactive fan areas, except on the more distal portions of the piedmont.

The piedmont experiences a complex mix of flood processes, which can be generally categorized as shallow "non-riverine" flooding, except within the impoundment area along Dike No. 4, where deep ponding can occur during more extreme events. Shallow flooding is typically more confined at or near the canyon outfalls and transitions to flood processes of a distributary nature followed by sheet flooding on the more distal portions of the piedmont. Past agricultural activities have resulted in areas elevated on fill, which cause a significant disruption in the natural flood processes on the northern section of the piedmont.

Evaluating flood conveyance on the piedmont surface and flood storage along the upstream side of Dike No. 4 are the key focal points of this study as these two flood-related phenomena are expected to influence (1) site planning and design in terms of providing sustainable flood protection around and/or through the development footprint and (2) mitigation of flood hazard impacts to adjacent properties and facilities, including the Guadalupe Creek Diversion Dikes, Dike No. 4, and the deep aquifer recharge basins, as a consequence of the planned development.

Given the nature of the flood environment on the piedmont, a two-dimensional flood routing model was selected in lieu of a more conventional one-dimensional hydraulic modeling scheme in order to avoid over-simplifying the complexities that are typical of such an environment. In addition, a two-dimensional flood routing model provides an improved measure for evaluating the transient behavior of the water surface profile along Dike No. 4 caused by the unbalanced delivery of floodwaters, as compared to conventional level-pool storage analysis, which might underestimate the maximum water surface at some locations along Dike No. 4.

Floodwaters temporarily impounded by Dike No. 4 typically percolate into the ground, but during extreme flood stages, may discharge to a functioning evacuation channel maintained by CVWD via an overflow outlet structure.

5.1 Goals and objectives

The primary goal of conducting two-dimensional flood routing is to approximate existing (baseline) and project-influenced flood patterns to use as a base for future development planning; it is not intended as a regulatory floodplain evaluation.

Objectives including: (1) spatial mapping of 1-percent annual chance flood depths, velocities, and flow rate distributions above, around, within, and below the Travertine property limits; (2) computing the 1-percent annual chance flood and Standard Project Flood maximum water surface elevation profiles along Dike No. 4.

5.2 Two-dimensional flood routing

The bulked (sediment/debris loaded) 1-percent annual chance flood and Standard Project Flood were used to determine the freeboard along Dike No. 4 as well as estimate the behavior of flood conveyance on the piedmont based on the IFSAR topographic dataset (Intermap Technologies, 2005) using FLO-2D PRO (Flo-2d, Inc., 2013), a two-dimensional, finite-difference scheme, flood routing model. The FLO-2D PRO model development includes the following aspects:

- General model definitions
- Topographic features
- Levees
- Hydraulic structures
- Infiltration and transmission losses
- Inflow boundary conditions

5.2.1 General model definitions

The model domain was defined to include the piedmont confined by Dike No. 4 to the east, the mountain front to the west and south, and the Guadalupe Creek Diversion Dikes and rock outcroppings to the north. The following information describes the general model definition:

- Domain of 101,833 grid elements
- 24-hour simulation time
- 50' x 50' grid element size
- Grid element elevations were interpolated from IFSAR topographic dataset (Intermap Technologies, 2005)
- A constant floodplain n-value of 0.045 was assigned to the entire domain
- A shallow n-value of 0.100 was assigned to the entire domain
- A limiting Froude number of 0.95 was assigned to the entire domain

5.2.2 Topographic features

The IFSAR topographic dataset (Intermap Technologies, 2005), which was used to determine the grid element elevations across the model domain, captures in some measure the influence of the following anthropogenic (human-made) features/disturbances:

- Guadalupe Creek Diversion Dikes (3 in all)
- Fill slope/berm/channelization along the southern edge of the vineyard

The following human-made features/disturbances are not significantly represented by the model domain elevations:

- Fill slope/berm along the west edge of the vineyard
- Recharge ponds and associated berms
- Rock outcrop cutout area below Guadalupe Creek Diversion Dike Nos. 1 and 2

No significant man-made disturbances were observed (in the field or on aerial imagery) on the piedmont upstream of the Project site that would potentially influence flood patterns.

5.2.3 Levees

The following levee definitions were applied to the model domain:

- *Dike No. 4*. The domain boundary was defined along the top of Dike No. 4 to serve as a levee of infinite height
- *Guadalupe Creek Diversion Dikes*. Levees were defined along each of the three alignments with the assumption that overtopping will not occur for all conditions and all flood events up to and including the Standard Project Flood; the domain boundary was defined for a portion along the top of Guadalupe Creek Diversion Dike No. 1 (north dike) to emulate the leveed conditions
- *Development footprint edge conditions*. A levee was defined around the entire edge of the development footprint to preclude run-on from offsite flows for the Project conditions
- *Avenue 62 and Jefferson Street bridge crossings*. Both bridges were leveed along their upstream and downstream faces to limit flow through the structure via a series of defined hydraulic structures

- *Devil Canyon breakout locations*. A levee was defined to divert floodwaters onto the overbank areas at a selected breakout location

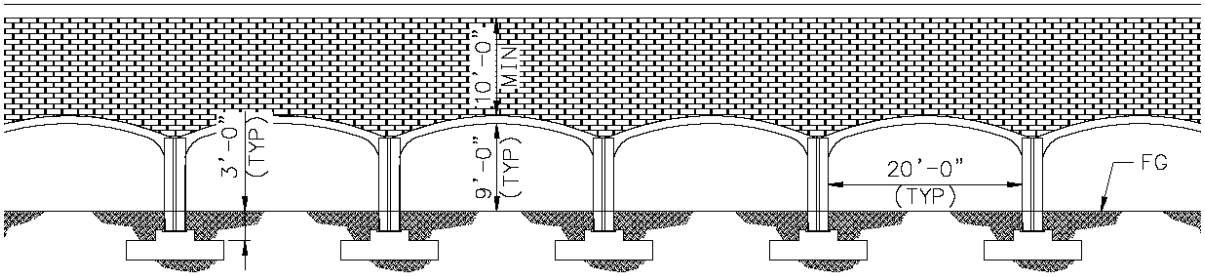
5.2.4 Hydraulic structures

The following hydraulic structures were defined in the model domain:

- *Avenue 62 and Madison bridge crossing*. The proposed Avenue 62 and Madison crossings were modeled using the FHWA culvert equations in the FLO-2D PRO computer program. The crossings were modeled as 8 – 20-ft wide by 8-feet height reinforced concrete box culverts.
- *Jefferson Street bridge crossing*. The proposed Jefferson Street crossing consists of a bridge and 14 arched pier walls with a thickness of 1.33 feet, spaced 21.33 feet from centerline to centerline, and 9 feet from the finish grade to the highest point along the low chord, whose rating curve was approximated using HEC-RAS v4.1 (see Technical Appendix) assuming a constant slope of 0.001 ft/ft and uniform cross sections to represent the channel section; a typical bridge crossing detail is shown in Figure 5-1.

The final design configuration for each proposed bridge crossing is unknown at this time; therefore, the rating curves were conceptually and simplistically approximated based on the assumption the openings, grades, and elevations would be constant along the span of each bridge to allow for the determination and application of a unit width rating curve. Updated hydraulic models should be prepared with the final design of the bridge crossings which shall incorporate the final grades and bridge configuration.

Figure 5-1 Conceptual arched bridge detail section (partial)



5.2.5 Coral Mountain rock cutout at the terminus of upper Guadalupe Creek

A hydraulic rating table was determined for the Coral Mountain rock cutout, which separates the upper and lower segments of Guadalupe Creek, assuming the water surface functions at critical depth despite the steep slopes through this section, which would normally result in supercritical flow.

The hydraulic rating table was constructed from the higher critical depth resulting from the two cross sections for each analyze discharge. In the plan view, cross section one (1) is curvilinear along the 150' elevation (invert) and cross section two (2) is more linear across the cutout where the invert is at the 144' elevation (see Figure 5-2). Critical depths were computed for each cross section using the Hydraulic ToolBox V4.2 (FHWA, 2014).

The resultant critical depths for selected discharges are shown in Table 5-1 with the governing depths highlighted in green. The critical depths for cross section 1 are relative to the 150' elevation whereas the critical depths for cross section 2 are relative to the 144' elevation. The critical depths for cross section 1 were increased 6 feet (150' – 144') so that the critical depths for both cross sections are relative to the same elevation. The section profile for cross section 1 in depicted in Figure 5-3 and the section profile for cross section 2 is presented in Figure 5-4.

Figure 5-2 Selected cross sections for Coral Mountain rock cutout

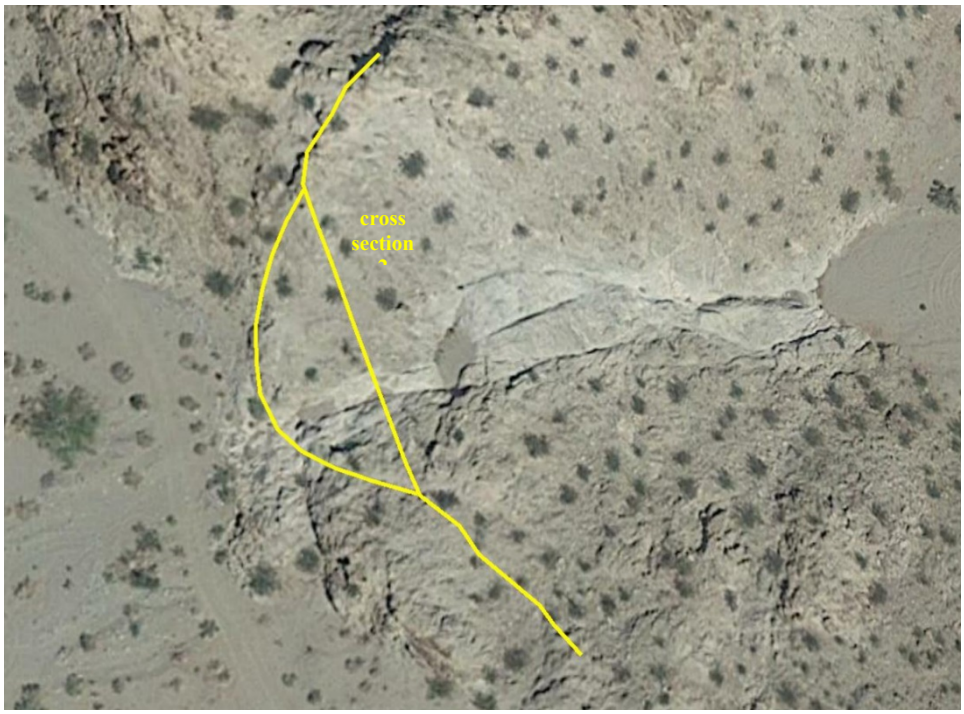


Table 5-1. Summary of discharges versus critical depth at selected rock cutout cross sections

Q {cfs}	critical depth, in feet, relative to invert elevation		
	cross section 1		cross section 2
	EL 150'	EL 144'	EL 144'
10	0.39	6.39	1.23
50	0.85	6.85	2.32
100	1.16	7.16	3.08
500	2.41	8.41	6.16
1,000	3.29	9.29	8.33
5,000	7.32	13.32	15.78
10,000	10.30	16.30	19.33
15,000	12.34	18.34	22.09

Figure 5-3 Cross Section 1 (critical depth shown for 10,000 cfs)

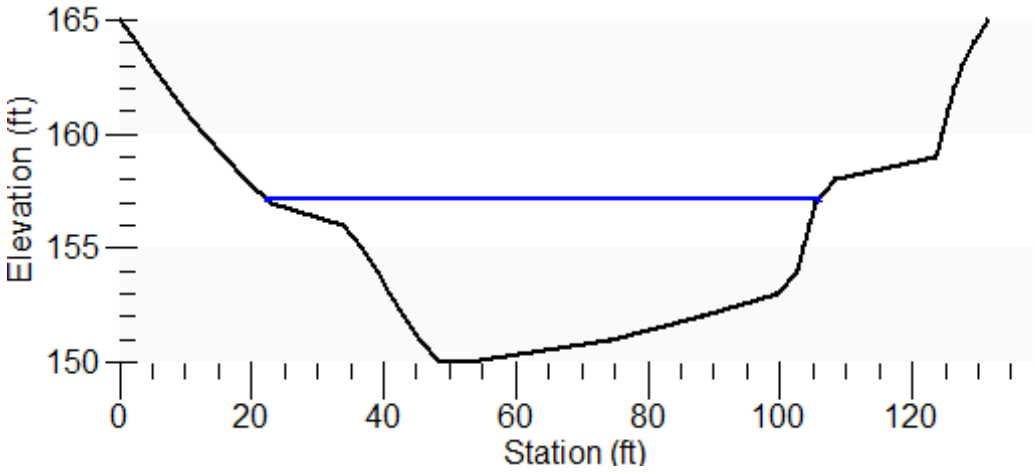
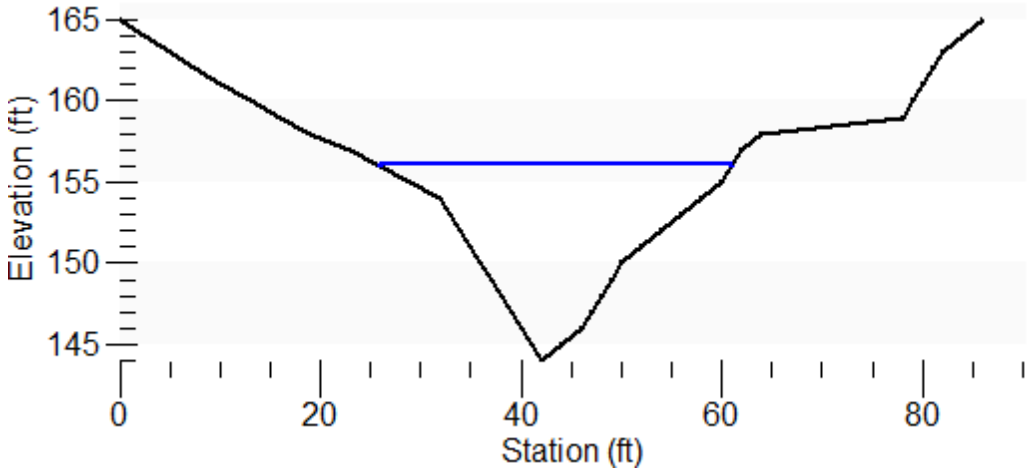


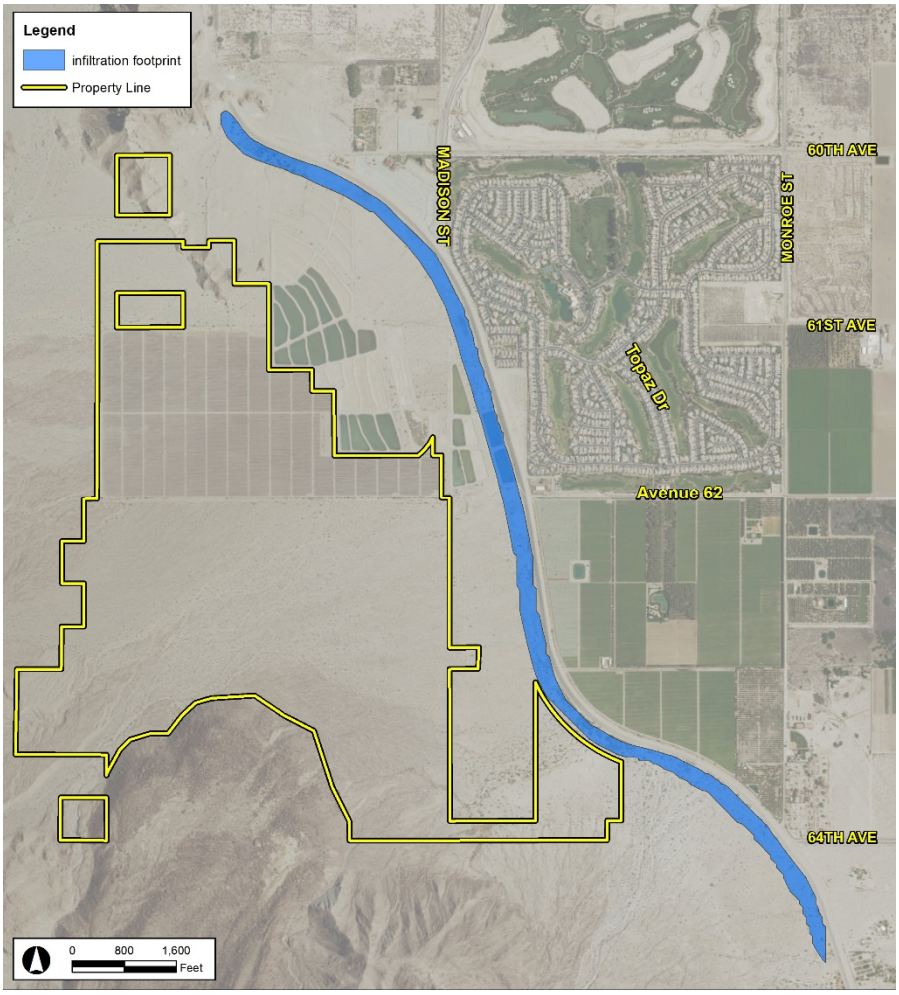
Figure 5-4 Cross Section 2 (critical depth shown for 10,000 cfs)



5.2.6 Infiltration and transmission losses

Infiltration was defined along the base of Dike No. 4 as shown in Figure 5-5, similar to PACE (2005), for the impoundment analysis of the Standard Project Flood only. The Green-Ampt infiltration method was used and parameters were adjusted to produce an average infiltration rate over the 24-hour simulation period that does not exceed the constant infiltration rate of 2.5 inches per hour originally assumed by PACE (2005).

Figure 5-5 infiltration area for model simulations focused on impoundment impacts to Dike No. 4



5.2.7 Model inflow boundary conditions

The definition of the inflow boundary conditions were subjected to combinations of the following elements and there possible variations as summarized in Table 5-2:

- Event: 1-percent annual chance flood or Standard Project flood
- Conditions: baseline or project
- Areal effects

The flood conditions along the project edge and near vicinity, including the Guadalupe Creek Dike Diversion System, were only analyzed for the 1-percent annual chance flood; and because the contributing drainage for anyone contiguous area or alignment of interest was less than 10 square miles, areal effects were not applied.

The flood conditions along Dike No.4 were analyzed assuming the entire watershed (27.07 square miles) is contributing; and therefore, areal effects were used to adjust precipitation depths. The 1-percent annual chance flood and Standard Project Flood were both analyzed to determine whether or not freeboard requirements are met.

For each model simulation focus, the baseline and project conditions were analyzed to facilitate the determination of impacts and related mitigation.

Table 5-2. Summary of inflow boundary conditions

inflow boundary conditions scenario	development conditions	simulation focus	flood event	areal effects
1	baseline	project edge and near vicinity	1-percent annual chance flood	contributing drainages < 10 sq mi
2	project			
3	baseline	Dike No. 4		entire watershed {27.07 sq mi} contributing >> 10 sq mi
4	project			
5	baseline		Standard Project Flood	
6	project			

Inflow hydrographs were defined at or near the canyon outfalls outside the Travertine property boundary. The definition of these inflow hydrographs are based on the translation of the combined hydrographs at hydrologic nodes J41, J42, J43 to the nodes located at or near the canyon outfalls (D411, D421A, D421B, D422, D423, D424, D431, D432, D441, and D451). D441 and D451 were not defined in the hydrologic model, but were added to identify the inflow locations representing hydrologic nodes D44 and D45.

The distribution of hydrographs were weighted by volume and include the runoff developed on the piedmont (D410, D420, and D430).

Inflow hydrographs were defined at or near each canyon outfall as follows:

- Baseline conditions only
  - The combined flood hydrograph at node J41 was assigned to node D411
  - The combined flood hydrograph at node J42 was proportioned to nodes D421A, D421B, D422, D423, and D424
- Project conditions only
  - The combined flood hydrograph at node J41 was proportioned to nodes D411 and D422
  - The combined flood hydrograph at node J42 was proportioned to nodes D421A, D421B, D423, and D424
- Baseline and project conditions
  - The combined flood hydrograph at node J43 was proportioned to nodes D431 and D432
    - The individual flood hydrograph at node D44 was assigned to node D441
    - The individual flood hydrograph at node D45 was assigned to node D451

Inflow flood hydrographs were depth proportioned across one up to several lateral grid elements based on a common critical water surface elevation across the applicable grid elements. All inflow flood hydrographs were bulked based on debris yield estimations.

