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Mr. John Svahn
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RE: Van Norden Dam Hydraulic Evaluation - CLIENT REVIEW DRAFT

Mr. Svahn:

This report summarizes the findings of a hydraulic analysis of the South Yuba River below Van Norden Dam near Soda Springs, California. The primary focus of this study is to evaluate the risk associated with a hypothetical failure of Van Norden Dam under its existing configuration and future configuration alternatives. Loss of life, property damage, and economic disruption are the primary factors driving the risk assessments, which in turn are used to recommend a hazard potential classifications following USFS FSM 7500 definitions. Ultimately this information is intended to be used for decision-making on a future dam configuration.

HYDROLOGIC SETTING

Lake Van Norden is situated just west of the Sierra Crest, and has a mean basin elevation of 7,300 ft. Elevations within the watershed range by more than 2,200 feet, and its highest points along the Sierra Crest exceed 9,000 feet (see Figure 1). Spring snowmelt typically drives annual peak flow rates, however the most extreme peaks of the last 60 years have been from early-winter rain-on-snow events. Most of the runoff arriving at Lake Van Norden comes from two locations: Upper Castle Creek drains Castle Valley (4.1 square miles) to the northeast, and the South Yuba River drains Summit Valley (5.8 square miles) to the southeast[1]. A number of intervening hillside areas also contribute runoff directly to the lake; the total drainage area at the Van Norden Dam spillway is 10.4 square miles. The mean annual precipitation of the watershed is 65.9 inches[2]. Soils are classified primarily as Hydrologic Soils Group D, with some B soils at lower elevations and within the lake footprint and surrounding meadow[3]. The majority of the basin is lightly developed with the most significant impacts to land cover from ski area infrastructure (Sugar Bowl, Donner Ski Ranch, and Boreal), the Union Pacific Railroad, and roads including Interstate 80, Old Highway 40, and numerous dirt roads. Rock outcrops are abundant along higher elevations, and are significant in terms of total impervious area.

The nearest streamflow gage on the South Yuba River (USGS Gage 11414000 near Cisco Grove) is located over 10 miles downstream of the dam and has a drainage area five times that of Lake Van Norden. For these reasons, it could not be reliably translated to the study site for a flood-frequency analysis. There is a streamflow gage on Upper Castle Creek (USGS Gage 11413900), however it only has a six-year record (1957 to 1963), and accounts for less than half of the total drainage area for the study site. A detailed hydrologic analysis and modeling effort is beyond the scope of this project, so regional regression equations for California (Gotvald et al., 2006) were used to estimate peak flow rates for various recurrence intervals (see Table 1). We recognize the inherent error in these estimates, and have made an effort to validate them against anecdotal information from area residents. The precise magnitude of a certain recurrence interval flood is not essential to this application since USFS, BOR, or other guidance does not mandate evaluating a certain flood for a dam of this size. Nevertheless, we have included the estimates from regional regression equations to help communicate the relative magnitudes of dam breach flows to a broad audience.

HYDRAULIC MODEL DEVELOPMENT

The US Army Corp of Engineers Hydrologic Engineering Center's River Analysis System (HEC-RAS) version 4.1, along with its geospatial extension for ArcGIS, HEC-GeoRAS version 4.3.93, was used to model Lake Van Norden and the South Yuba River downstream of Van Norden Dam. A digital terrain model (DTM) in the ArcInfo TIN format was developed from a survey completed for this study by Andregg Geomatics. The survey included a regional 2-foot interval photogrammetric contour map supplemented with detailed channel survey data collected with a total station. The survey data were combined with a bathymetry survey done by Balance in July 2013 to create a seamless surface of Lake Van Norden and the South Yuba River from the dam to just downstream of Interstate-80. The data were thoroughly reviewed for quality control in preparation for subsequent steps.