Summaries of this translation/distribution of the combined hydrographs to the inflow locations at or near the canyon outfalls are summarized as follows:

- Table 5-3 (Baseline 1-percent annual chance flood, no areal effects)
- Table 5-4 (Baseline 1-percent annual chance flood, areal effects)
- Table 5-5 (Baseline Standard Project Flood, areal effects)
- Table 5-6 (Project 1-percent annual chance flood, no areal effects)
- Table 5-7 (Project 1-percent annual chance flood, areal effects)
- Table 5-8 (Project Standard Project Flood, areal effects)

Table 5-3. Baseline 1-percent annual chance flood inflow distribution, no areal effects

combined flow node at Dike No. 4	subbasin/ node	flood volume {ac-ft}	bulking factor	bulked flood volume {ac-ft}		redist. fraction of combined flow
				initial	redist.	
J41	D411	1,192	1.21	1,443	1,610	1.00
	D410	137	1.21	167	0	0.00
	J41	1,329	1.21	1,610	1,610	-
J42	D421A	336	1.27	425	471	0.32
	D421B	441	1.27	558	618	0.42
	D422	45	1.20	54	60	0.04
	D423	189	1.21	227	252	0.17
	D424	39	1.22	48	53	0.04
	D420	111	1.26	140	0	0.00
	J42	1,161	1.25	1,453	1,453	-
J43	D431	604	1.17	705	844	0.95
	D432	24	1.51	36	43	0.05
	D430	105	1.40	146	0	0.00
	J43	732	1.21	888	888	-
D44	D44	70	1.28	89	89	-
D45	D45	41	1.34	55	55	-

Table 5-4. Baseline 1-percent annual chance flood inflow distribution, areal effects

combined flow node at Dike No. 4	subbasin/ node	flood volume {ac-ft}	bulking factor	bulked flood volume {ac-ft}		redist. fraction of combined flow
				initial	redist.	
J41	D411	1,078	1.22	1,313	1,463	1.00
	D410	123	1.22	150	0	0.00
	J41	1,201	1.22	1,463	1,463	-
J42	D421A	303	1.28	387	428	0.33
	D421B	400	1.27	509	563	0.42
	D422	41	1.20	49	54	0.04
	D423	171	1.21	207	229	0.18
	D424	35	1.23	43	48	0.04
	D420	100	1.27	127	0	0.00
	J42	1,050	1.26	1,322	1,322	-
J43	D431	543	1.17	637	764	0.95
	D432	21	1.54	33	39	0.05
	D430	94	1.41	133	0	0.00
	J43	658	1.22	803	803	-
D44	D44	63	1.29	81	81	-
D45	D45	37	1.36	50	50	-

Table 5-5. Baseline SPF inflow distribution, areal effects

combined flow node at Dike No. 4	subbasin/ node	flood volume {ac-ft}	bulking factor	bulked flood volume {ac-ft}		redist. fraction of combined flow
				initial	redist.	
J41	D411	1,882	1.12	2,103	2,523	1.00
	D410	384	1.09	420	0	0.00
	J41	2,266	1.11	2,523	2,523	-
J42	D421A	559	1.14	640	748	0.33
	D421B	684	1.15	785	918	0.40
	D422	95	1.09	103	121	0.05
	D423	287	1.12	320	374	0.16
	D424	87	1.11	96	112	0.05
	D420	296	1.11	328	0	0.00
	J42	2,007	1.13	2,272	2,272	-
J43	D431	1,192	1.09	1,302	1,617	0.95
	D432	54	1.20	65	81	0.05
	D430	288	1.15	331	0	0.00
	J43	1,533	1.11	1,697	1,697	-
D44	D44	173	1.12	195	195	-
D45	D45	107	1.14	122	122	-

Table 5-6. Project 1-percent annual chance flood inflow distribution, no areal effects

combined flow node at Dike No. 4	subbasin/ node	flood volume {ac-ft}	bulking factor	bulked flood volume {ac-ft}		redist. fraction of combined flow
				initial	redist.	
J41	D411	1,192	1.21	1,443	1,615	0.96
	D422	45	1.20	54	61	0.04
	D410	151	1.18	178	0	0.00
	J41	1,388	1.21	1,675	1,675	-
J42	D421A	336	1.27	425	469	0.34
	D421B	441	1.27	558	615	0.44
	D423	189	1.21	227	251	0.18
	D424	39	1.22	48	53	0.04
	D420	111	1.17	129	0	0.00
	J42	1,115	1.24	1,388	1,388	-
J43	D431	604	1.17	705	844	0.95
	D432	24	1.51	36	43	0.05
	D430	105	1.40	146	0	0.00
	J43	732	1.21	888	888	-
D44	D44	70	1.28	89	89	-
D45	D45	41	1.34	55	55	-

Table 5-7. Project 1-percent annual chance flood inflow distribution, areal effects

combined flow node at Dike No. 4	subbasin/ node	flood volume {ac-ft}	bulking factor	bulked flood volume {ac-ft}		redist. fraction of combined flow
				initial	redist.	
J41	D411	1,078	1.22	1,313	1,467	0.96
	D422	41	1.20	49	55	0.04
	D410	136	1.18	161	0	0.00
	J41	1,255	1.21	1,522	1,523	-
J42	D421A	303	1.28	387	426	0.34
	D421B	400	1.27	509	561	0.44
	D423	171	1.21	207	228	0.18
	D424	35	1.23	43	48	0.04
	D420	99	1.18	116	0	0.00
	J42	1,008	1.25	1,262	1,262	-
J43	D431	543	1.17	637	764	0.95
	D432	21	1.54	33	39	0.05
	D430	94	1.41	133	0	0.00
	J43	658	1.22	803	803	-
D44	D44	63	1.29	81	81	-
D45	D45	37	1.36	50	50	-

Table 5-8. Project SPF inflow distribution, areal effects

combined flow node at Dike No. 4	subbasin/ node	flood volume {ac-ft}	bulking factor	bulked flood volume {ac-ft}		redist. fraction of combined flow
				initial	redist.	
J41	D411	1,882	1.12	2,103	2,526	0.95
	D422	95	1.09	103	124	0.05
	D410	411	1.08	444	0	0.00
	J41	2,387	1.11	2,650	2,650	-
J42	D421A	559	1.14	640	752	0.35
	D421B	684	1.15	785	923	0.43
	D423	287	1.12	320	376	0.17
	D424	87	1.11	96	113	0.05
	D420	303	1.07	324	0	0.00
	J42	1,920	1.13	2,164	2,164	-
J43	D431	1,192	1.09	1,302	1,617	0.95
	D432	54	1.20	65	81	0.05
	D430	288	1.15	331	0	0.00
	J43	1,533	1.11	1,697	1,697	-
D44	D44	173	1.12	195	195	-
D45	D45	107	1.14	122	122	-

5.2.8 Model exclusions

The Dike No. 4 outlet structure was not defined thereby eliminating any outflow from the impoundment. For the exclusion of topographic features, see Section 5.2.2.

5.2.9 Model variations for flood pattern uncertainty

Given the uncertainty in flood pattern behavior on and along the edges of the active areas, one model variation was simulated to determine a worse-case composite flood environment as it relates to the aggregate of maximum flood depths and velocities as well as the distribution of flood rates along the edge conditions and in near vicinity of the planned development. This variation considers the upper breakout location (DC1) on the southern bank/terrace of the Devil Canyon active floodplain previously identified in Figure 3-5. This model variation was implemented by obstructing the Devil Canyon active floodplain at DC1 to force floodwaters onto the southern overbank/terrace.

The results from this model variation is not intended for the purpose of mapping the flood hazard, but rather, to provide a depiction of flood patterns based on recent digital topographic mapping for the purpose of conceptualizing viable flood protection and conveyance alternatives for the planned development.

No variations were considered for the Middle Canyon Area, since the flood conveyance below the canyon outfalls is generally confined with a preferred direction on a portion of the piedmont/fans, which is considered inactive, due partly to the substantial amount of soil development that has occurred over time.

No flow path variations were considered for the Toro Canyon Area outfalls since no development is proposed near this region.

5.2.10 Model simulation results

In regard to the graphical depiction of simulation outcomes, the maximum flood depths and velocities were displayed based on a minimum depth threshold of 0.5 feet.

The simulated maximum flood depths produced by the 1-percent annual chance flood routing models focused on conveyance along the edge conditions and near vicinity of the planned development are presented as follows:

- Baseline conditions (Figure 5-6)
- Baseline conditions with forced split at breakout location DC1 previously identified in Section 3 (Figure 5-7)
- Composite of baseline conditions above (Figure 5-8)
- Project conditions (Figure 5-9)
- Project conditions with forced split at breakout location DC1 previously identified in Section 3 (Figure 5-10)
- Composite of project conditions above (Figure 5-11)
- Change in composite maximum flood depths from baseline to project conditions (Figure 5-12)

The simulated maximum flood velocities produced by the 1-percent annual chance flood routing models focused on conveyance and temporary impoundment along the edge conditions and near vicinity of the planned development are presented as follows:

- Baseline conditions (Figure 5-13)
- Baseline conditions with forced split at breakout location DC1 previously identified in Section 3 (Figure 5-14)
- Composite of baseline conditions above (Figure 5-15)
- Project conditions (Figure 5-16)
- Project conditions with forced split at breakout location DC1 previously identified in Section 3 (Figure 5-17)
- Composite of project conditions above (Figure 5-18)
- Change in composite maximum flood velocities from baseline to project conditions (Figure 5-19)

The simulated maximum flood depths produced by the 1-percent annual chance flood routing models focused on impoundment along the interior of Dike No. 4 and are presented as follows:

- Baseline conditions (Figure 5-20)
- Project conditions (Figure 5-21)
- Change in flood depths from baseline to project conditions (Figure 5-22)

The simulated maximum flood depths produced by the Standard Project Flood routing models focused on impoundment along the interior of Dike No. 4 are presented as follows:

- Baseline conditions (Figure 5-23)
- Project conditions (Figure 5-24)
- Change in flood depths from baseline to project conditions (Figure 5-25)

5.2.10.1 Dike No. 4 freeboard evaluation

A profile comparison of ground elevations and maximum water surface elevations along Dike No. 4, including past results determined as part of the FEMA Physical Map Revision Application (PACE, 2005), are presented in Figure 5-13. A portion of each ground elevation profile is missing due to spatial gaps in

topographic mapping. In addition, the 2011 LiDAR ground elevation profiles show a significant amount of variability, which may suggest there are mapping irregularities.

A comparison of the follow water surface profiles are presented in Figure 5-26:

- 1-percent annual chance flood, baseline conditions
- 1-percent annual chance flood, project conditions
- 1-percent annual chance flood (PACE, 2005)
- Standard Project Flood, baseline conditions
- Standard Project Flood, project conditions
- Standard Project Flood (PACE, 2005)

5.2.10.2 Guadalupe Creek Diversion Dikes

The "conveyance" models identified above were also used to determine the aggregate 1-percent annual chance maximum flow rate as measured from floodplain cross sections defined across the conveyance area between the Guadalupe Creek Diversion Dikes:

- Baseline conditions: 7,000 cfs
- Baseline conditions with forced split at breakout location DC1: 0 cfs
- Project conditions: 13,704 cfs
- Project conditions with forced split at breakout location DC1: 13,790 cfs

Profile comparisons of ground elevations and 1-percent annual chance maximum water surface elevations for the upper Guadalupe Creek Diversion Dikes are presented as follows:

- Baseline (no breakout) and project (with forced split at breakout location DC1) conditions maximum water surface profiles adjacent to the Northern Guadalupe Creek Diversion Dike (Figure 5-27)
- Baseline (no breakout) and project (with forced split at breakout location DC1) conditions maximum water surface profiles adjacent to the Southern Guadalupe Creek Diversion Dike (Figure 5-28)
- Baseline (no breakout) and project (with forced split at breakout location DC1) conditions maximum water surface profiles along the centerline between the upper Guadalupe Creek Diversion Dikes (Figure 5-29)

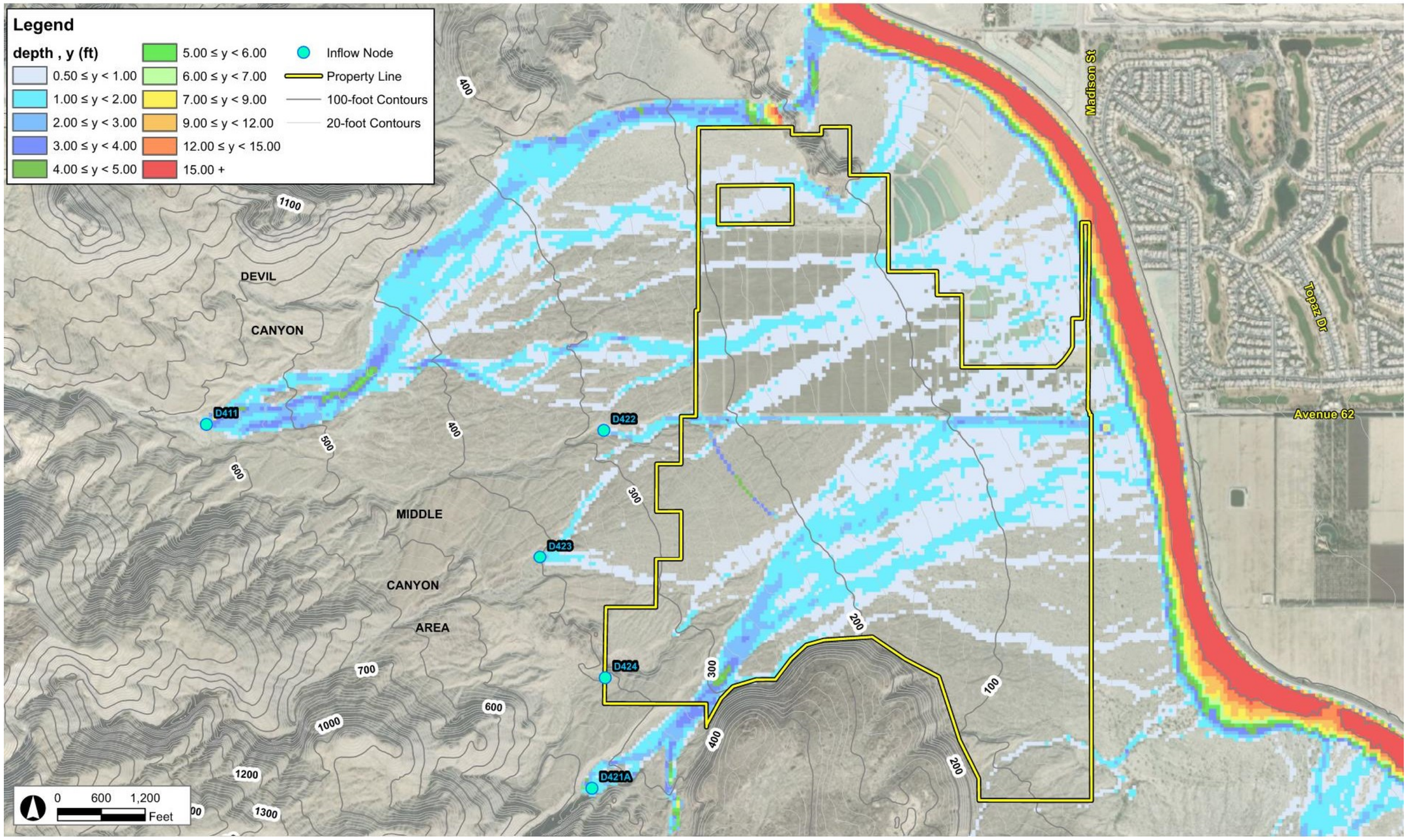
Profile comparisons of flood depths and velocities adjacent to the upper Guadalupe Creek Diversion Dikes are presented as follows:

- Baseline (no breakout) and project (with forced split at breakout location DC1) conditions flood depth and velocity profiles adjacent to the Northern Guadalupe Creek Diversion Dike (Figure 5-29)
- Baseline (no breakout) and project (with forced split at breakout location DC1) conditions flood depth and velocity profiles adjacent to the Southern Guadalupe Creek Diversion Dike (Figure 5-30)
- Baseline (no breakout) and project (with forced split at breakout location DC1) conditions flood depth and velocity profiles along the centerline between the upper Guadalupe Creek Diversion Dikes (Figure 5-31)

For reference, a plan view of the upper Guadalupe Creek Diversion Dikes, as interpreted from the as-built drawing (Figure 2-1), is shown in Figure 5-32.

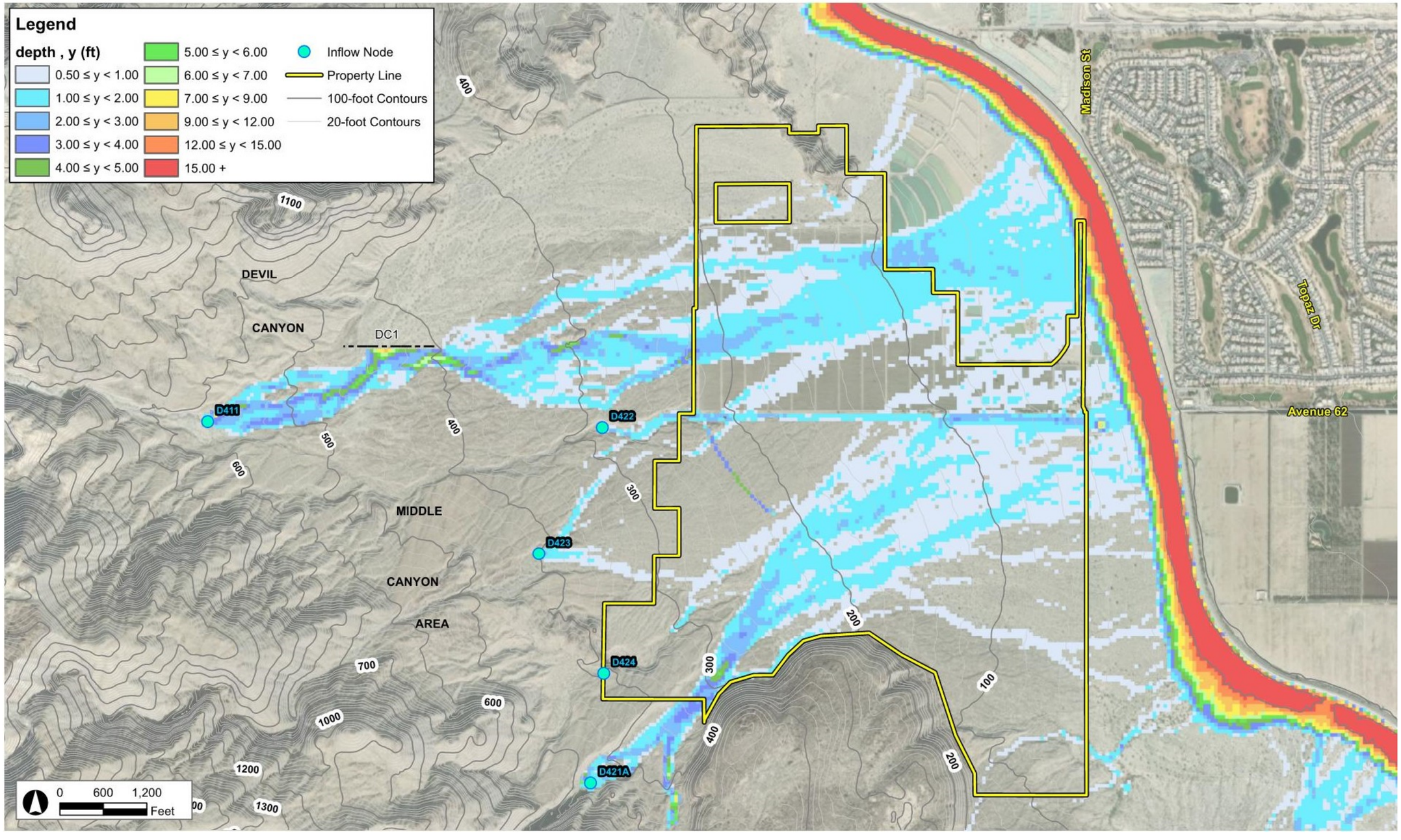
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Figure 5-6 Travertine baseline 1-percent annual chance maximum depths ≥ 0.5 feet



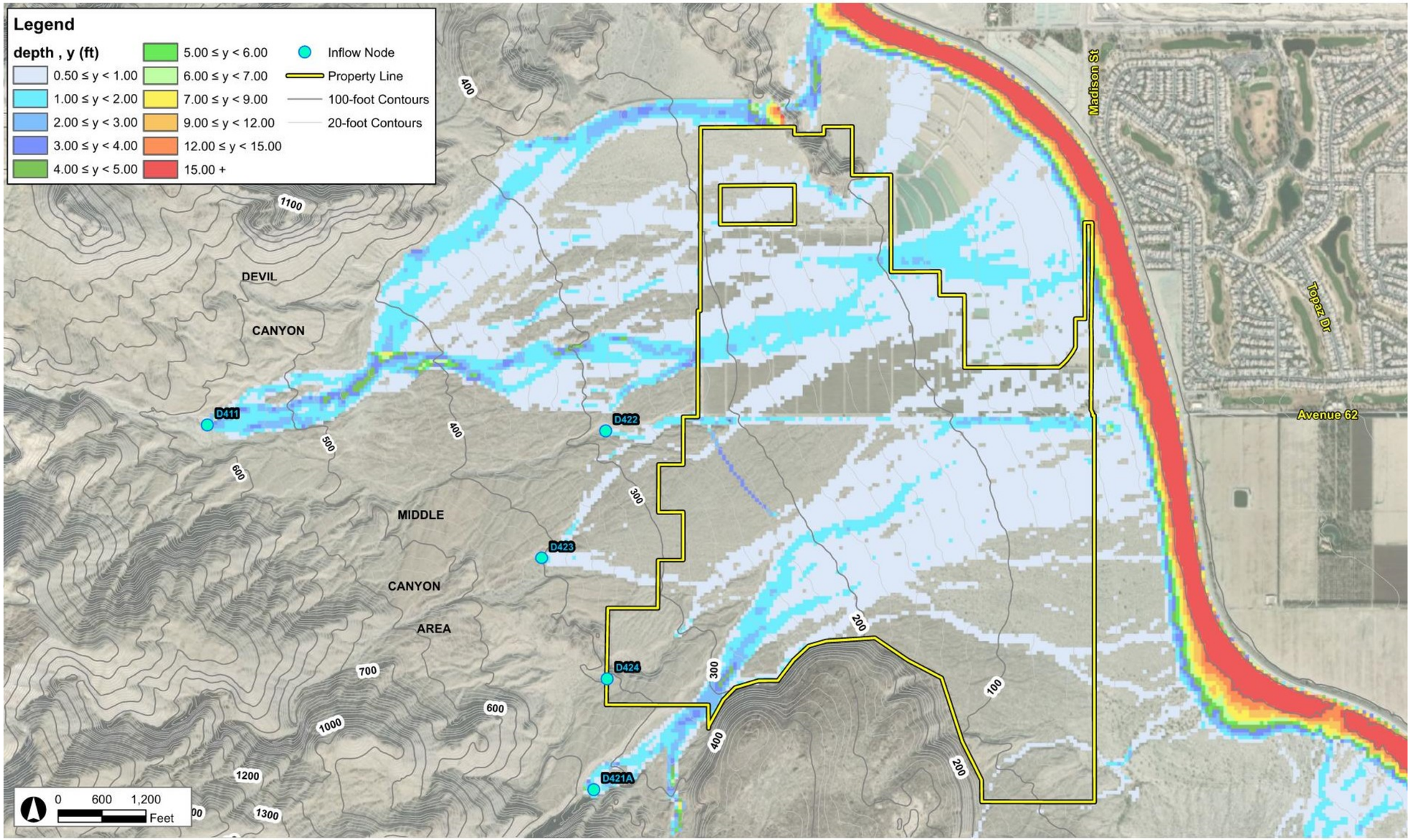
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Figure 5-7. Travertine baseline 1-percent annual chance maximum depths ≥ 0.5 feet with DC1 forced split in effect



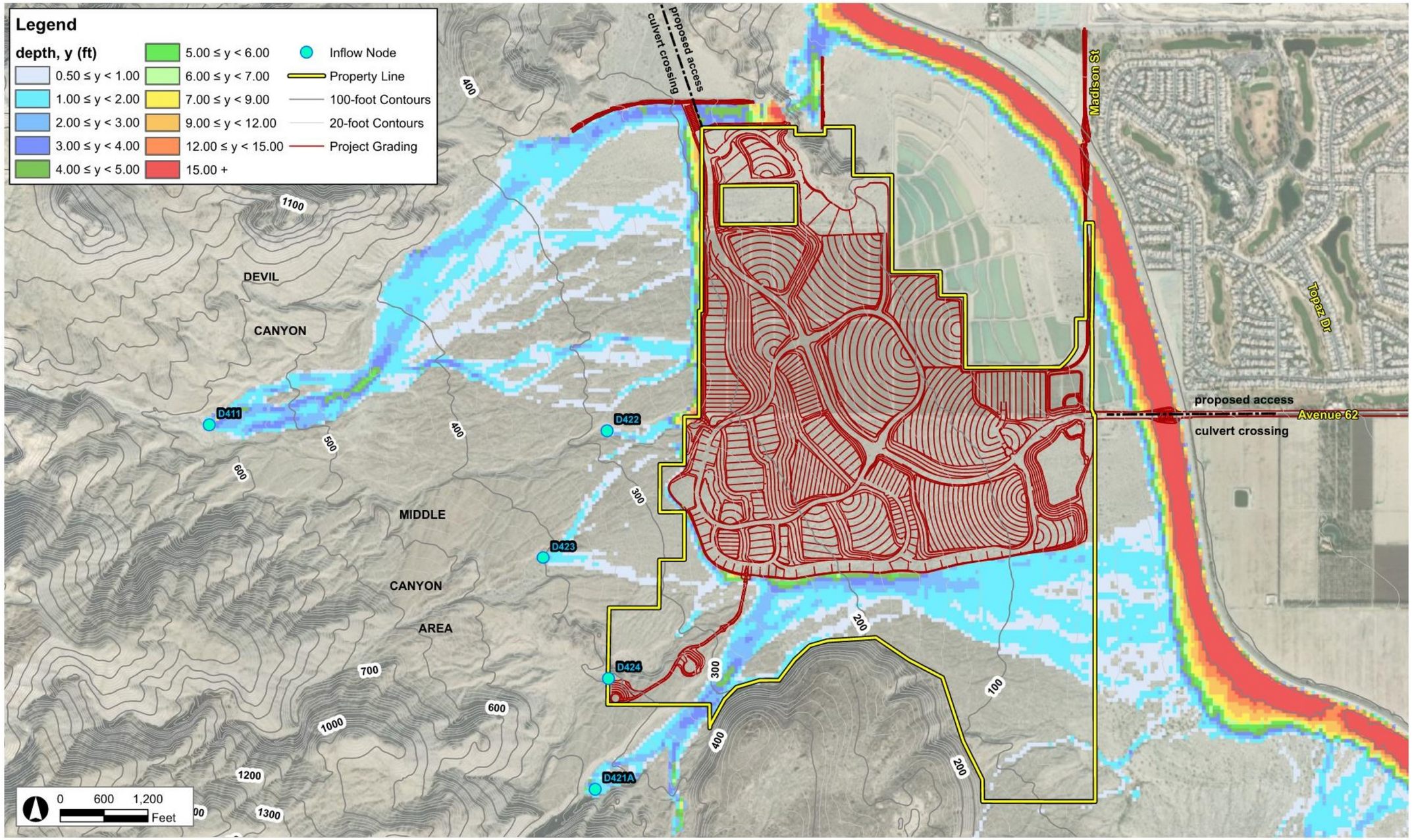
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Figure 5-8. Travertine composite baseline 1-percent annual chance maximum depths ≥ 0.5 feet



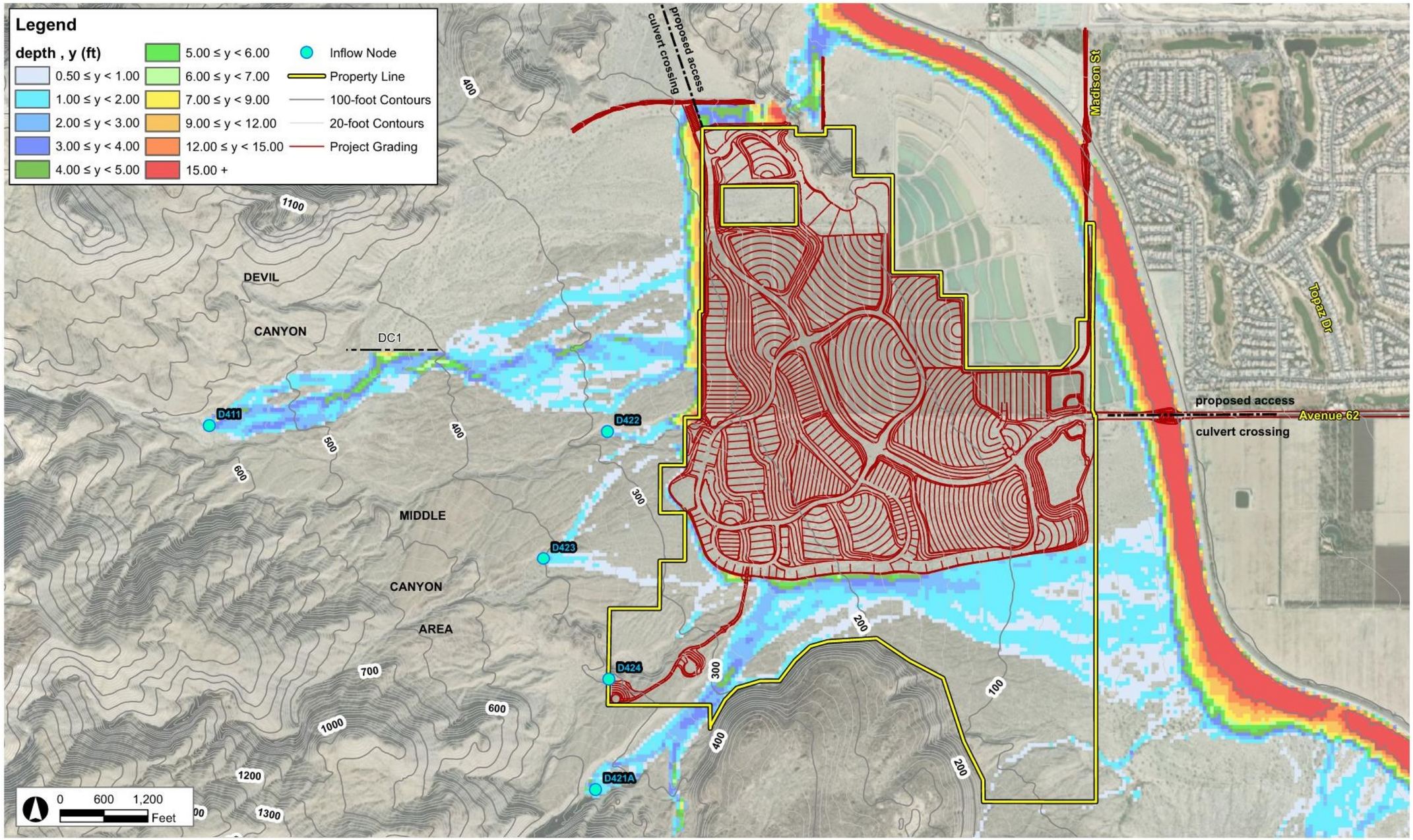
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Figure 5-9. Travertine project 1-percent annual chance maximum depths ≥ 0.5 feet



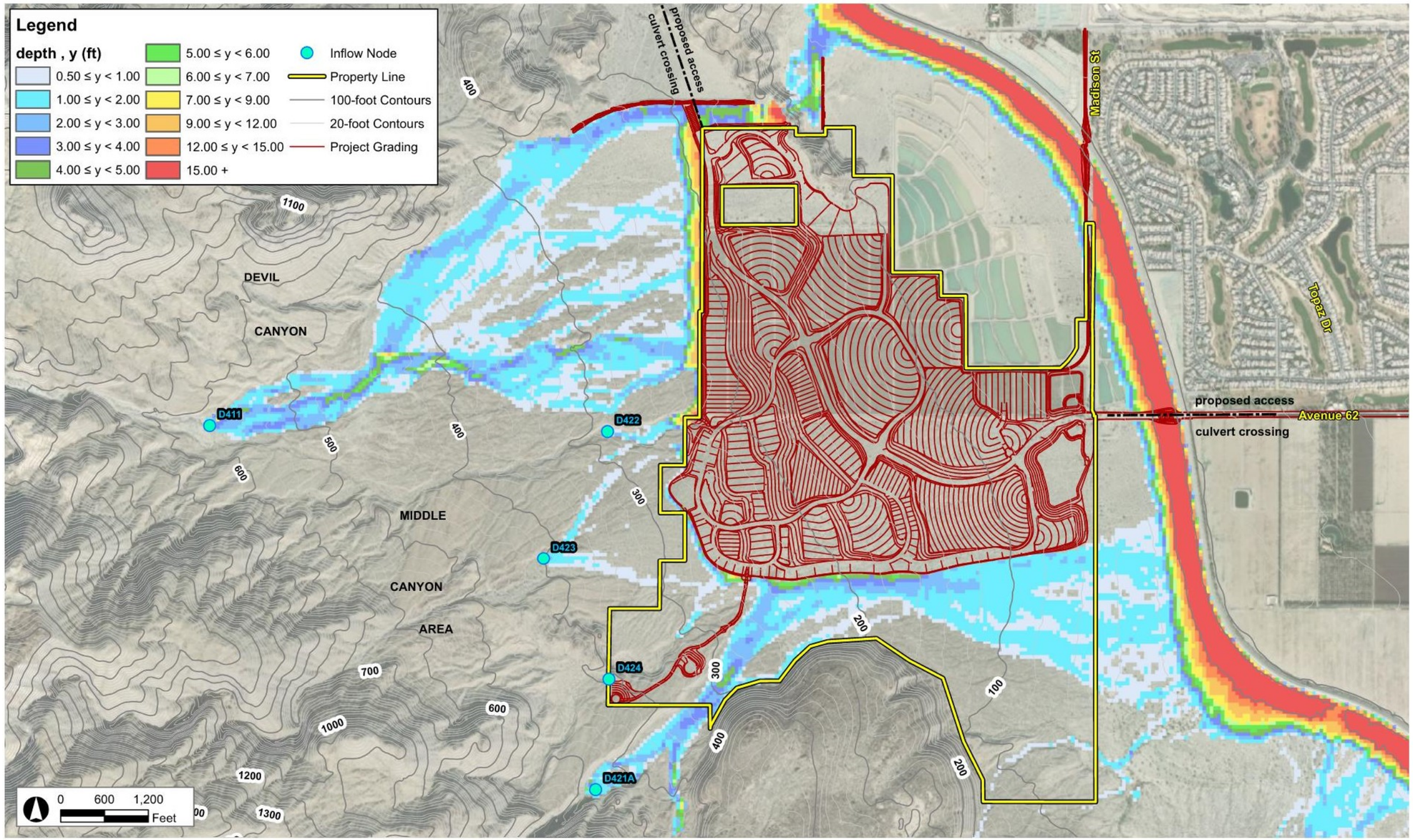
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Figure 5-10. Travertine project 1-percent annual chance maximum depths ≥ 0.5 feet with DC1 forced split in effect



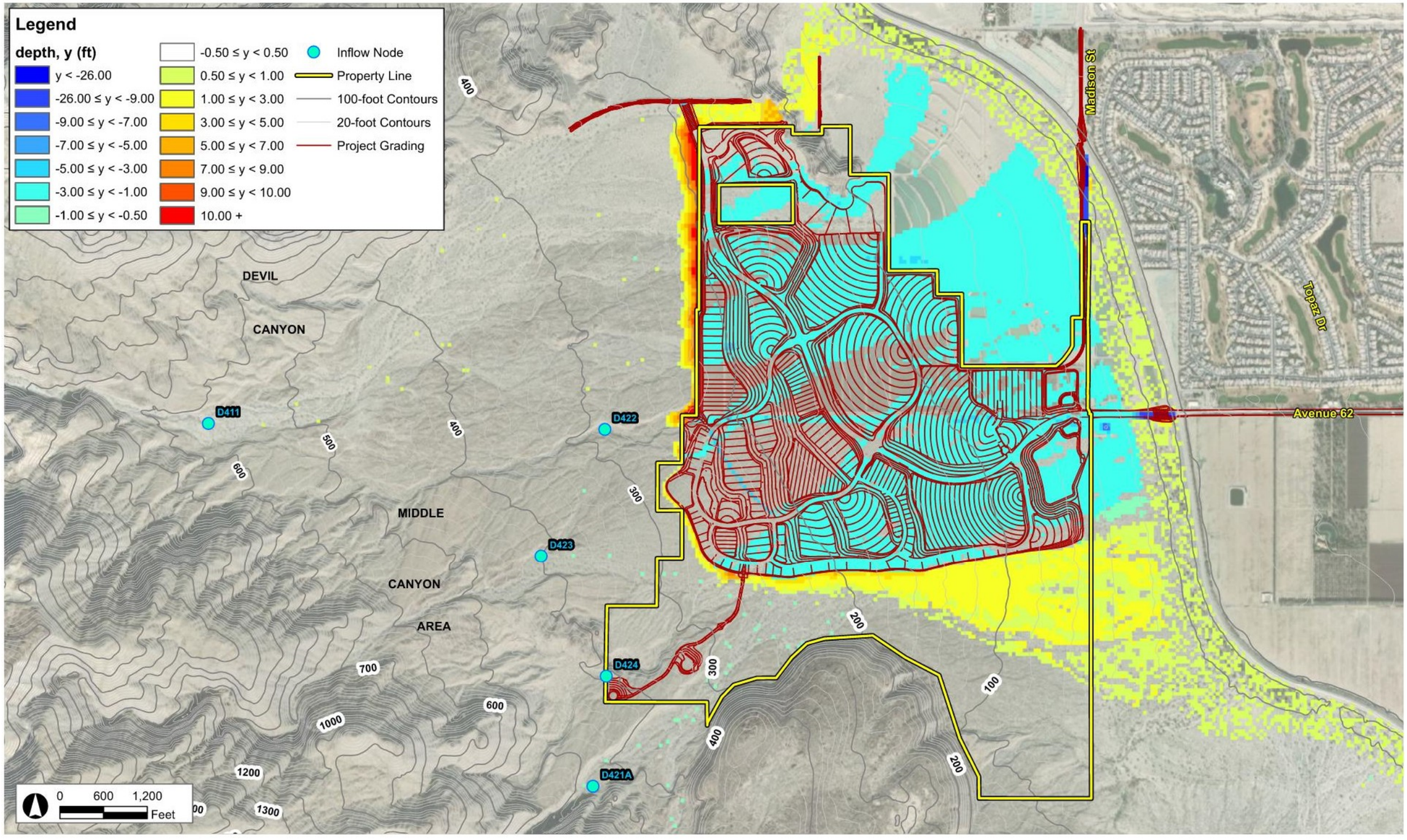
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Figure 5-11. Travertine composite project 1-percent annual chance maximum depths ≥ 0.5 feet



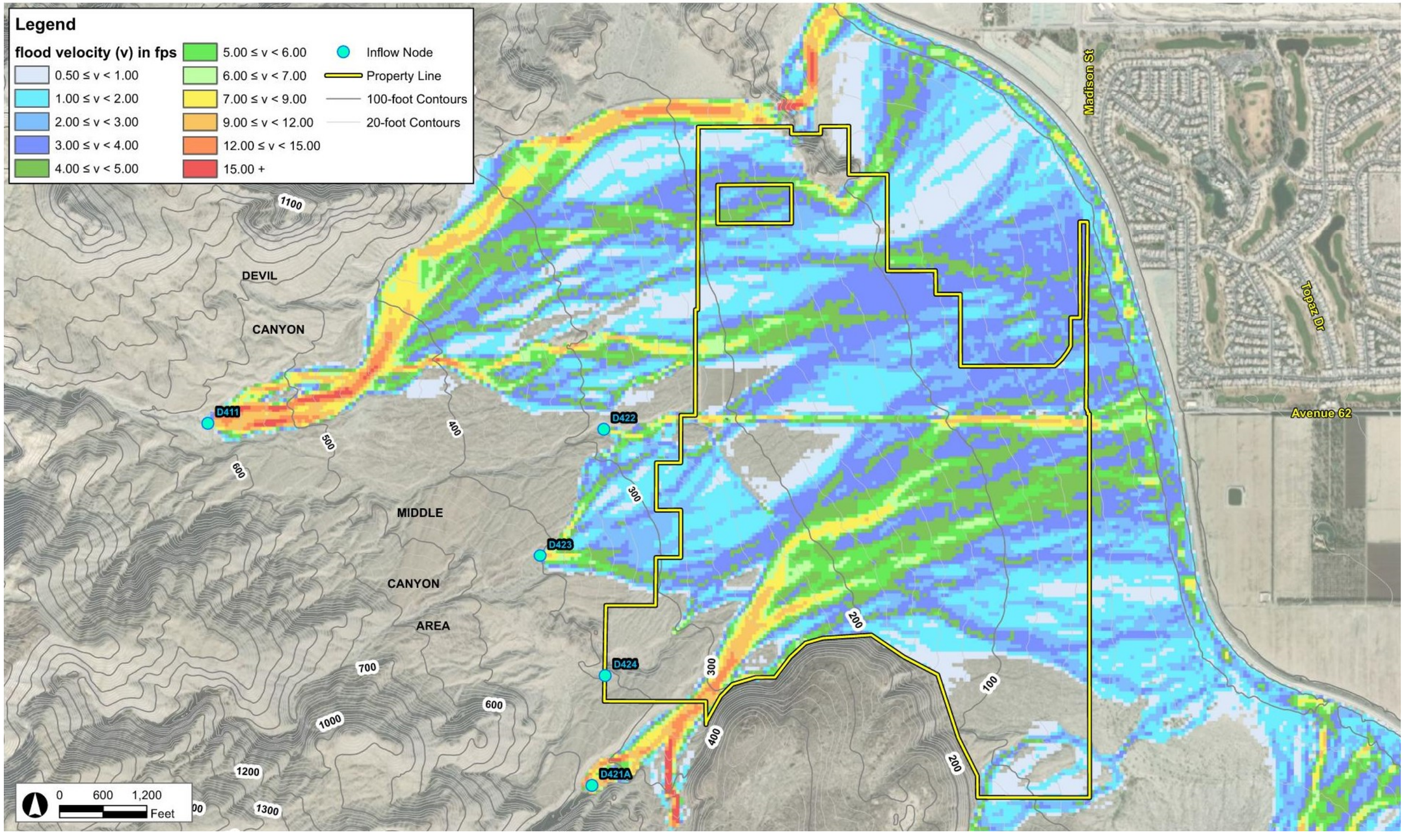
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Figure 5-12. Travertine change in composite 1-percent annual chance maximum depths from baseline to project conditions



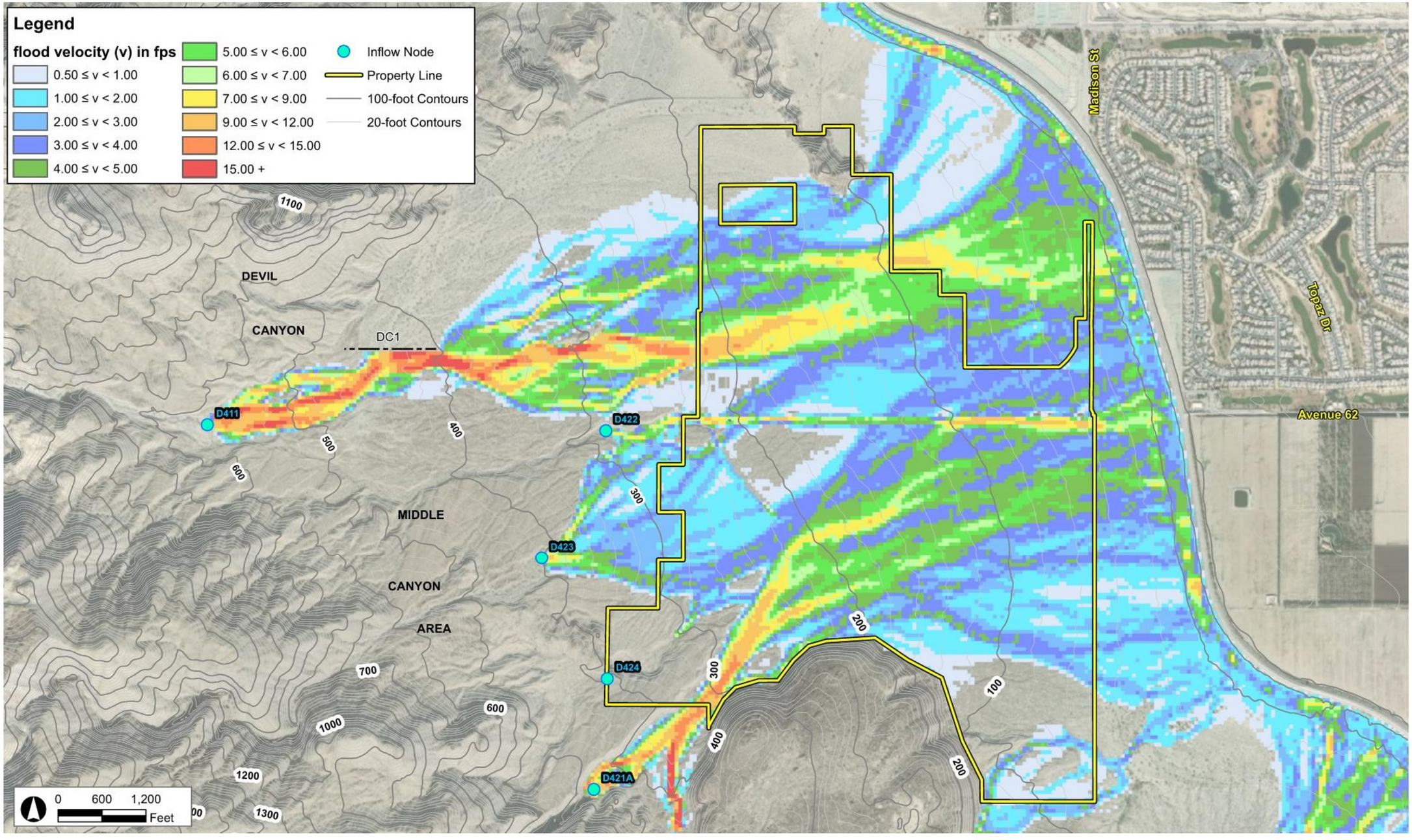
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Figure 5-13. Travertine baseline 1-percent annual chance maximum velocities (maximum depths ≥ 0.5 feet)



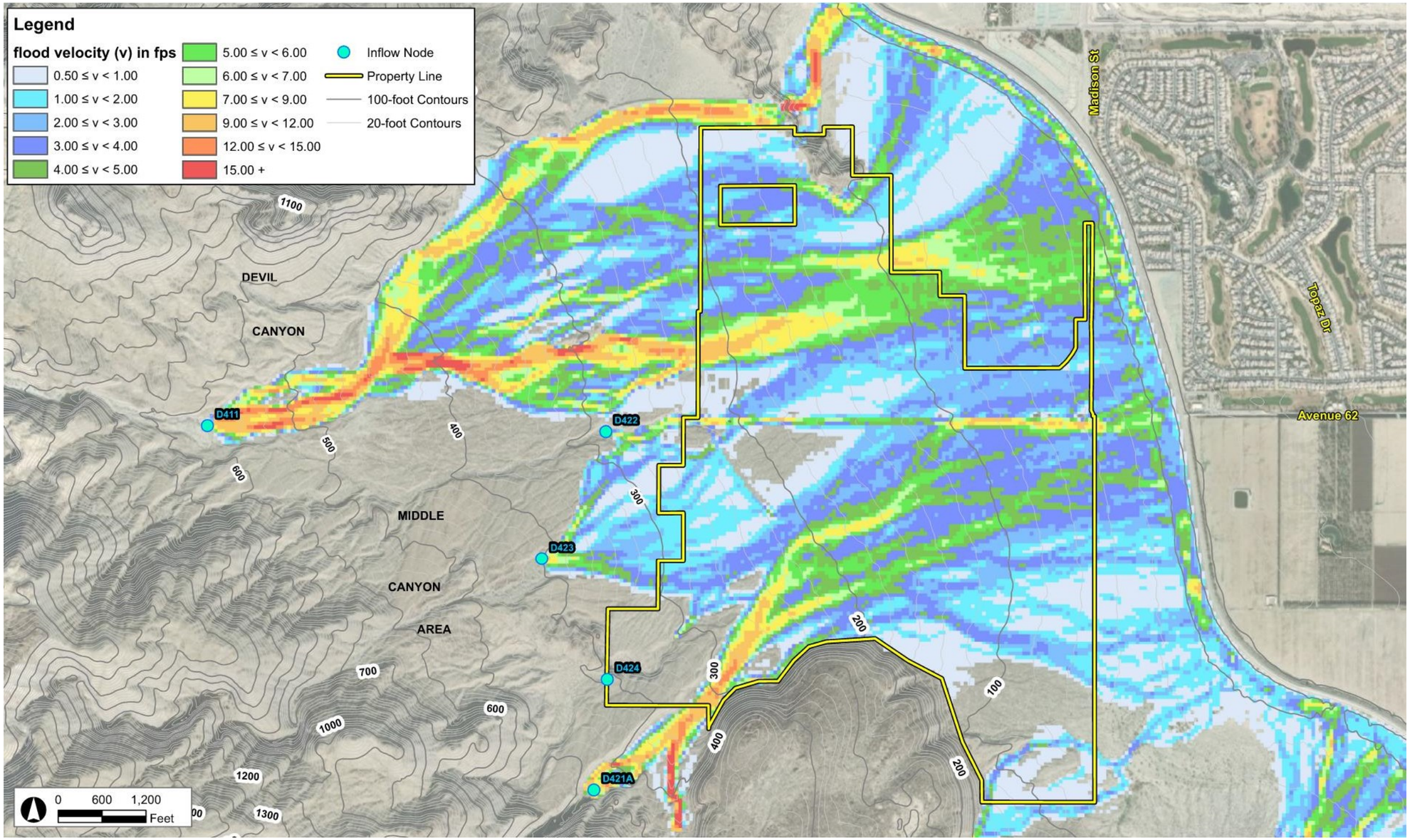
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Figure 5-14. Travertine baseline 1-percent annual chance maximum velocities (maximum depths ≥ 0.5 feet) with DC1 forced split in effect



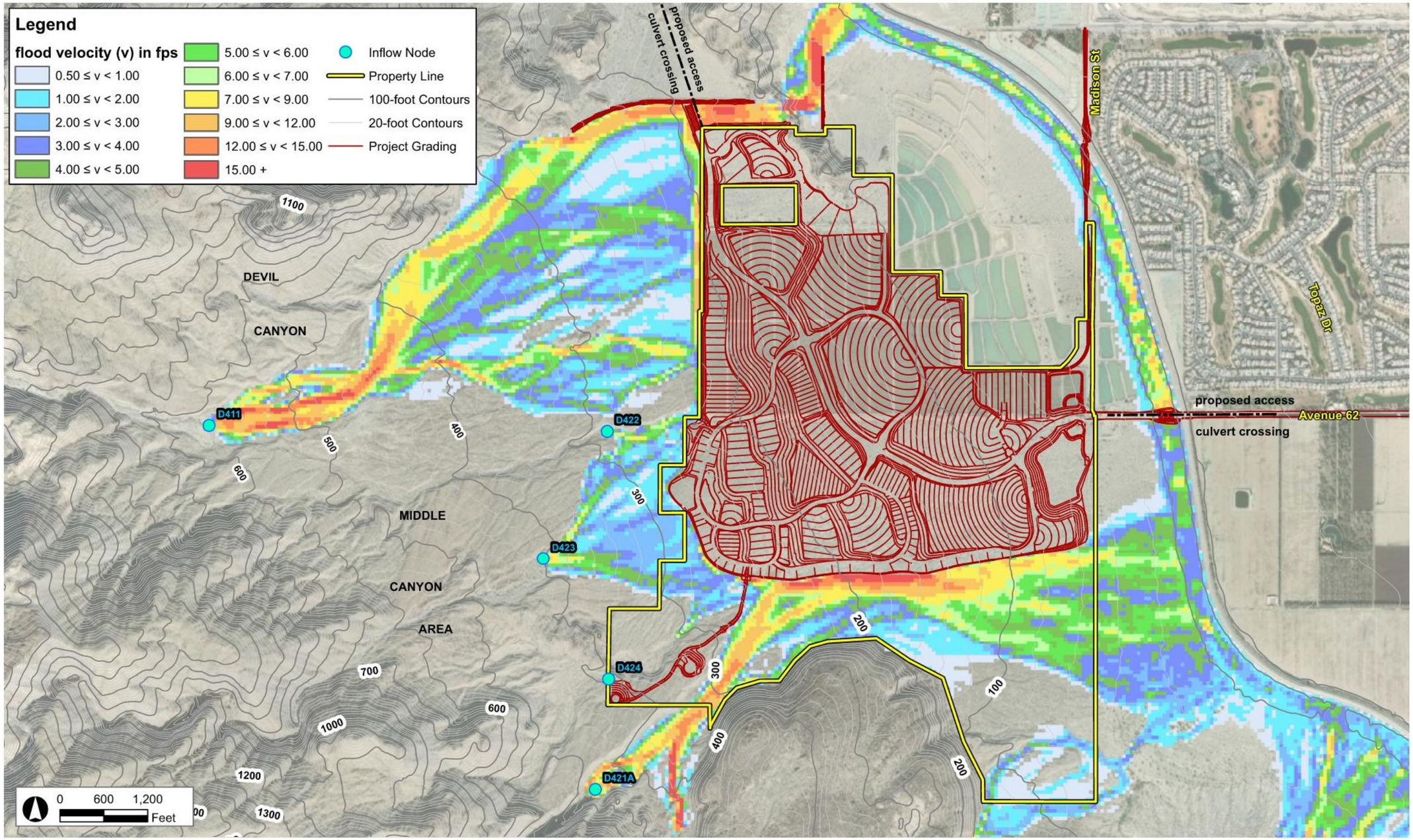
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Figure 5-15. Travertine composite baseline 1-percent annual chance maximum velocities (maximum depths ≥ 0.5 feet)



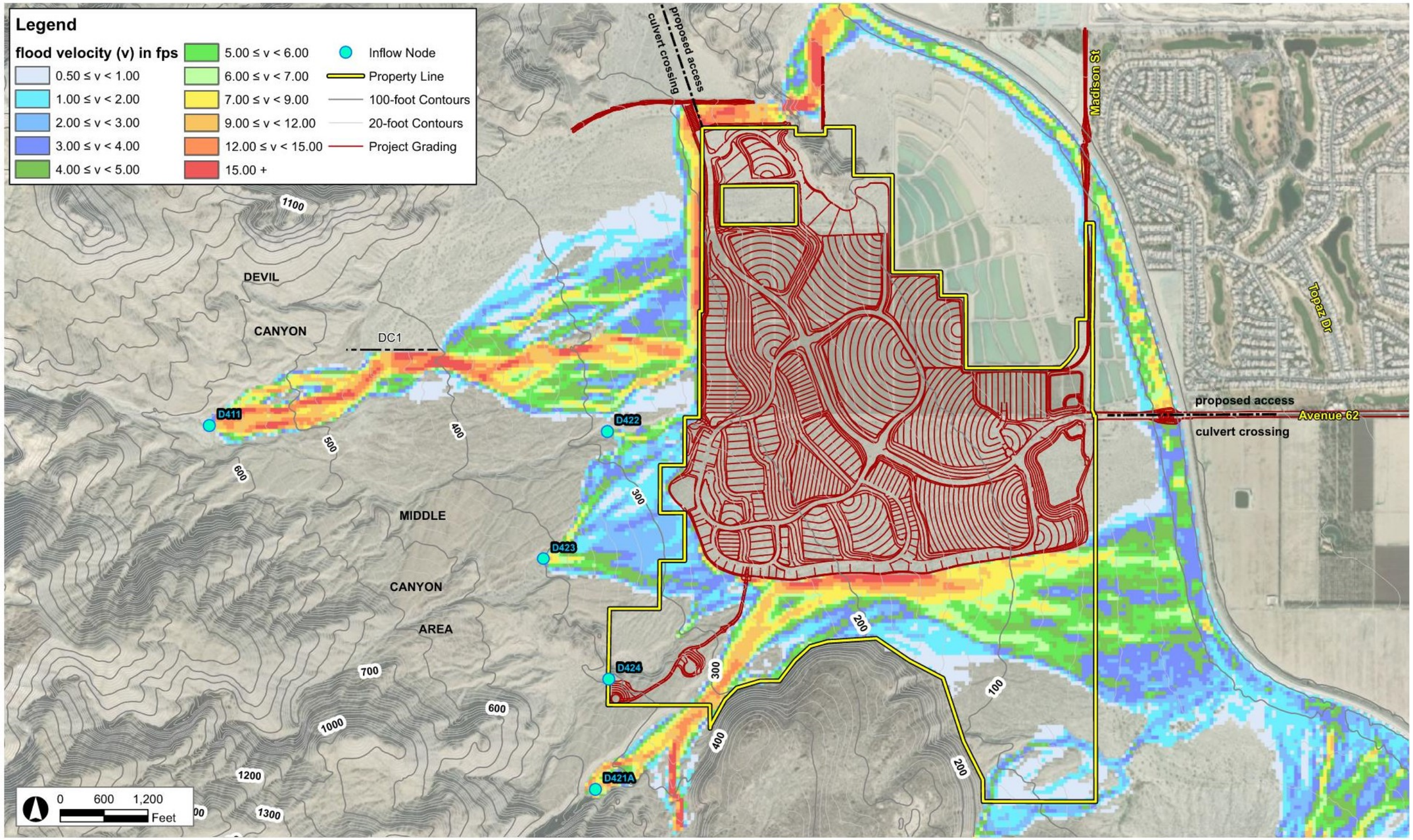
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Figure 5-16. Travertine project 1-percent annual chance maximum velocities (maximum depths ≥ 0.5 feet)



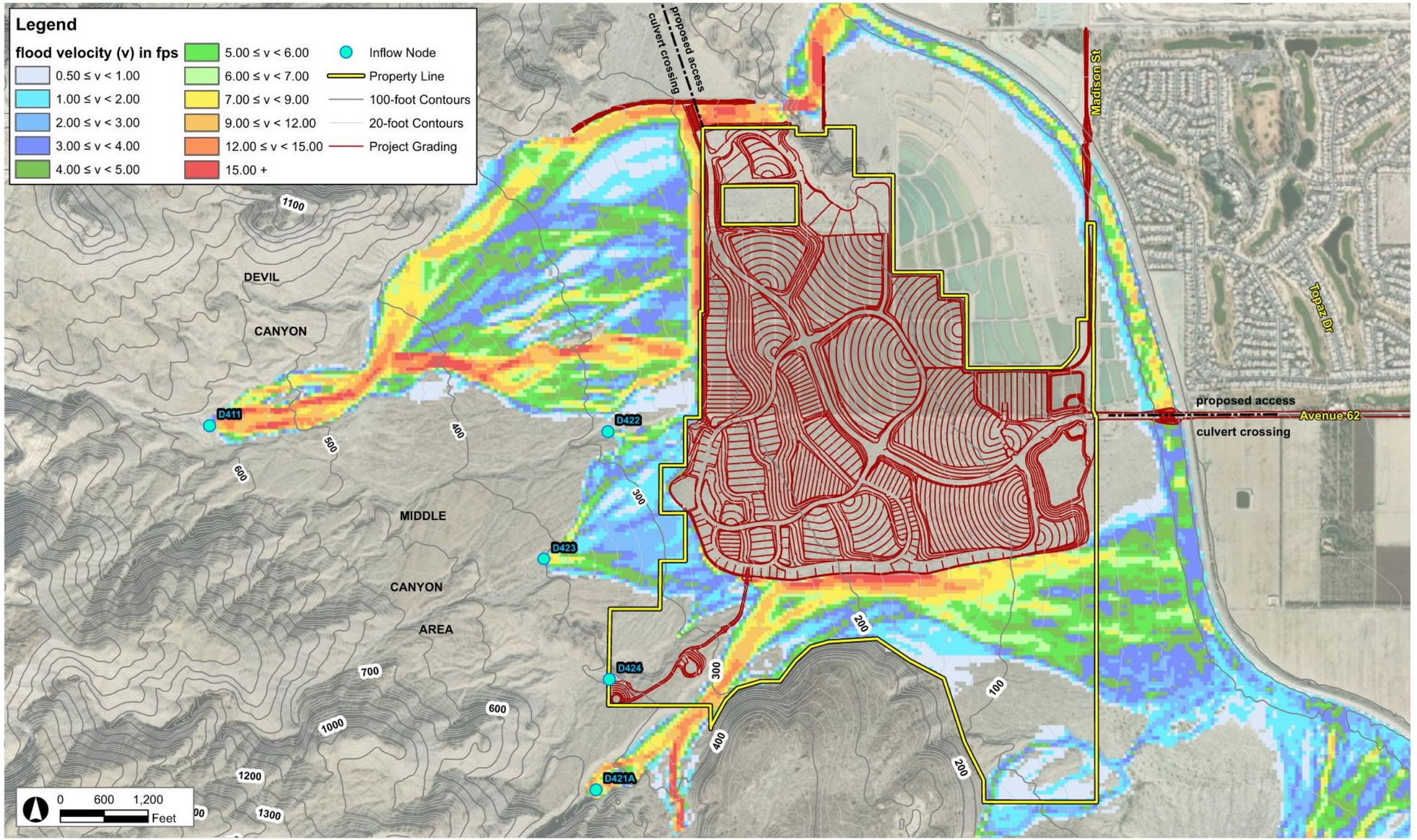
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Figure 5-17. Travertine project 1-percent annual chance maximum velocities (maximum depths ≥ 0.5 feet) with DC1 forced split in effect



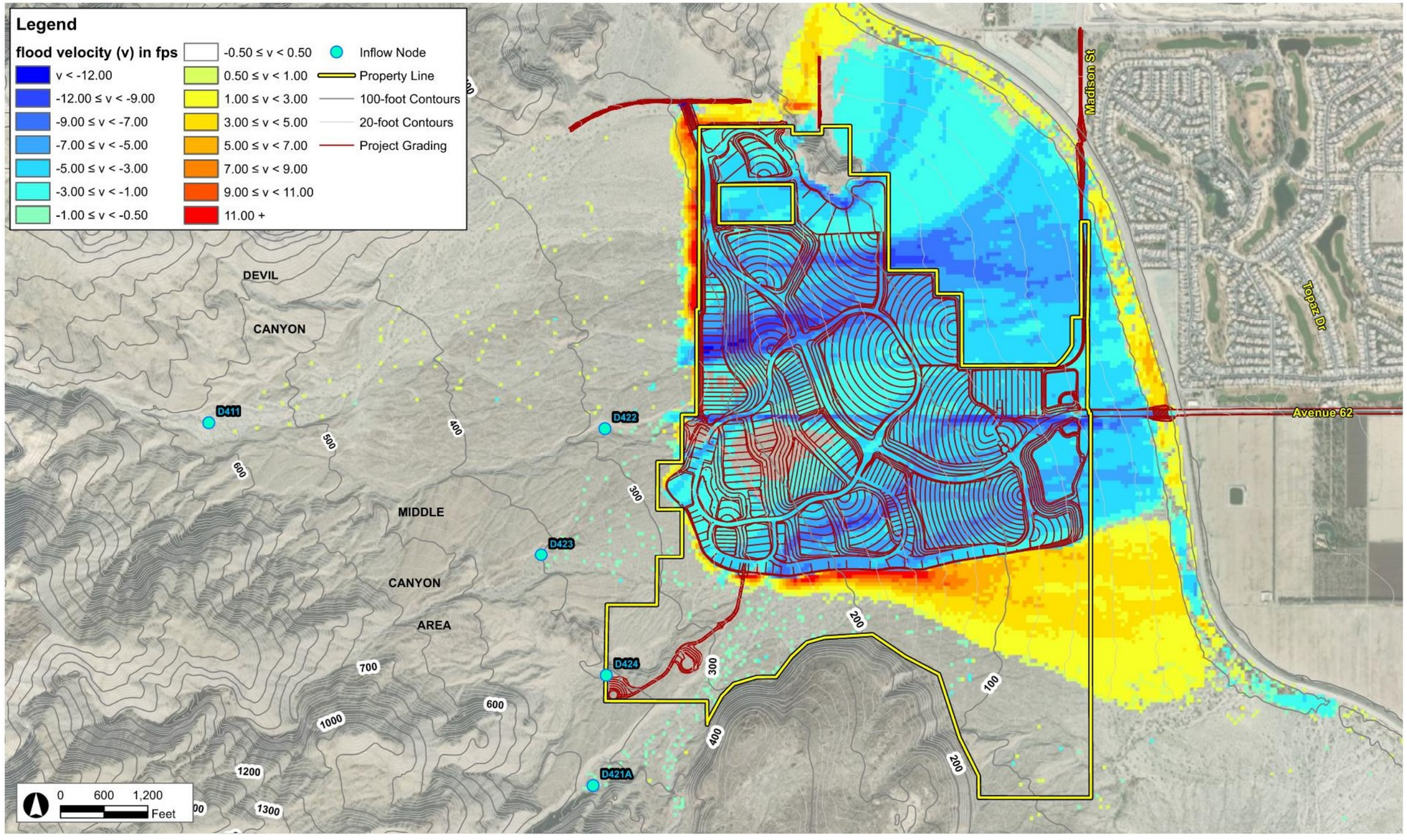
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Figure 5-18. Travertine composite project 1-percent annual chance maximum velocities (maximum depths ≥ 0.5 feet)



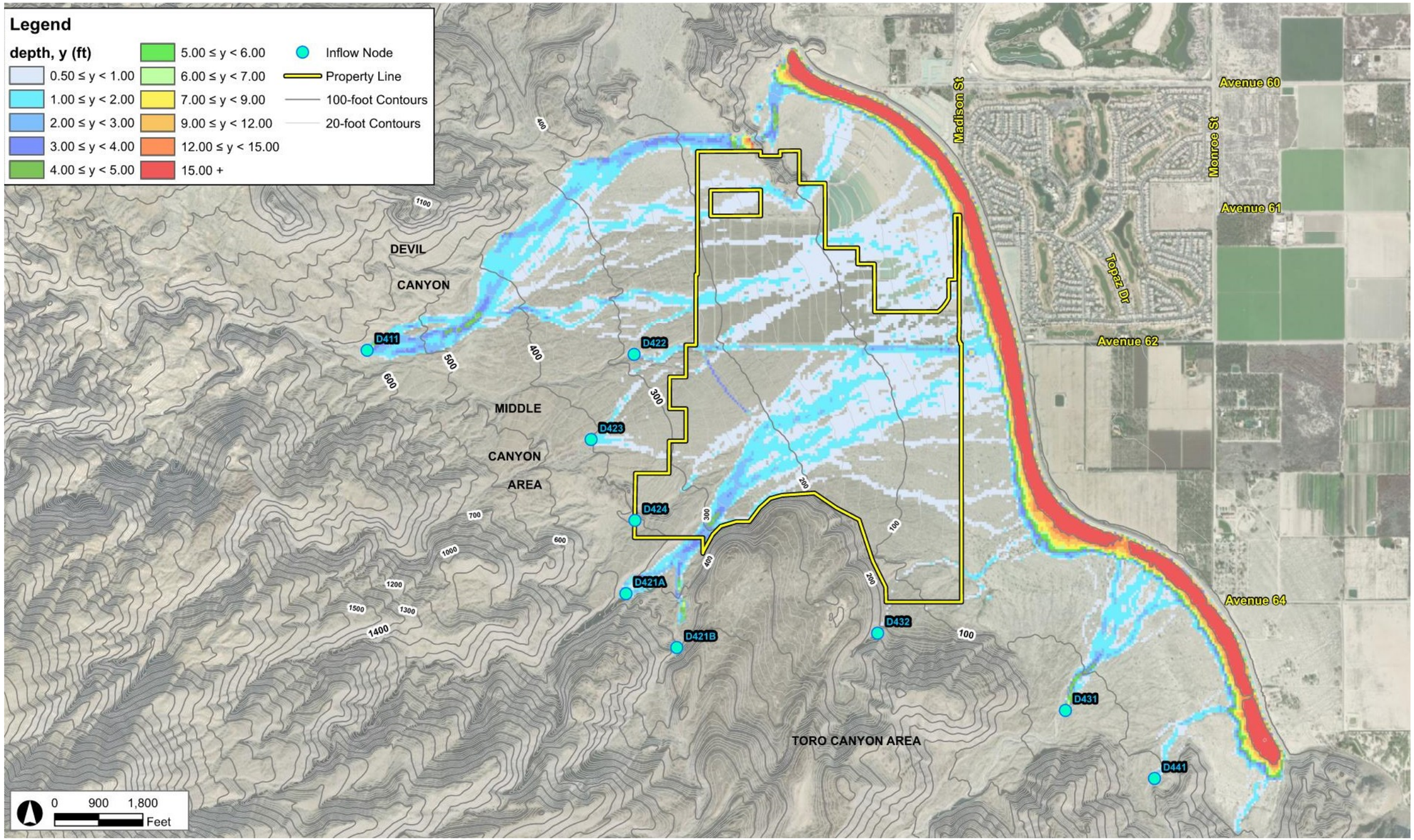
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Figure 5-19. Travertine change in composite 1-percent annual chance maximum velocities (maximum depths ≥ 0.5 feet) from baseline to project conditions



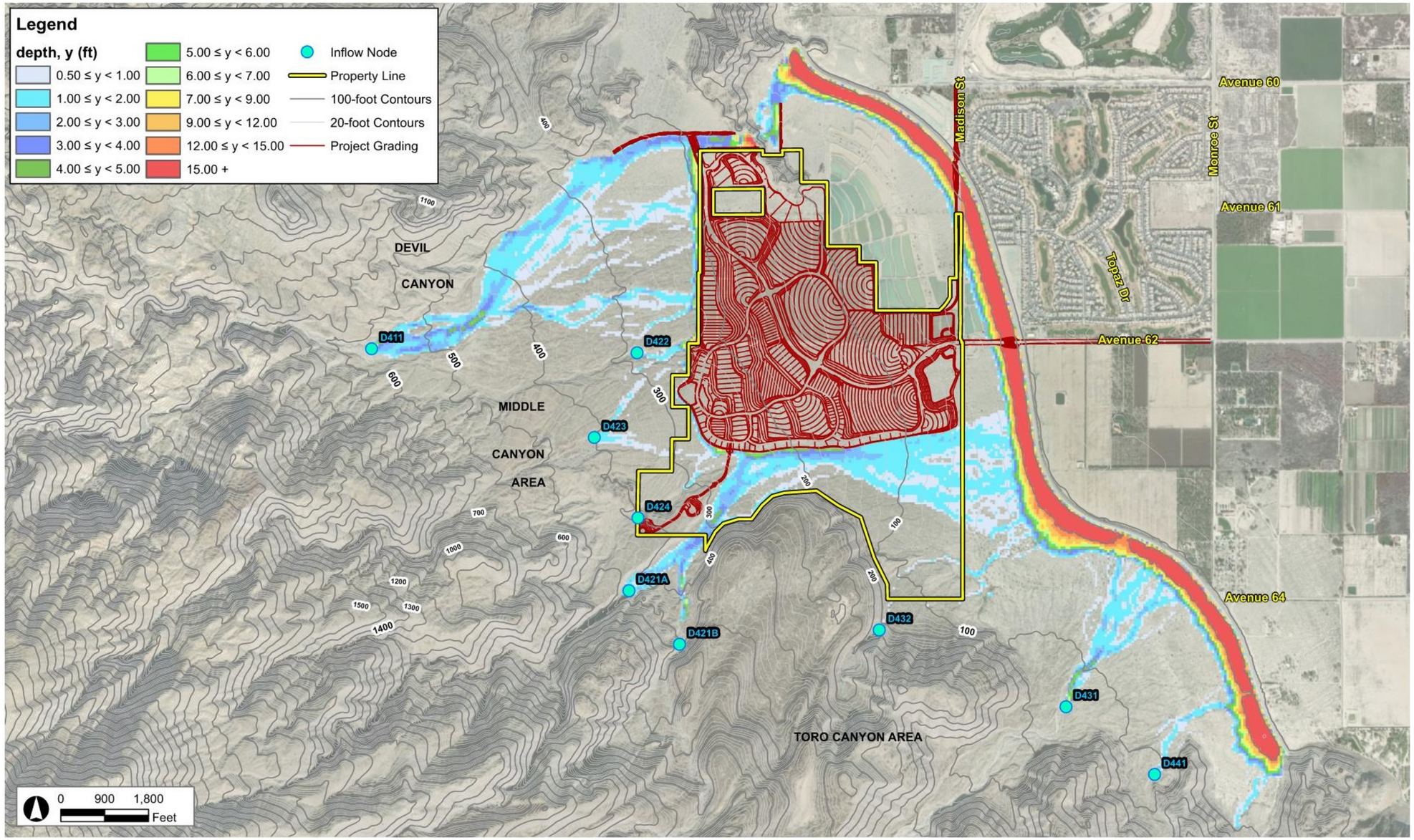
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Figure 5-20. Dike No. 4 baseline 1-percent annual chance maximum depths ≥ 0.5 feet



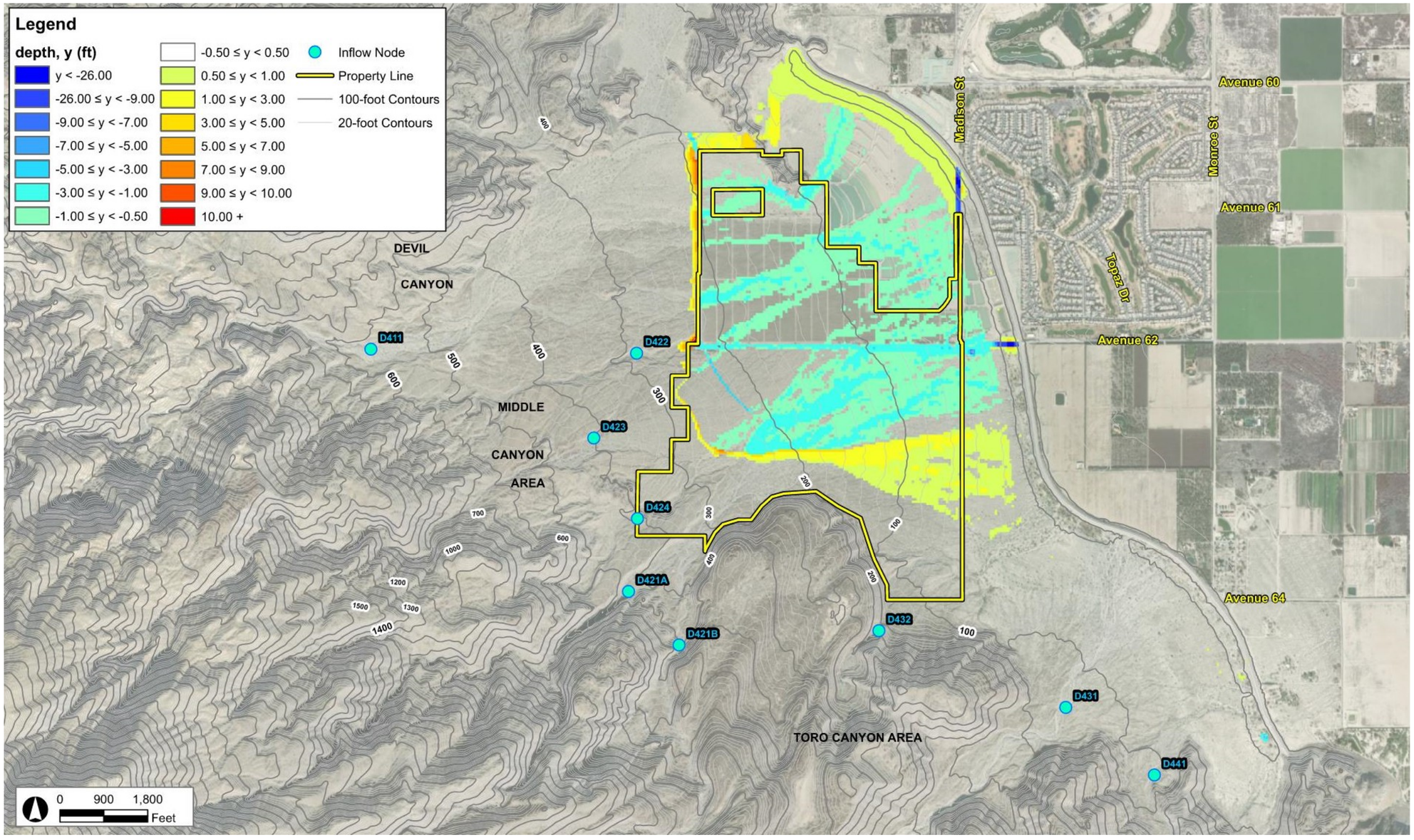
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Figure 5-21. Dike No. 4 project 1-percent annual chance maximum depths ≥ 0.5 feet



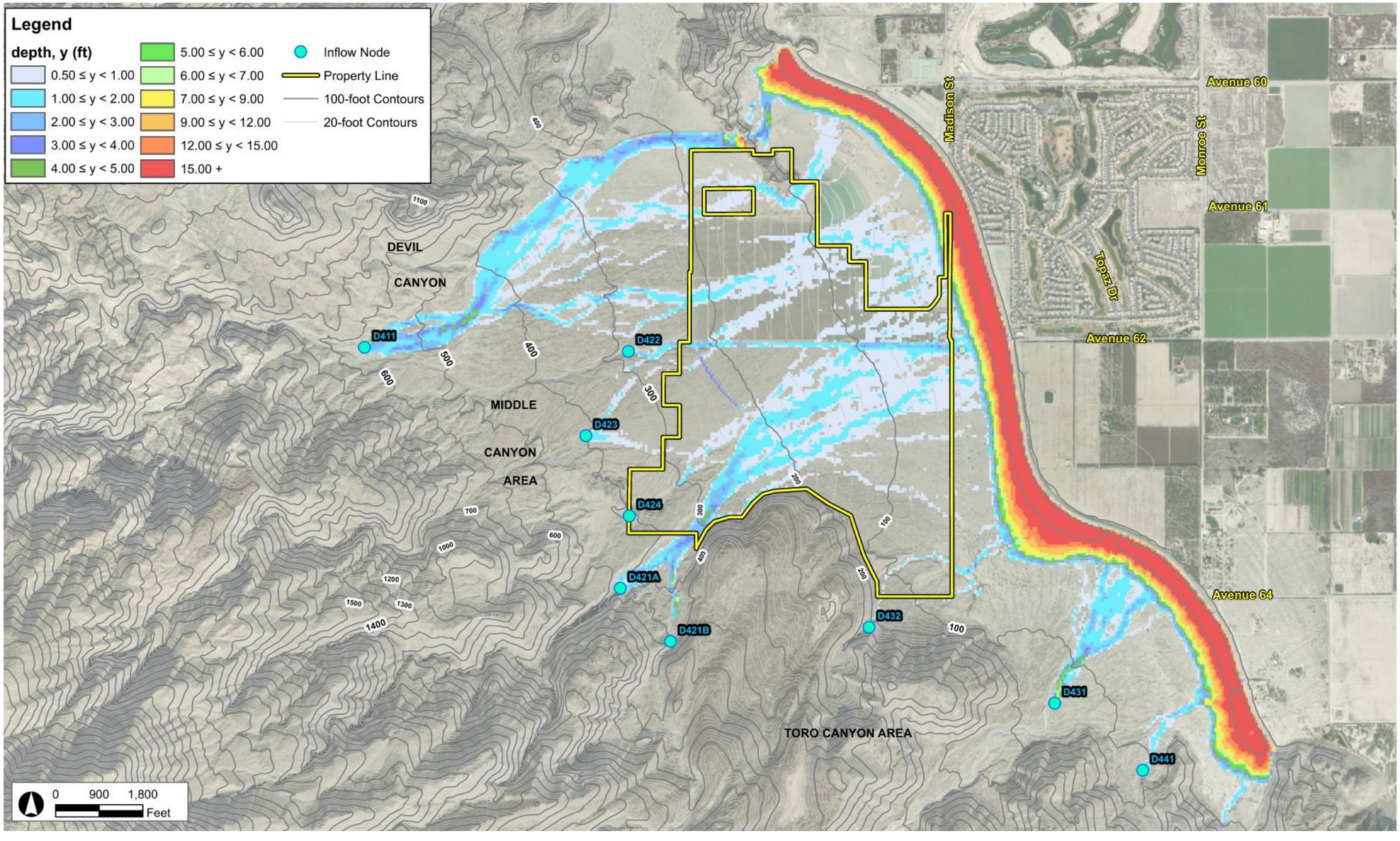
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Figure 5-22. Dike No. 4 change in 1-percent annual chance maximum depths ≥ 0.5 feet from baseline to project conditions



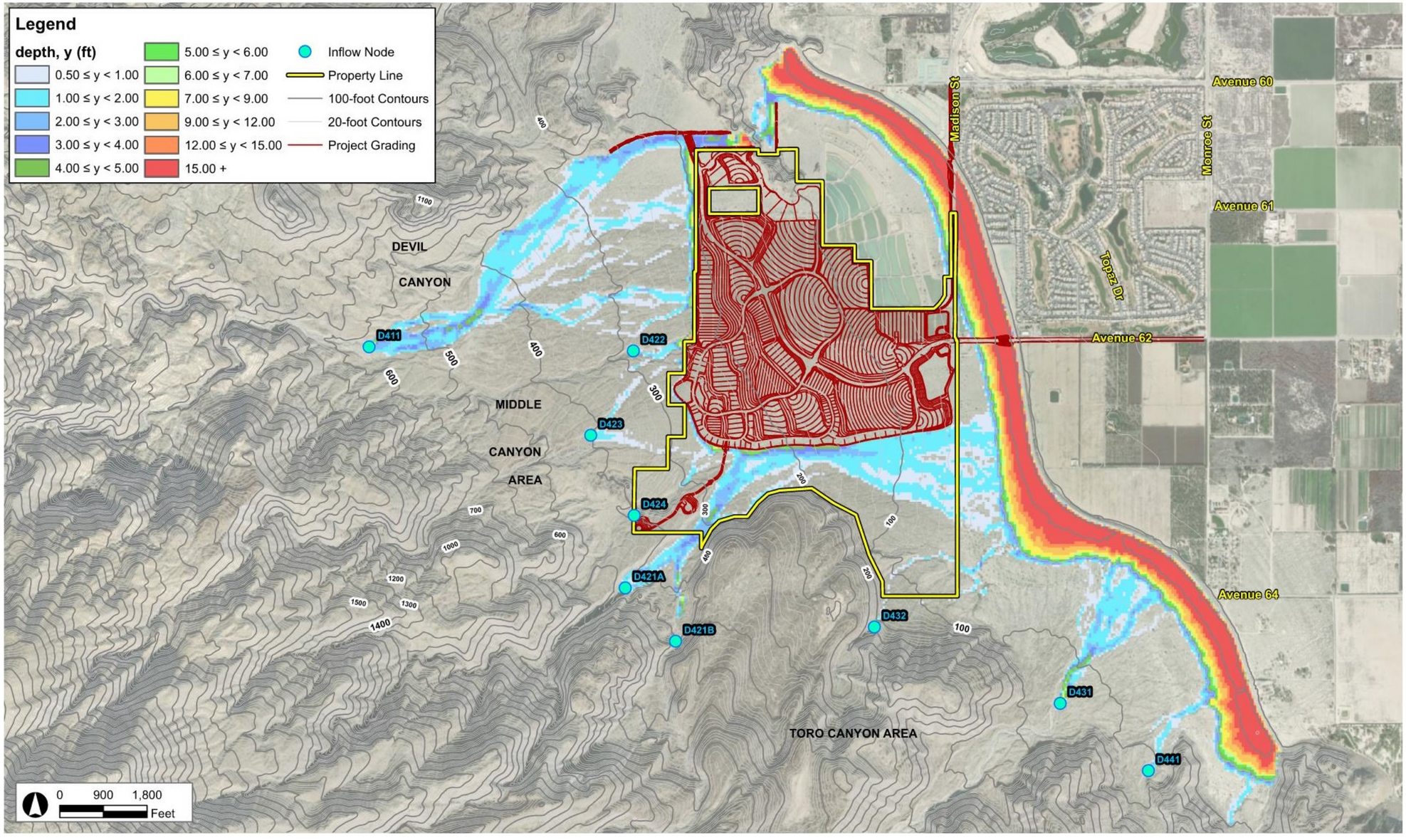
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Figure 5-23. Dike No. 4 baseline Standard Project Flood maximum depths ≥ 0.5 feet



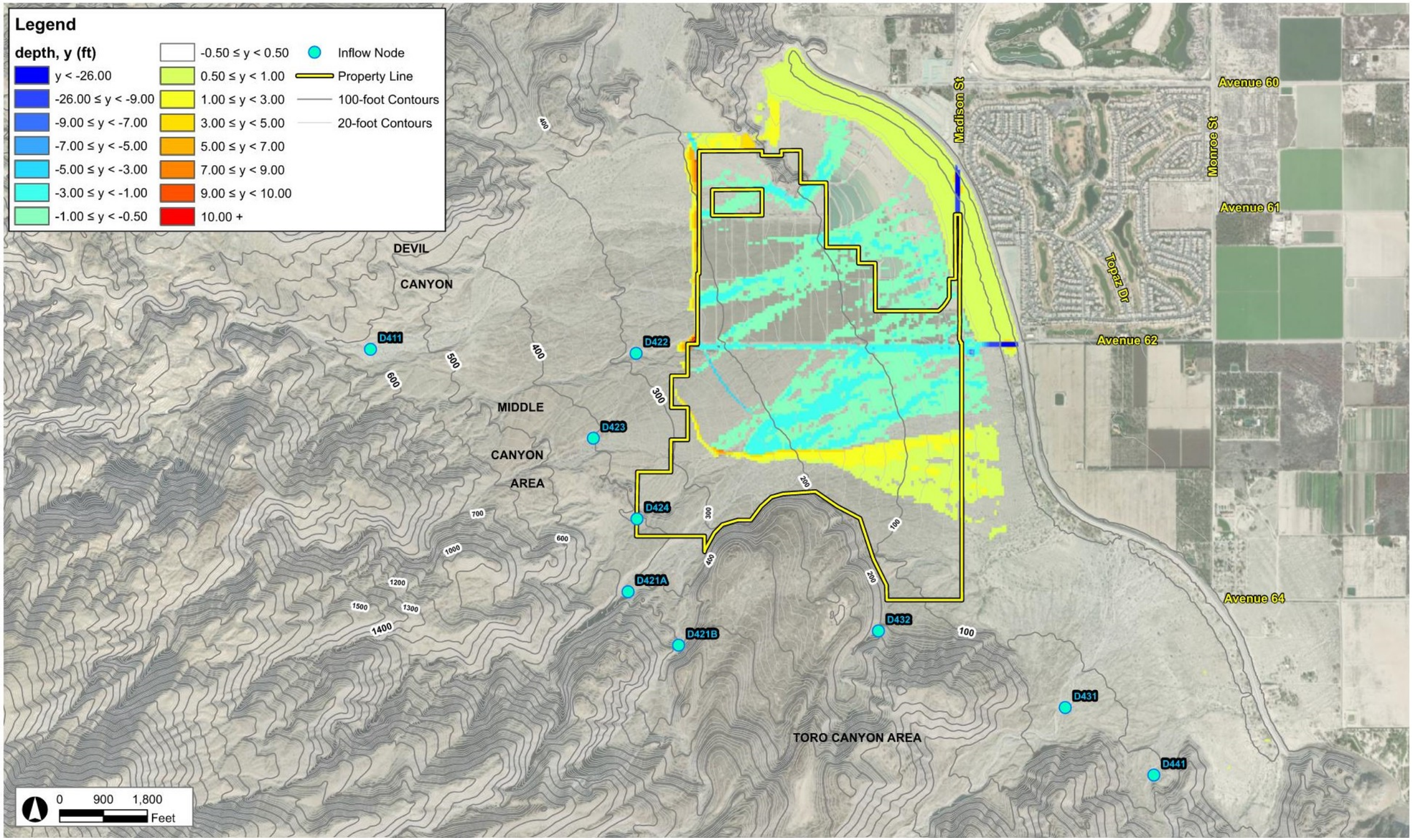
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Figure 5-24. Dike No. 4 project Standard Project Flood maximum depths ≥ 0.5 feet



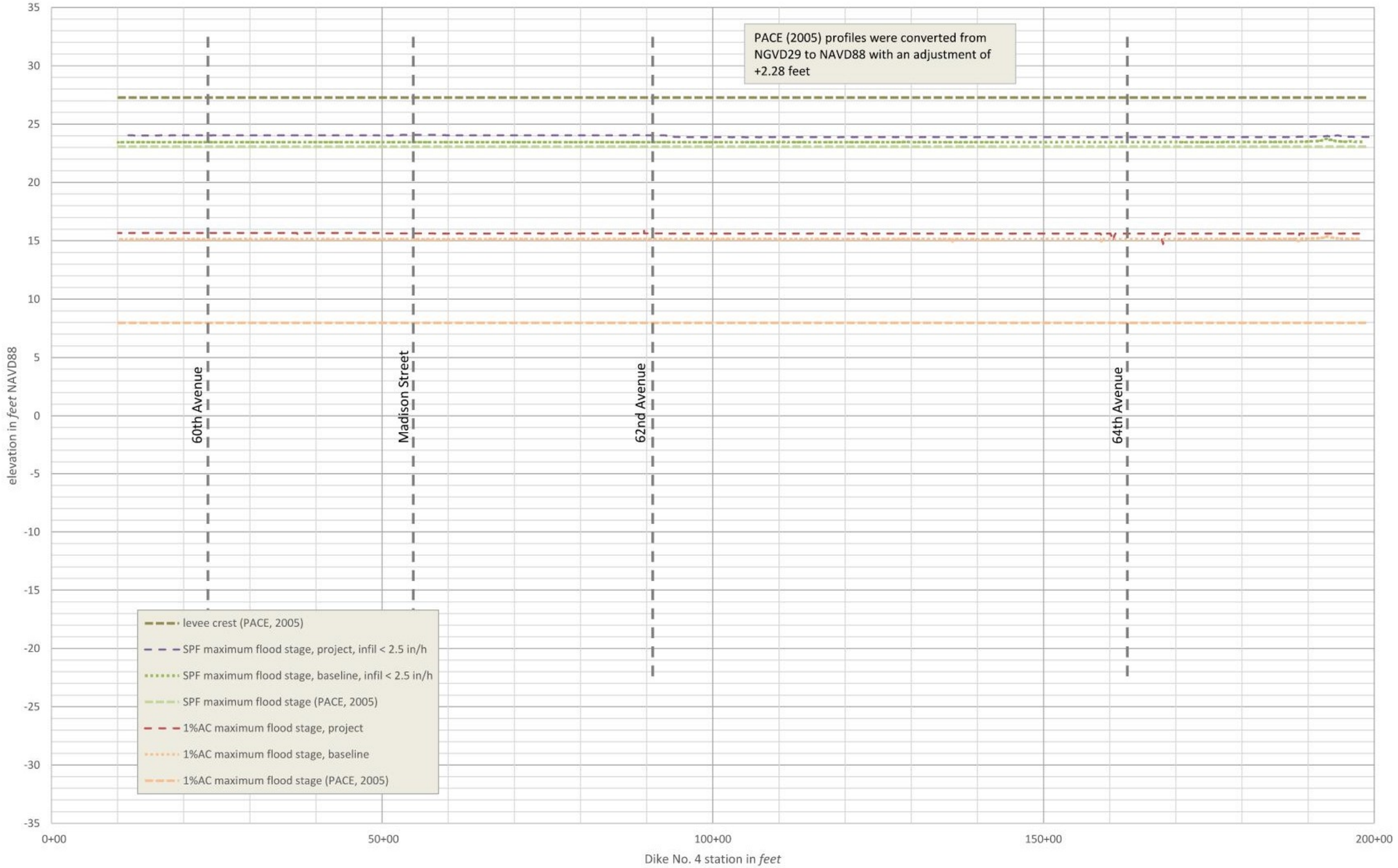
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Figure 5-25. Dike No. 4 change in Standard Project Flood maximum depths ≥ 0.5 feet from baseline to project conditions



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Figure 5-26. Comparison of water surface and ground elevation profiles along Dike No. 4



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Figure 5-27. 1-percent annual chance water surface profiles adjacent to the North Guadalupe Creek Diversion Dike

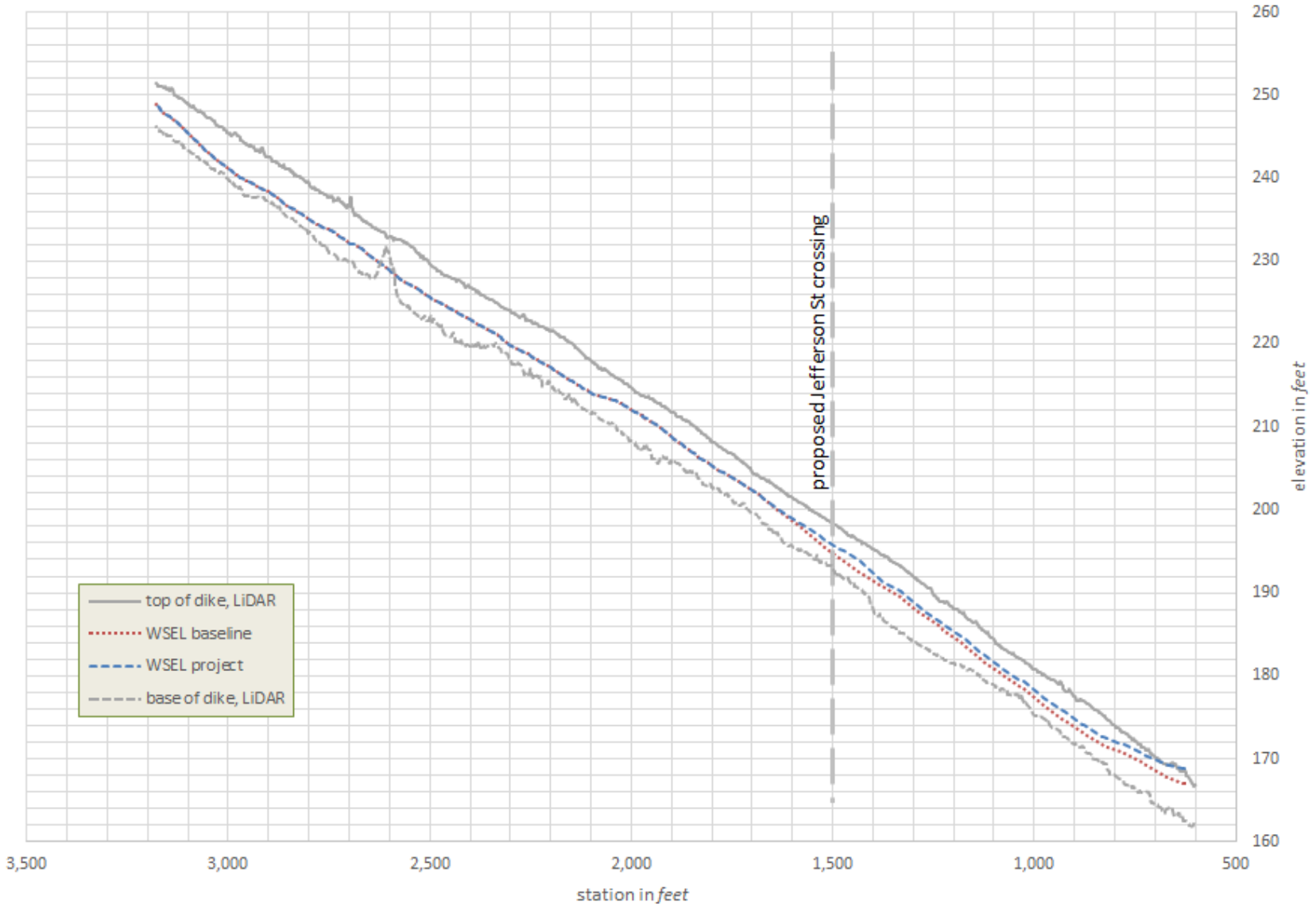


Figure 5-28. 1-percent annual chance water surface profiles adjacent to the South Guadalupe Creek Diversion Dike

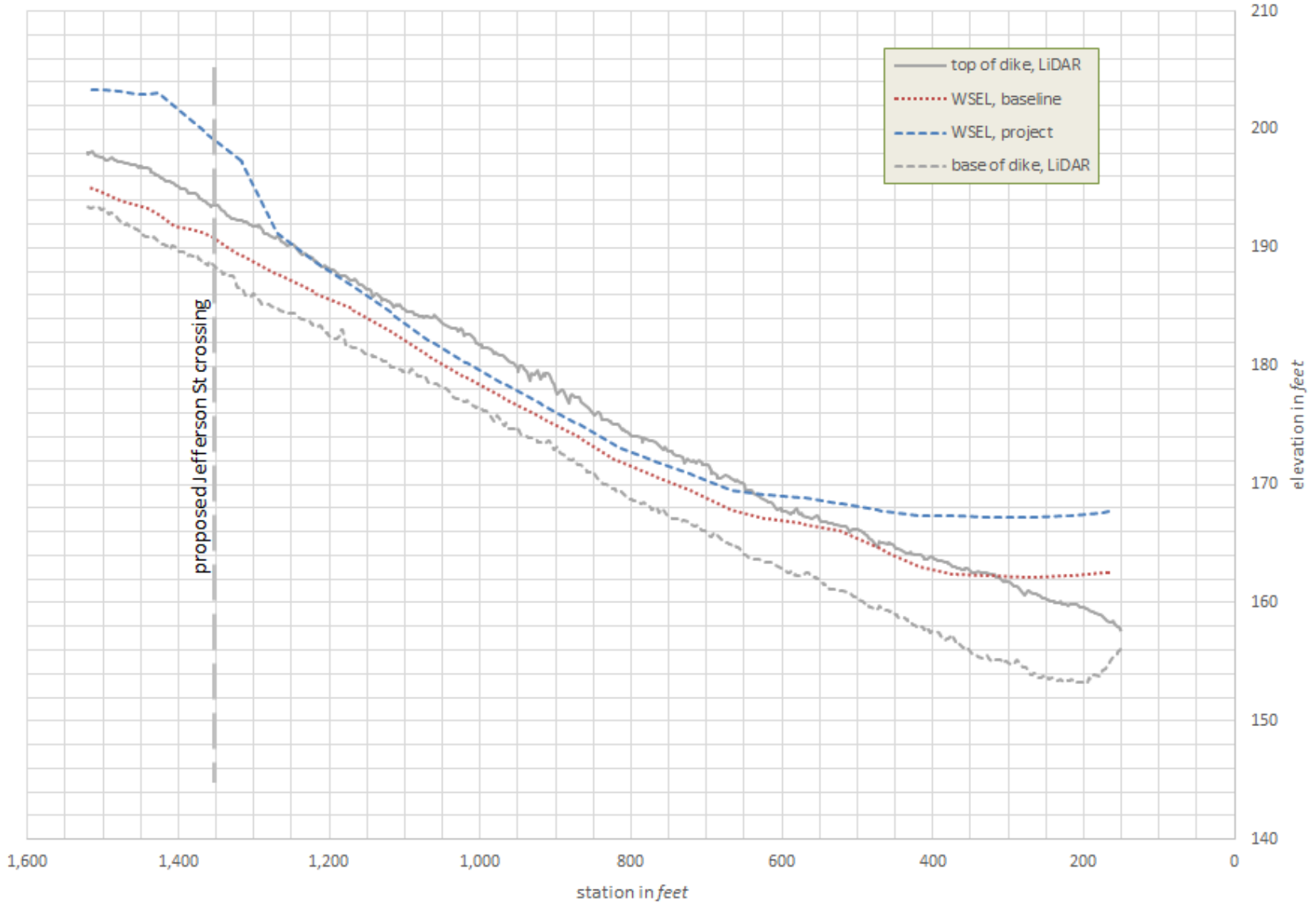


Figure 5-29. 1-percent annual chance water surface profiles along the centerline of Guadalupe Creek Channel

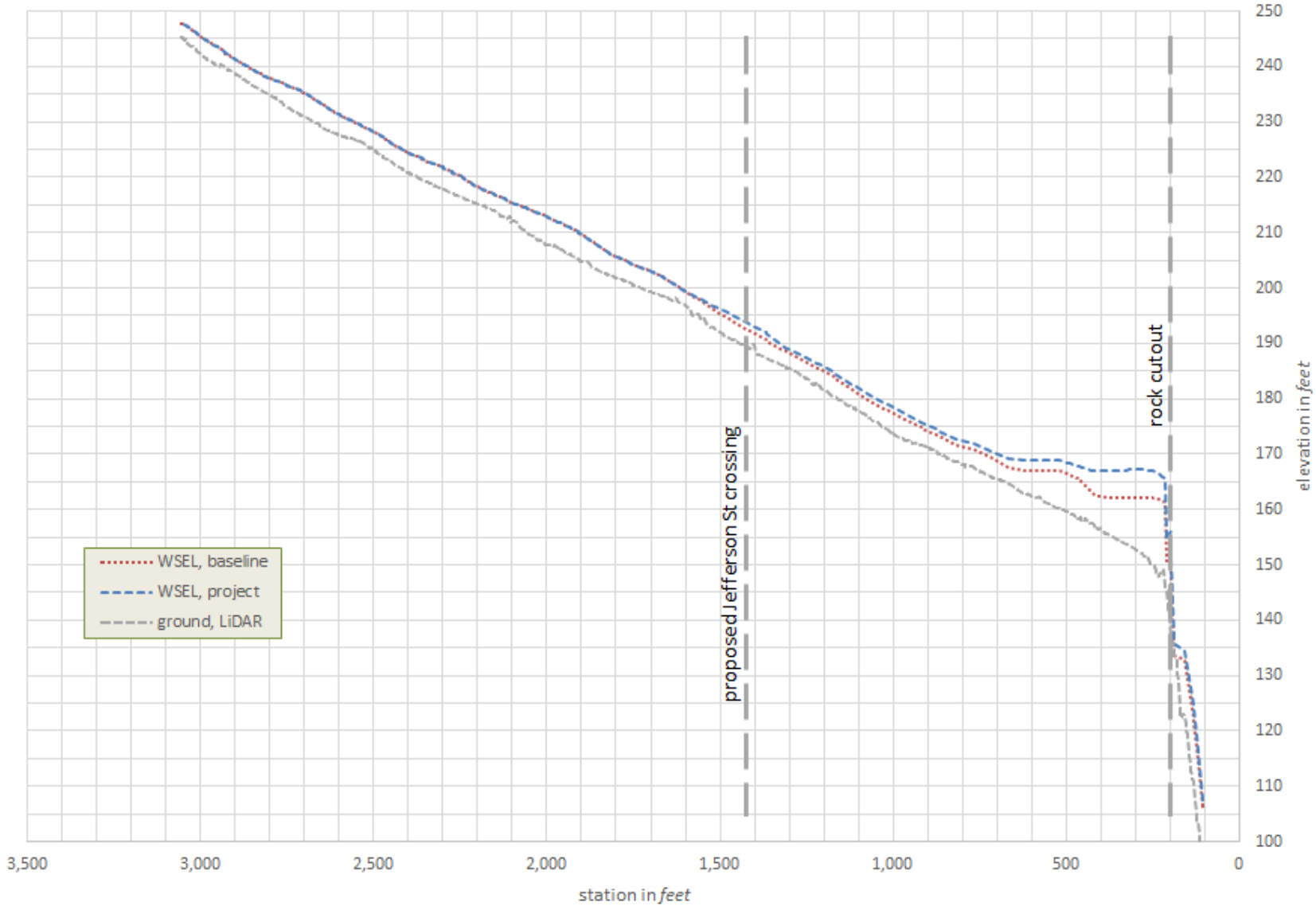


Figure 5-30. 1-percent annual chance flood depths and velocities adjacent to the North Guadalupe Creek Diversion Dike

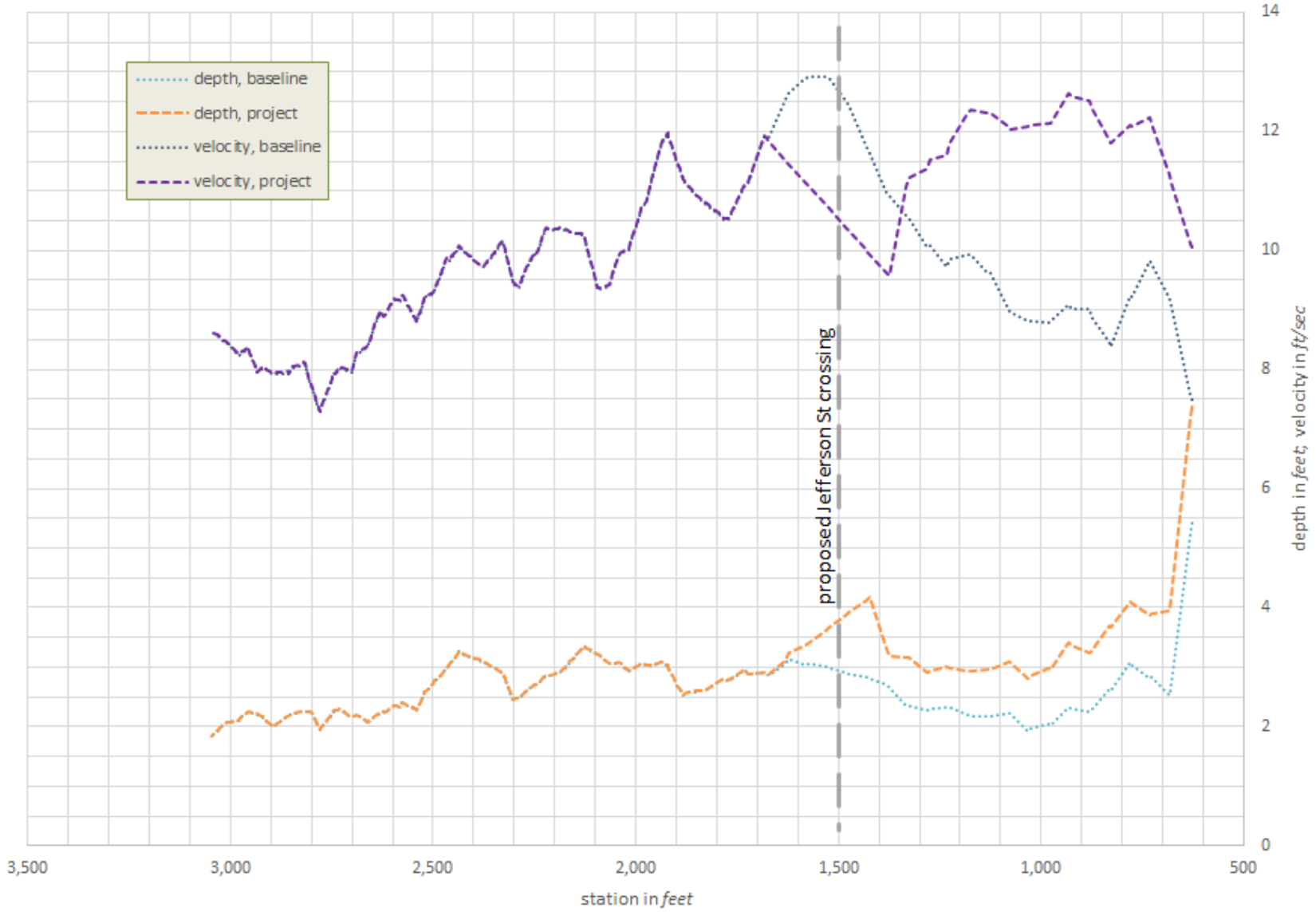


Figure 5-31. 1-percent annual chance flood depths and velocities adjacent to the South Guadalupe Creek Diversion Dike

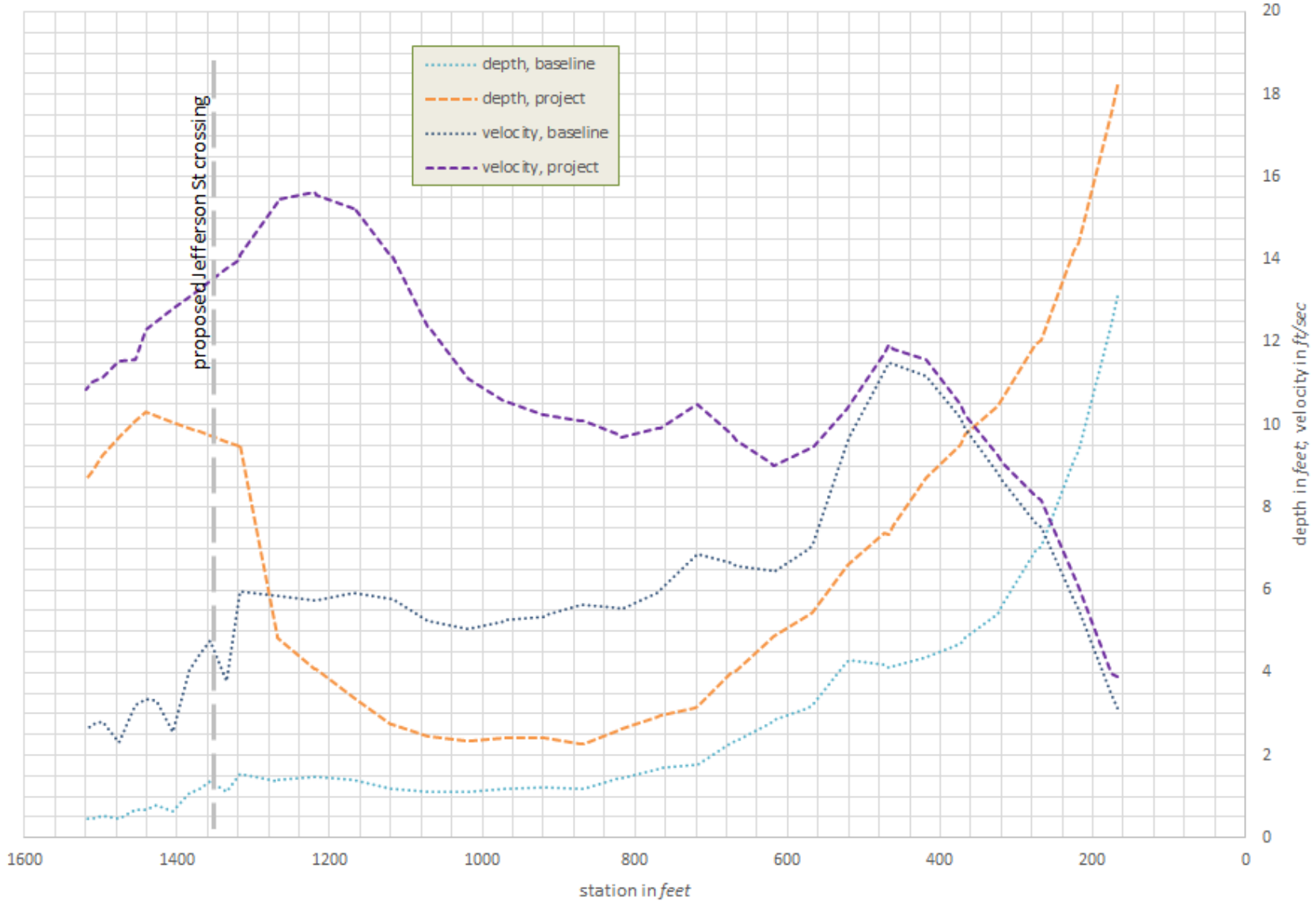


Figure 5-32. 1-percent annual chance flood depths and velocities along the centerline of Guadalupe Creek Channel

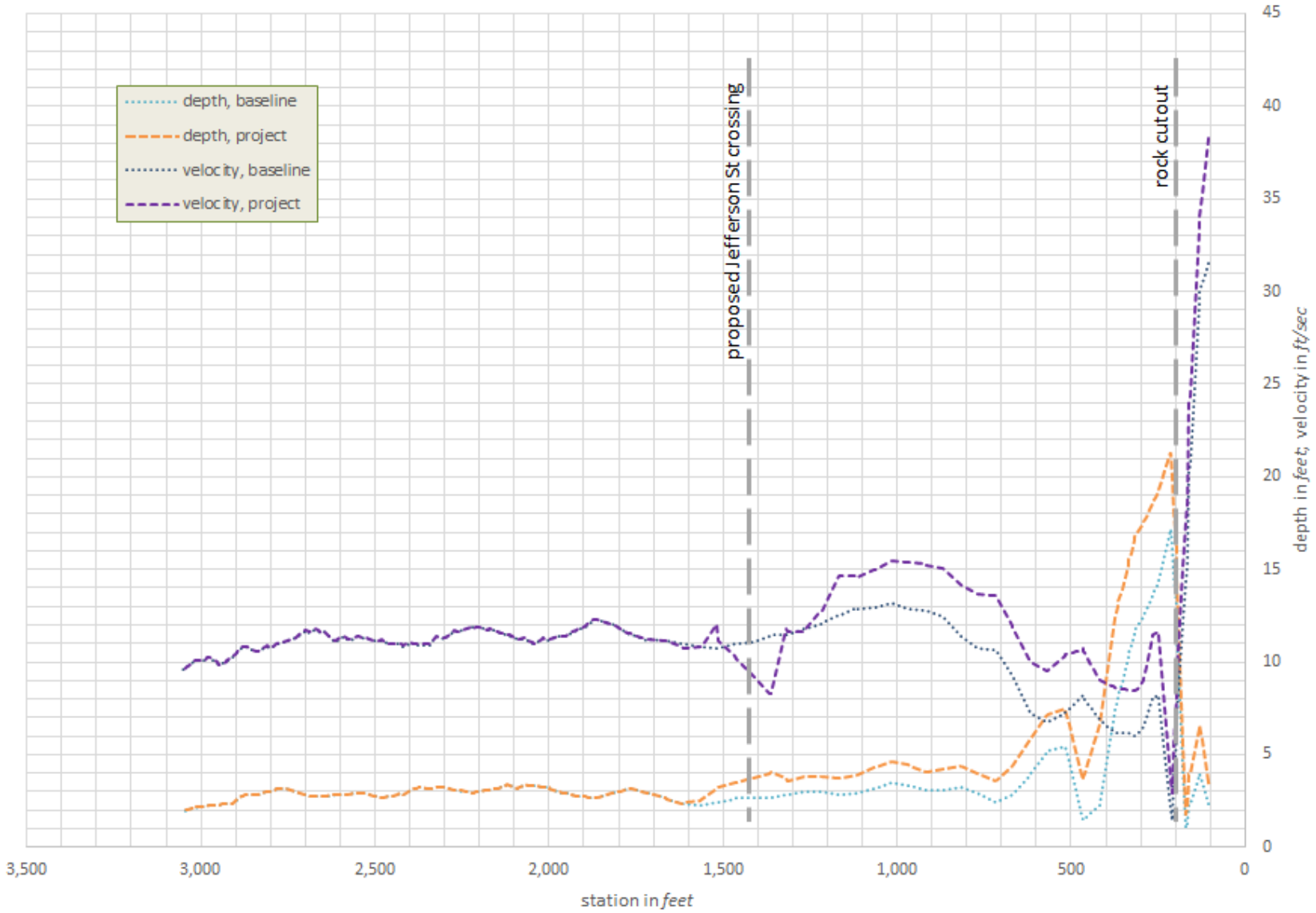
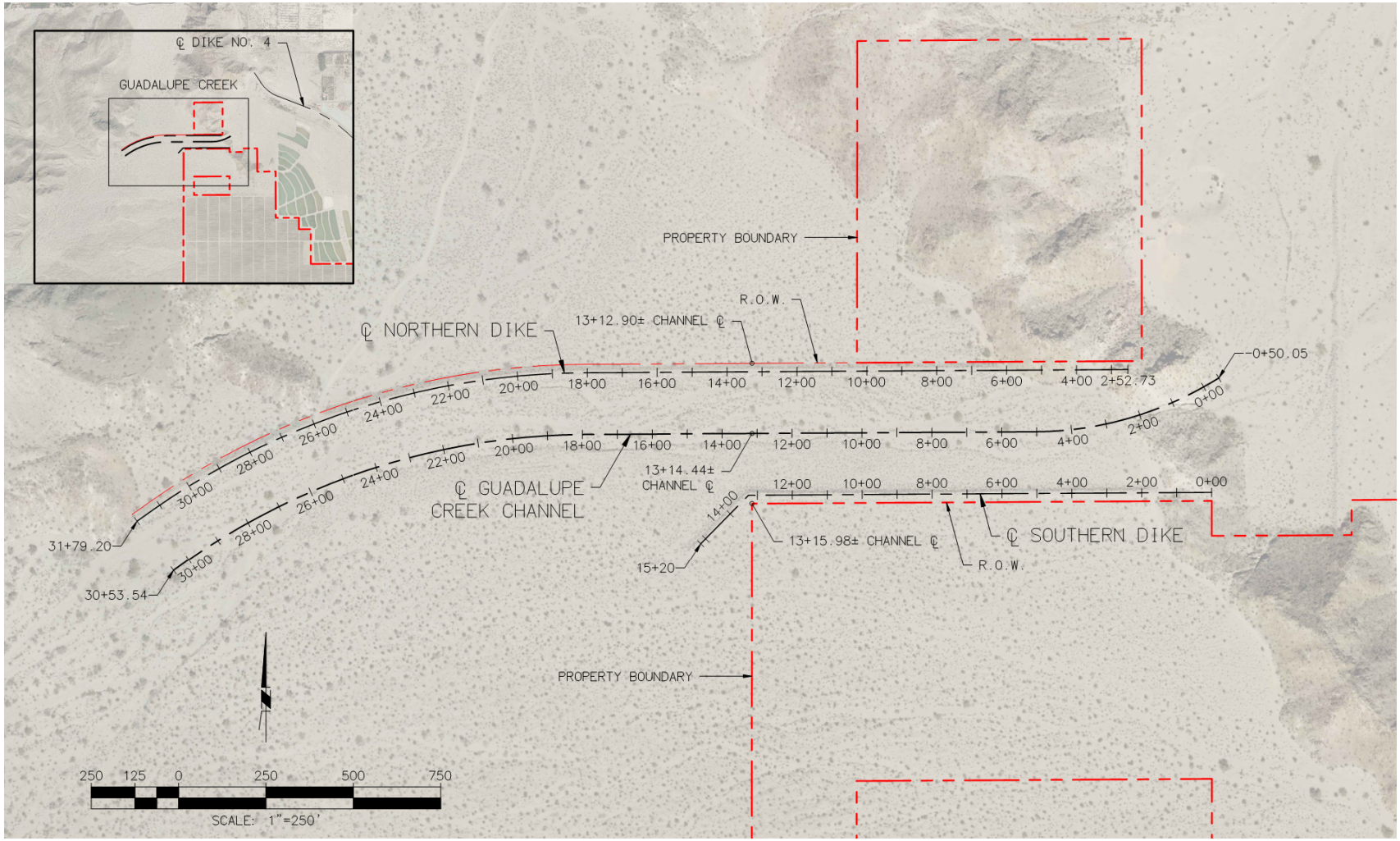


Figure 5-33. Plan view of upper Guadalupe Creek Diversion Dikes



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6 COMPARISON TO PREVIOUS STUDIES FOR WEST SIDE DIKE NO. 4

6.1 Regional hydrology

The hydrologic models developed as part of this study in support of the flood hazard evaluation performed for the proposed Travertine Development and effected resource facilities was used to analyzed the 1-percent annual chance storm event and SPS/SPF using the framework of the Synthetic Unit Hydrograph Method prescribed in the Riverside County Hydrology Manual (RCFCWCD, 1978), accompanied by the following implemented hydrologic processes and parameterization assumptions:

- **Watershed delineation**. Drainage divides were determined from IFSAR topographic data (Intermap Technologies, 2005)
- **Precipitation**. The 1-percent annual chance storm was represented using the 6-hour USACE hypothetical (synthetic) storm based on NOAA Atlas 14 spatial precipitation data; the SPS/SPF precipitation depth and storm pattern were based on the Indio Storm of September 24, 1939; the depth-area relationship based on the September 24, 1939 Indio Storm (USACE, 1980) was applied in the determination of the 1-percent annual chance flood and SPS/SPF for contributing drainages in excess of 10 square miles
- **Infiltration**. Precipitation losses were based on an initial abstraction and constant loss rate, but excluded the use of a low loss fraction (low loss rate) that is typically applied in Riverside County, but commonly ignored in the Coachella Valley.
- **Unit hydrograph**. Runoff response was determined in part using the lag formula developed for Southern California (RCFCWCD, 1978; USACE, 1962), which involved the following aspects:
  1. Topographic-based lag parameters were determined from IFSAR topographic data (Intermap Technologies, 2005)
  2. The basin factor, which has been historically related to hydraulic roughness was estimated from assumed correlations between hydraulic roughness and identifiable terrain characteristics (Travertine/developed areas, Mountain and hillslopes, and alluvial/relict surfaces/piedmont)
  3. The Whitewater S-graph (RCFCWCD, 1978; USACE, 1980) was used in conjunction with the lag to transform the unit hydrograph

Flood hydrographs were produced for the 1-percent annual chance storm event and Standard Project Flood at concentration points located at or near canyon outfalls above the Travertine property and along the base of the dike using HEC-HMS Version 4.2 (USACE, 2016).

6.1.1 Model development comparison – PACE (2005) vs. Travertine

Notable differences related to the hydrologic models developed for Travertine and PACE (2005) are as follows:

- PACE (2005) developed single-area flood hydrographs (no channel routing); the hydrology developed for Travertine relied on linked node models segregating major canyon subbasins from the piedmont drainage
- PACE (2005) globally assigned a lag basin roughness ("*n*-bar") value of 0.035 to all subbasins; for Travertine, the computed "*n*-bar" value for each subbasin, which ranged from 0.030 to 0.045 with an average value of 0.040.

6.1.2 Precipitation comparison – PACE (2005) vs. Travertine

The precipitation data used by PACE (2005) to develop flood hydrographs for the Dike No. 4 watershed was compared to the precipitation data used to develop the flood hydrographs for Travertine in Table 6-1. There is very little difference in the parameterization for the SPS/SPF; however, for the 1-percent annual

chance storm, PACE (2005) relied on NOAA Atlas 2 (RCFCWCD, 1978; NWS, 1973) frequency-duration precipitation mapping in conjunction with the Cathedral City precipitation gauge record to estimate the precipitation depths for the watershed, which were substantially less than what was estimated from the NOAA Atlas 14 (NWS, 2014) frequency-duration precipitation spatial datasets used for Travertine. NOAA Atlas 14 was not available until 2006, which followed the completion of the LOMR for Dike No. 4 (PACE, 2005).

Table 6-1. Precipitation comparison – PACE (2005) vs. Travertine

parameter		PACE {2005}	Travertine	
			baseline	project
drainage area {sq mi}		27.7	27.07	
SPF	P {inches}	6.45		
	DAR	Bechtel {1997}	USACE {1980}	
		0.82	0.93	
	P_{DAR} {inches}	5.29	6.00	
	storm pattern	September 24, 1939 Indio 6-hour storm		
100-year storm	P {inches}	NA2 {RCFCWCD, 1978}	NA14 {NWS, 2014}	
		2.91	3.83	
	DAR	NA2 {RCFCWCD, 1978}	USACE {1980}	
		0.94	0.93	
	P_{DAR} {inches}	2.74	3.56	
	storm pattern	6-hour hypothetical storm based on Cathedral City gauge	synthetic 6-hour storm based on NA14 {NWS, 2014}	
loss rate {in/h}		0.200	0.286	0.278

6.2 Flood conveyance and impoundment

Two-dimensional flood routing model simulations were performed, using FLO-2D PRO, to evaluate the flood intensities and patterns on the piedmont as well as impoundment of floodwaters along Dike No. 4.

The baseline and project conditions along Dike No. 4 satisfy the minimum freeboard requirements for the Standard Project Flood (one foot) and the 1-percent annual chance flood event (four feet) as stated in Ordinance 1234.1. No outflow was permitted from the impoundment. Storage loss due to sedimentation was accounted for by bulking flood hydrographs for 100 percent of the computed debris yield. For the Standard Project Flood simulations, the Green-Ampt parameters were adjusted to produce an average infiltration rate over the 24-hour simulation period such that the constant infiltration rate of 2.5 inches per hour originally assumed by PACE (2005) is not exceeded along the base of Dike No. 4. The resultant average infiltration rate over the 24-hour simulation period was approximately 2.4 inches per hour for each set of conditions modeled.

The map of 1-percent annual chance flood depths is generally consistent with the geomorphic mapping of active and inactive fan surfaces shown previously in Figure 3-4 and Figure 3-5. Portions of the mapped active areas did not flood or were limited to very shallow flooding less than 0.5 feet in depth and is of a non-erosive nature, which is consistent with the geomorphic observations indicating that portions of active fans have not been exposed to significant flood processes in hundreds of years. Some portions of the inactive fan below Devil Canyon were subject to overflow; the most impactful "breakout" location (DC1) was further evaluated by forcing all flows within the active floodplain onto the overflow terrace.

Flood depths and velocities for the baseline and project conditions were determined and mapped to support site planning and the design of flood conveyances required to provide flood protection for the planned development. These maps are not intended for regulatory floodplain evaluation.

6.2.1 Flood runoff and debris comparison – PACE (2005) vs. Travertine

The computed runoff volume and debris as determined by PACE (2005) was compared to the computed runoff volume and debris as determined for Travertine and is presented in Table 6-2.

There is a 2.6 percent increase in the SPS/SPF net bulked volume for the Travertine baseline conditions relative to the results reported by PACE (2005). There is less than a 0.3 percent increase in the net bulked volume going from baseline conditions to project conditions.

Table 6-2. Flood runoff and debris comparison – PACE (2005) vs. Travertine

parameter		PACE {2005}	Travertine	
			baseline	project
fire factor		3.0		
A-T factor		1.00	0.52	
SPF	water volume {ac-ft}	6,108	6,087	6,121
	debris yield {ac-ft}	1,060	723	708
	delivery efficiency {%}	50	100	
	net debris yield {ac-ft}	530	723	708
	net bulked volume {ac-ft}	6,638	6,810	6,829
100-year storm	water volume {ac-ft}	1,649	3,009	3,020
	debris yield {ac-ft}	509	710	698
	delivery efficiency {%}	100		
	net debris yield {ac-ft}	509	710	698
	net bulked volume {ac-ft}	2,158	3,719	3,718

6.2.2 Flood stage comparison

The results of the storage analysis performed by PACE (2005) were compared to the results of a similar analysis conducted for Travertine, as shown in Table 6-3. For Travertine, a two-dimensional flood routing model was used to analyze the impoundment of floodwaters along Dike No. 4. This scheme captures the effects related to the unbalanced delivery of floodwaters to the base of Dike No.4, resulting in a variable maximum water surface profile along the dike; PACE (2005), on the other hand, previously evaluated the flood storage along Dike No. 4 based on a level-pool analysis, which produces a single maximum water surface elevation for the entire length of the dike.

Table 6-3. Dike No. 4 flood stage comparison – PACE (2005) vs. Travertine

parameter		PACE {2005}	Travertinr	
			baseline	project
levee crest {feet}		27.28		
SPF	infiltration {in/h}	2.4		
	maximum WSEL {feet}	22.58	23.55	24.13
	average WSEL {feet}	22.58	23.49	23.82
	min. freeboard {feet}	4.70	3.73	3.15
100-year storm	infiltration {in/h}	0.0		
	maximum WSEL {feet}	9.18	15.18	15.53
	average WSEL {feet}	9.18	15.15	15.47
	min. freeboard {feet}	18.10	12.10	11.75

Note: all elevations based on NAVD88

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7 FLOOD HAZARD IMPACTS AND MITIGATION

7.1 General

This chapter includes a summary of the impacts associated with the planned project and a discussion of the mitigation measures proposed to address the project impacts and provide flood risk management to protect the project improvements. A detailed flood hazard assessment has been prepared to identify the expected flood hazards at the project site and affected vicinity (adjacent properties and regional facilities), resulting from the 100-year and SPF storm events. The geomorphic conditions, expected flow rates, depths, and velocities for the assessment of the Project impacts have been identified in the previous chapters. The purpose of this chapter is to identify and discuss the project impacts and develop the necessary mitigation measures and conceptual flood protection systems for the Project to convey the floodwaters around the site and avoid significant flood hazard impacts to adjacent properties and downstream facilities.

The Specific Plan Area is located on one of the eastern piedmonts along the base of the Santa Rosa Mountains, above the flood impoundment zone of West Side System Dike No. 4 (BOR, 1947; Bechtel, 1991), and adjacent to the Guadalupe Creek Diversion Dikes (CVCWD, 1968; Bechtel, 1991) to the north. The piedmont bajada on which the site is located is composed of moderately steep-sloped relict (inactive) and active alluvial fans. These site conditions and existing flood control infrastructure are key considerations used in the development of the drainage master plan.

7.2 Flood Hazard Mitigation Plan

7.2.1 Alluvial Fan Flood Protection Measures

Three general approaches are typically used for flood management on an alluvial fan. These approaches are:

1. Whole Fan Protection
2. Subdivision or Localized Protection
3. Single Lot/Structure Protection

Whole fan protection would generally be considered as a regional flood protection system undertaken by a government agency or collection of landowners. Whole fan protection can be achieved through the use of levees, channels, detention basins and/or dams. This method includes large-scale structural measures appropriate to use on extensively developed fans and are more cost-effective in high-density situations. Structures must be designed to intercept watershed flow and debris at the apex and transport the flow around the entire urbanized fan. These structures are most often financed through federal or state sources, but can also be financed through special regional districts, local governments, or developers.

Subdivision or localized protection can be provided to protect projects in the absence of whole fan protection. These measures may include; local dikes, conveyance channels/swales, site plans to convey flow, streets design to convey flow, or elevation on armored fill. These are smaller scale measures that can be used for moderate density development on alluvial fans and are designed to convey water and debris around or through the individual development.

Single lot or structure protection measures are for protection of isolated lots or structures. These measures may include elevated or properly designed foundations, or floodwalls and berms. The measures are most effective when used for low-density developments.

The Project is located on active and inactive alluvial fan surfaces, and subject to the requirements to convey offsite flow through or around the site development. Therefore, alluvial fan flood protection

measures need to be incorporated into the project to convey the anticipated flood flows, and prevent adverse floodplain impacts to adjacent properties and downstream facilities.

Since the development area of the project is only a portion of the piedmont bajada of active and inactive alluvial fans, localized subdivision protection is proposed to protect the project in the absence of any whole fan protection system. Mitigation measures for the site development will also be identified to address any project related impacts on the existing flood protection systems including the West Side Dike System No. 4, and the Guadalupe Creek Diversion Dikes.

7.2.2 Flood Protection System Selection

The Travertine project is the only currently planned development upstream of the West Side Dike No. 4. Currently, no whole fan (regional) flood protection systems exist in this area. The Project is an isolated development on the piedmont bajada, therefore, a regional flood protection system of improvements from the fan apex to the base of the dike would not be practical or appropriate as it would result in greater environmental impacts. Single lot or structure protection is also not appropriate due to the size of the planned development. As such, subdivision or localized protection is necessary for the Project, which includes collection, conveyance, and redistribution of flood flows.

Physical and legal constraints also govern the types of flood protection measures that can be used for the site development. The Project site is located along a poorly defined drainage corridor near the outlet of the Devil, Middle (North and South), and Rock Avalanche Canyons, and conveys runoff from the canyons through the project site to Dike No. 4. The Project is bounded by the Guadalupe Creek Diversion Dike No. 2 to the north, and the CVWD Infiltration Ponds and Dike No. 4 to the east. The west and south sides of the project are subject to active and inactive alluvial fan flows. Based on the available tools and project constraints, the proposed regional flood protection measures include a combination of perimeter embankments and drainage swales along the west and south site development boundaries, and improvements to the Guadalupe Creek Diversion Dikes on the north side. The on-site plan will be designed to capture and convey on-site runoff and discharge to water quality treatment facilities prior to releasing upstream of Dike No. 4.

7.2.3 Conceptual Flood Protection System

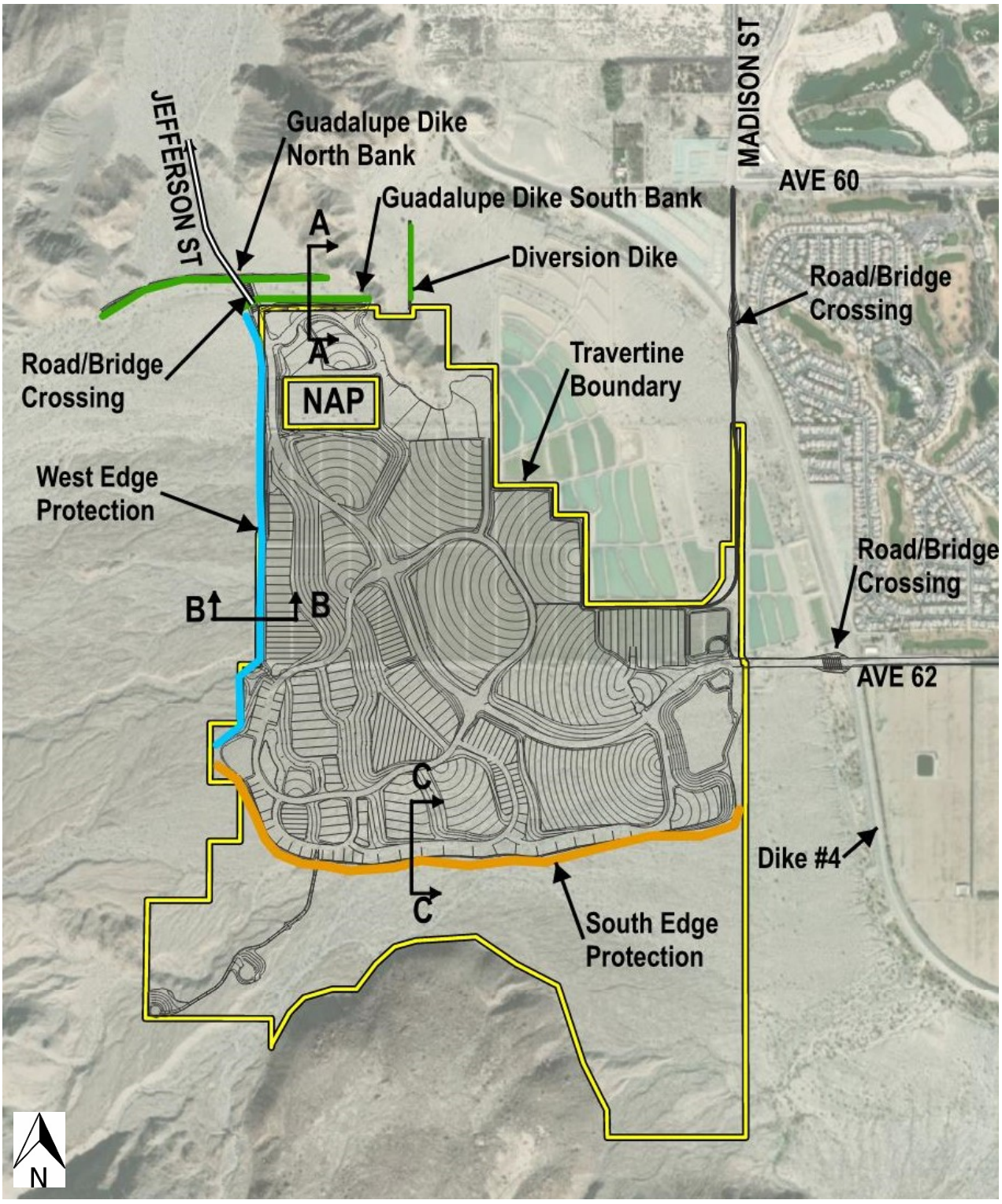
A conceptual flood protection system has been identified and incorporated into the proposed Travertine site plan to intercept flood waters along the west and south boundary and convey the floodwaters around the site development area. The Project proposes to use a protected embankment, elevated fill, and a graded swale along the west boundary to collect and convey the unconfined alluvial fan flows (Devil and the small un-named canyon) to the north side of the site and into the Guadalupe Creek Diversion Dikes. The dikes are proposed to be improved to accommodate the design flows rates and upgraded to meet Federal requirements for a levee system. It is not anticipated that the existing Guadalupe Creek Diversion Dikes would meet Federal criteria for an accredited levee system.

The runoff from the Middle North and South, and Rock Avalanche Canyons would be intercepted along the south edge of the planned development. A protected embankment and elevated fill would be used to convey flows easterly along the south boundary to Dike No. 4.

The Conceptual drainage plan for the Travertine development ensures that all residents of the community, as well as downstream facilities and properties, will be protected from periodic flooding that is experienced in the region. Figures 5-8 (composite 1-percent annual chance event maximum depth) and 5-15 (composite 1-percent annual chance event maximum velocities) depicted the existing flow conditions that come on to the project site and were used for the conceptual design of the flood protection system.

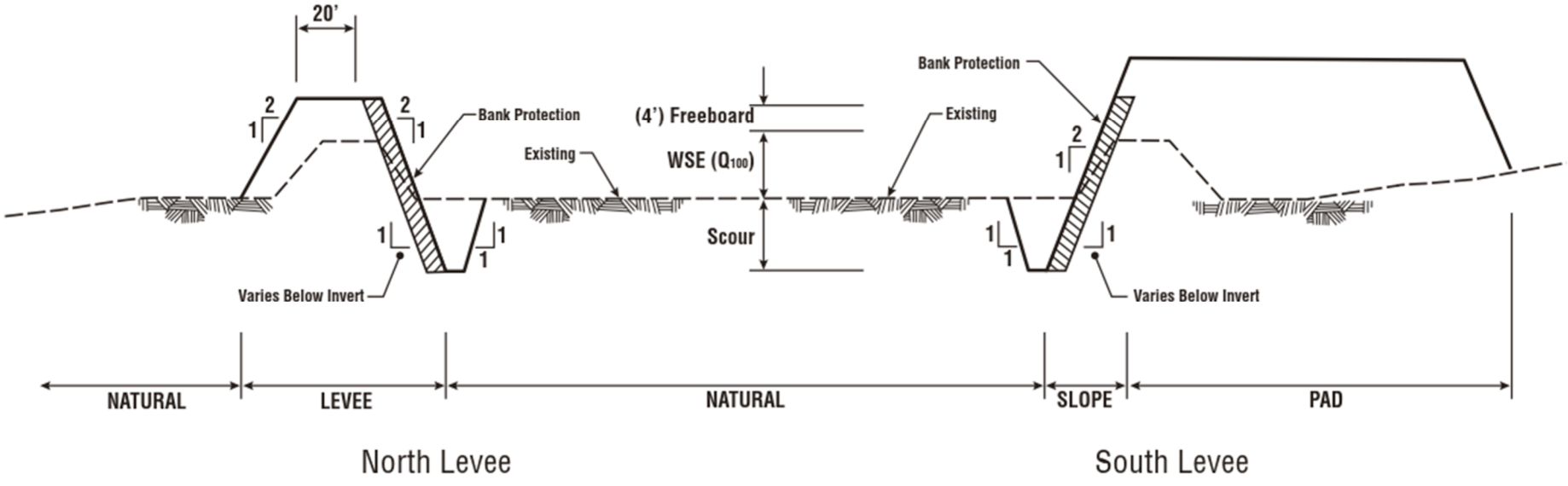
The Project site plan with the proposed flood protection system is shown on Figure 7-1, Flood Protection Plan. Typical sections along the perimeter facilities are shown in Figures 7-2 and 7-3.

Figure 7-1. Flood protection plan



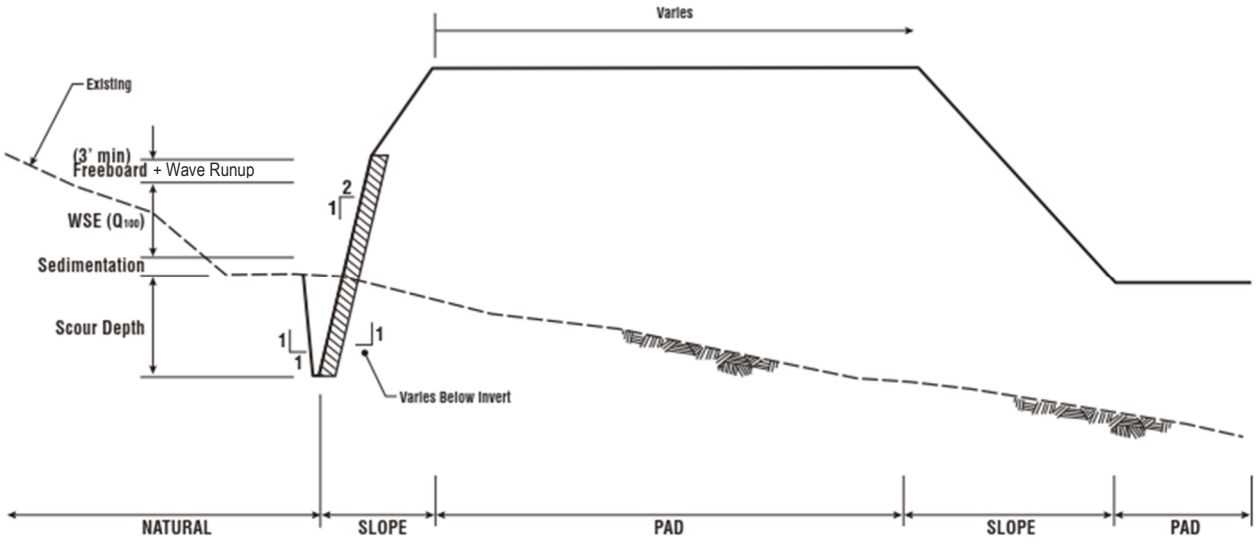
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Figure 7-2. Flood conveyance typical sections- Guadalupe Diversion Dike

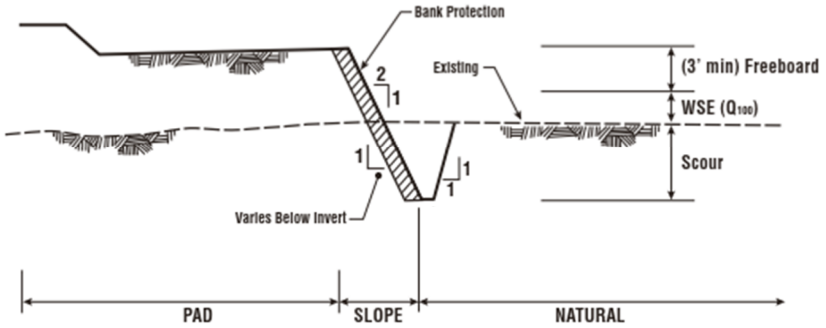


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Figure 7-3. Flood conveyance typical sections- West and South banks



West Edge
SECTION B-B



South Edge
SECTION C-C

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7.3 Project Impacts

7.3.1 West Side Dike No. 4

The proposed project will have little to no impact on the runoff volumes generated from the watershed tributary to the Dike No. 4. Table 6-2 provides a summary of the net bulked runoff volume tributary to Dike No. 4. The net bulked volume for the 1-percent annual chance event is effectively the same in the existing and project conditions. The increase in clear-water runoff volume associated with the site development is effectively neutralized by the resultant reduction in sediment yield associated with the development. Therefore, the net bulked volume is essentially the same at 3,718 acre-feet. During an SPF event, the net bulked runoff volume is increased from 6,810 acre-feet to 6,829 acre-feet in the project condition. This is an increase of only 0.3%.

The project will include the extension of Avenue 62 and Madison Street over Dike No. 4, and a minor re-direction of flow from the unnamed canyon (Node 422) to the Guadalupe Dikes. These project elements will have a minor impact on the maximum flood stage profile along the Dike. The results of the flood routing analysis and potential impacts are summarized in Table 6-3. In the SPF event, the maximum water surface depth is increased from 23.55 feet to 24.13 feet in the project condition. During the 1-percent annual chance event, the maximum water surface depth increases from 15.18 to 15.53 feet, resulting in a minimum freeboard of 11.75 feet. The increased depth still provides for a minimum freeboard of 3.15 feet which far exceeds the prior criteria of one foot adopted by CVWD for the SPF event, and the 1-percent annual chance event freeboard of 11.75 feet also far exceeds to current standard of 4-feet. In addition, the flood routing analysis was prepared using the total runoff volume from the existing and project conditions. The onsite improvements will include storm water retention and infiltration to meet the water quality requirements. This storage volume will reduce the runoff volume generated in the project condition and mitigate some of the impact to the water surface profile. The remaining impact is not significant in terms of the function and operation of the Dike No. 4.

The final design of the Avenue 62 and the Madison Street crossings of the dike shall be evaluated to ensure that a significant change in the water surface profile along the dike does not occur. The final design shall also review the geotechnical analyses used to certify the dike to ensure that the change in water surface elevation does not adversely impact the stability or seepage analyses.

7.3.2 Guadalupe Creek Diversion Dikes

The Guadalupe dikes were constructed in 1968. No documentation for the design was available, however, it is reasonable to assume that the facility was designed to handle the total flow from the Devil Canyon watershed. The construction of the proposed project will also result in additional flow from the unnamed canyon being diverted to the Guadalupe dikes. The 1-percent annual chance peak flow rate (bulked) to the dike from Devil Canyon was determined to be 12,615 cfs at node J41 (Table 4-13). With the proposed improvements, the total peak flow rate (bulked) tributary to the dike was determined from the flood routing analysis to be 13,196 cfs. This is an increase of less than 5 percent. The resultant change in the water surface profile, flood depths, and flow velocities are shown on Figures 5-24 through 5-30. The impacts to the dike are generally located downstream of the proposed Jefferson Road crossing, where the flow diversion occurs. Above this location, the flow depths and velocities along the northern dike are similar between the existing and project conditions.

The Guadalupe Creek Diversion Dikes downstream of the diversion are proposed to be improved as a part of the project. This will mitigate any adverse impacts to the dikes associated with the project. The dikes will be designed to convey the new flow rates with the freeboard and scour protection as required by CVWD, and in accordance with Federal standards for levee certification.

7.3.3 California Drainage Law

California Drainage Law states that property owners have the right to protect themselves from flooding as long as they do not unreasonably increase flood risk for adjacent property owners. To accomplish this, CVWD has identified that flows must be reasonably received and released in the historical flow paths at the historical flow depths and velocities (CVWD, 2020).

The 2-dimensional hydraulic analysis completed for the project was used to evaluate the drainage impacts to the adjacent parcels. An impact analysis of the 1-percent annual chance storm event was prepared using the results of the hydraulic modeling. The baseline (pre-project) flooding depths and velocities were compared with the project conditions to identify the change in flow depth and velocities as a result of the proposed improvements. The project impacts are illustrated on Figures 5-12 (depth) and 5-19 (velocity).

The primary impacts from the project are on the adjacent parcels owned by the United States Bureau of Land Management (BLM) along the West Bank, and the United States Bureau of Reclamation (BoR) lands along the Guadalupe Dike and Dike No. 4. The project team is in discussions with both agencies to permit the proposed improvements and associated drainage impacts on the governmental lands. Minor impacts were also identified to a few parcels along the eastern property line south of 62 Avenue. These impacts include a minor increase in the flow depths and velocities. The majority of the properties along the eastern property line will see a significant reduction in the potential flood hazard as a result of the project. These include parcels owned by CVWD, BoR, and a private property owner.

The final impacts will be identified and coordinated with the property owners during the Final Map stages.

7.4 Design Requirements

The conceptual design and layout for the proposed flood protection for the project has been developed and evaluated as a part of this master plan. More detailed engineering and design, consistent with design standards established by the City and the Coachella Valley Water District, will be completed at the Final Map stages of development, resulting in the precise location, alignment, and sizing of all regional drainage facilities. The objectives for the flood control and drainage facilities throughout the Travertine properties are:

- Protect all buildings from damage from the 100-year storm in any of the drainage areas that cross or are within the property.
- Safely discharge all flows leaving the property.
- Control and manage runoff and sediment flow around the site.
- Mitigate adverse impacts to existing facilities.
- Assure the reliable operation of each drainage feature through a full range of flow events.
- Where levees, channels, bridge, or embankments are used, provide adequate freeboard and scour.
- Assure that embankment, levee, and channel lining or stabilization is used to control scouring of channel side slopes or inverts.

The following summarizes the requirements and criteria to be evaluated as a part of the more detailed facility design. The Regional Drainage System is assumed to include the West and South Bank Protection, Guadalupe Creek Diversion Dikes, and the Jefferson, Avenue 62 and Madison Street Bridge crossings.

Regional Drainage System Design Requirements:

1. All facilities shall be designed in accordance with the latest version of the CVWD *Development Design Manual.*

2. The regional hydrology analysis identified in Chapter 4 – Regional Hydrology of this report is acceptable for use in the final design. Regional facilities shall be designed using the bulked 1-percent annual chance event.

3. Updated hydraulic analyses utilizing a refined grid-cell size and detailed topography, grading and facility alignments shall be prepared to determine design water surface elevations and flow velocities along the perimeter flood barriers and Guadalupe Diversion Dikes.

4. Evaluate flow depths and velocities on a reach by reach basis to determine: a) water surface elevations, b) freeboard requirements, c) lining requirements in terms of materials and lining thickness, d) scour depths, e) potential for deposition of sediments, and f) the need for channel stabilization to control degradation or bed incision.

5. Adjust flood protection system configuration (in terms of barrier and levee heights/scour depths and bridge crossing configurations) based on the refined hydraulic analysis. Determine the optimum configuration of channels, barriers, and levees with necessary containment and erosion control structures which will provide the 100-year flood protection.

6. Bridges at the Jefferson Road crossing of the Guadalupe Dike and the Avenue 62 and Madison Street crossings of Dike No. 4 shall be designed in accordance with the scour requirements in Section K-3.11 of the Development Design Manual.

7. Prepare detailed designs and specifications for facilities including levee improvements, erosion protection (natural appearing where possible), and channel stabilization structures for the required facilities.

8. Prepare an Operations and Maintenance (O&M) plan for the regional flood protection system facilities in conformance to the requirements in Section 8, *Design Criteria Stormwater Facility* of the Development Design Manual.

9. Obtain a Conditional Letter of Map Revision (CLOMR) from the FEMA for the areas of the site within the mapped floodplain areas prior to the start of grading operations. Obtain a Letter of Map Revision (LOMR) to remove the areas of the site within the Special Flood Hazard Areas (SFHA) prior to occupancy.

Consideration of re-naturalization, preservation of natural features, and reduction of visual impacts will be made during the various stages of the final design. In addition, all drainage facilities for on-site drainage will be designed following the same process.

The following sections provide additional details and information for the design of the individual regional and local drainage facilities.

7.4.1 West and South Bank Protection

The west and south banks are subject to active and inactive alluvial fan flows. The volume and peak flow rates tributary to these boundaries have been determined as a part of the hydrology and hydraulic analysis completed for this master plan. The worst-case scenarios along these edges are shown on Figure 5-8 and Figure 5-15. The final design should consider the use of a more detailed flood routing analysis using a refined grid-cell size, and the latest topographic mapping and grading plans.

The West Bank is subject to direct impingement flows as a result of the unpredictable flow patterns on the active alluvial fan. The scour analysis for the determination of the scour protection limits shall consider direct impingement in addition to general scour associated with flows parallel to the embankment fill.

The location and orientation of the West Bank may also result in the deposition of sediment along the bank during certain flow events. The determination of the top of lining or embankment fill height shall consider the potential for sediment deposition and wave runup. Flows from D423 are currently shown to have 2 flow paths on the alluvial fan. Future design studies shall consider the potential for flows to concentrate into only one of the two shown flowpaths and its impact on the design of the West Bank.

The South Bank is subject to flows from the Middle Canyons and Rock Avalanche Canyon. The bank is proposed to be roughly parallel to the direction of flow and should be designed as a standard channel bank. The top of bank shall provide a minimum of 3 feet of freeboard above the 1-percent annual chance storm event flow depths or water surface profile. The final grading for the South Bank shall be reviewed with CVWD to ensure that the bank is considered an incised channel and not a levee. Scour shall be determined in accordance with Guideline K-3. Flow from D421A may have the potential to impact the South Bank further west than shown in the current analysis. The final design of the South Bank shall consider this potential migration west of flood flows and the impact on the final design. Similar to the West Bank design, flows from D423 are currently shown to have 2 flow paths on the alluvial fan. Future design studies for the South Bank shall consider the potential for flows to concentrate into only one of the two shown flowpaths.

7.4.2 Guadalupe Creek Diversion Dikes

The final design for the dikes shall include more detailed 1D and 2D hydraulic models prepared in conjunction with the design of the adjacent embankment fill and Jefferson Road bridge crossing. The models should be prepared using a refined grid-cell size and the latest topographic mapping and detailed plans.

The north and south Guadalupe Creek Diversion dikes shall be designed to meet FEMA requirements as stipulated in Title 44, Code of Federal Regulations, Chapter 1, Section 65.10 (44CFR65.10) and all current engineering manuals and engineering technical letters of the USACE related to levee design and construction that are referred to in the Federal Code. Ownership and maintenance of the levees is a CVWD responsibility. The engineer shall consult with CVWD prior to preparation of the final design and technical studies.

7.4.3 Jefferson Road, Avenue 62, and Madison Street Bridge Crossings

The Jefferson Road, Avenue 62, and Madison Street roadway extensions into the project site will require crossings of the Guadalupe Creek Diversion Dikes and Dike No. 4. The conceptual design for these crossings include the use of multiple arch bridges. The bridge configuration and sizing shall be determined during the final design and incorporated into the hydraulic models. The final design shall address freeboard and scour calculations as well as impacts to the dikes based on the updated hydraulic modeling.

The Avenue 62 and Madison Street crossings have the potential to impact the exiting CVWD recharge ponds directly adjacent to the crossings. The final design shall include a detailed analysis of the impacts to the ponds and identify any adverse impacts and mitigation measures.

7.4.4 On-Site Drainage and Storm Water Retention

The onsite drainage system is designed to capture, convey, and mitigate storm water runoff from within the site development. The system is designed to provide protection for the 1-perecent annual chance event through a combination of street flow and a storm drain pipe system. All runoff shall receive water quality treatment in accordance with the City ordinance and MS-4 permit requirements. Two onsite storm water detention basins are designed to reduce the project condition peak runoff flow rates prior to being released to Dike #4. The basins will also provide dead storage to retain and infiltrate the required

stormwater quality volume. The onsite drainage system of storm drain pipes and basins is illustrated on Figure 7-4.

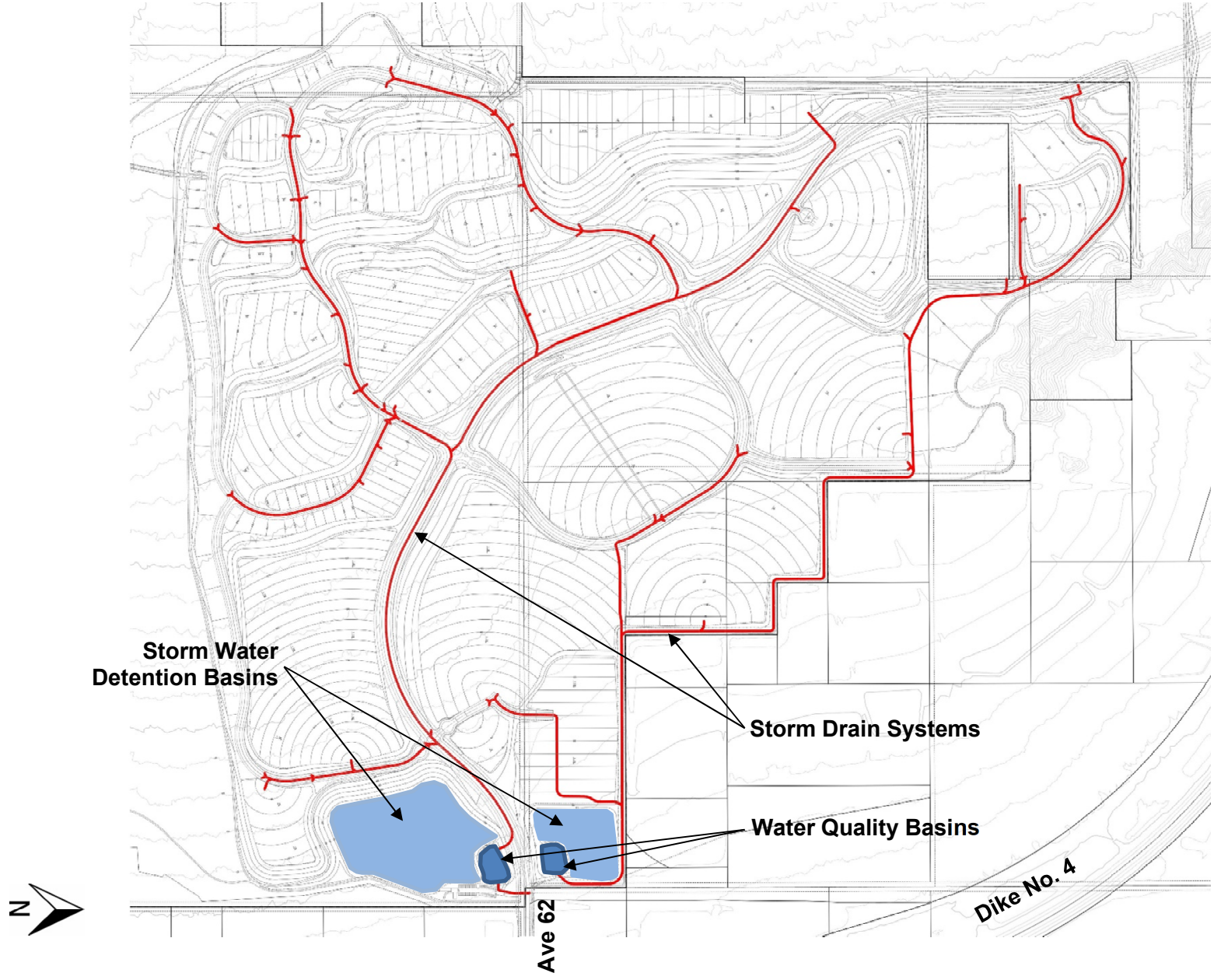
The onsite analysis is provided in the report titled, *Travertine Project, Preliminary Hydrology Study, Tentative Tract Map 37387*, prepared by Proactive Engineering Consultants, Inc., dated October 12, 2020.

7.4.5 Operations and Maintenance Plan

In general, the ownership and maintenance of the flood control facilities is proposed to be split between the development association and CVWD. The West and South Bank protection system along with the on-site drainage system and detention basins would be maintained by the development association. The improvements to the Guadalupe Creek Diversion Dikes and the existing Dike #4 would continue to be maintained by CVWD.

Sediment is anticipated to deposited along the West Bank of the project site as a result of the potential 90-degree change in flow path direction. It is anticipated that the O&M plan will include provisions to monitor and remove sediment along the West Bank to maintain the required conveyance and freeboard conditions. Other aspects of the bank maintenance shall be identified based on the final design configuration of the systems. A Flood Control Facilities Operations and Maintenance Manual for the proposed improvements shall be prepared and submitted to CVWD for review and approval. The manual shall meet the requirements of Section 5.8.9 of the Development Design Manual.

Figure 7-4. Onsite drainage plan



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