The limits of the hydraulic model are from the Van Norden Dam, to a point roughly 200 feet downstream of the Interstate 80 culvert (total distance along the South Yuba River is 5,100 feet). The lower limit of

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[1] Basin elevations estimated from the National Hydrologic Dataset (NHDPlus), 30 meter resolution DEM

[2] Mean annual precipitation estimated from the Parameter-Elevation Regressions on Independent Slopes Model (PRISM) climatic dataset, 800 meter resolution; includes data from 1971 to 2000

[3] Soil Survey Staff, Natural Resources Conservation Service, United States Department of Agriculture. Web Soil Survey. Available online at http://websoilsurvey.nrcs.usda.gov/. Accessed 12/22/13.

the model was set at this location because: (1) backwater effects on the upstream side of the I-80 culvert are accounted for; this is important because there are several residences in the vicinity, (2) housing density adjacent to the channel and floodplain is extremely sparse between the I-80 culvert and Kingvale, located three miles downstream, and (3) the timing and magnitude of additional inflows from Lower Castle Creek[4] would need to be accounted for; which is only reliably done with an accurate and detailed hydrologic model.

A level pool routing scheme was chosen to simulate hydraulic interactions between Lake Van Norden and the dam spillway. This option requires less input data (only a stage-storage relationship and inflow hydrograph are needed), and results in a more computationally stable model. The alternative, fully dynamic routing, represents the storage characteristics of the reservoir through a series of bathymetric cross sections, thus can simulate the propagation of the flood wave through the reservoir, as well as non-uniform water surface elevations as the reservoir drains through the breach. Though detailed bathymetric data were available for this study, the dendric nature of the lake bottom would be difficult to adequately represent with cross sections.

Cross sections were cut from the DTM in GeoRAS at intervals ranging from 50 to 200 feet, depending on the uniformity of the terrain. Since the DTM was largely based on a photogrammetric survey that does not penetrate water, the cross sections needed to be augmented to account for the channel shape below the water surface. A low flow channel was added to each cross section based on the shape of the nearest cross section surveyed with the total station. The additional area added to cross sections through this step was very small compared to the total flow area during a dam breach. Finally, the interpolate cross section tool in HEC-RAS was used to smooth transitions between cross sections, and to bring the Courant number[5] close to one. The spacing of the interpolated cross sections did not exceed 25 feet.

Manning's n values used to represent roughness within the channel banks was 0.085 for most cross sections. The channel bed is dominated by boulders, however the resolution of the topographic data is too coarse for individual boulders to appear in the cross section shape. The Manning's n of 0.085 was selected to account for the roughness arising from the turbulence created by boulder protrusions, and is based on field measurements from comparable systems in Colorado by Jarrett (1985). Cross sections near bridges where the channel bed was observed to have been cleared were assigned a Manning's roughness of 0.06. A 200-foot long segment of the study reach with a steeper-than-average slope was assigned a Manning's n of 0.12. Preliminary model runs indicated supercritical flow over this segment which is not realistic for appreciable lengths in boulder-bed channels (Dobbie and Wolf, 1953; Thompson and Campbell, 1979; as cited by Jarrett, 1985). We feel this is a valid assumption based on field observations.

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[4] Downstream of I-80 at the confluence with Lower Castle Creek, the South Fork Yuba River watershed area increases approximately two-fold.

[5] In one-dimensional hydraulic modeling, the Courant number is the ratio of the product of flood wave velocity and the time step to cross section spacing. It is a tool used to balance cross section spacing with time step to minimize oscillations in the solution which may lead to the model becoming unstable.

Manning's n values for above channel banks were 0.15 for most cross sections in the reach. The banks were set at first continuous line of vegetation encountered beyond the boulder channel bed. We acknowledge the seasonality in this estimate from the presence of leaves and/or snow cover. However, since subsequent portions of this study concern the population at risk, a value at the high end of the conceivable range was chosen to provide a conservative estimate of inundation extents. The portion of the study reach between Van Norden Dam and the Soda Springs Road bridge has a gravel parking area along the right bank. The Manning's n along this overbank segment was 0.05.

Within the study reach there are two bridge and two culvert crossings. The precise locations of the abutments, piers, soffits, and decks were entered in the model based on survey data collected by Andregg. Ineffective flow areas were added to the cross sections upstream and downstream of each crossing to simulate the expansion, contraction, and eddying of flow around embankments.

Houses were added to cross sections as obstructions based on the shape of their footprints in Andregg's photogrammetry survey. The footprint shapes of certain houses were adjusted because they were cantilevered over the hillside. Preliminary model runs were used to indicate what houses would be the first to be inundated, and these structures were revisited in the field to estimate living floor elevations. A hand level was used to relate the approximate living floor elevations to known elevations on nearby bridges or culverts.

DAM BREACH PARAMETERIZATION

Methods for characterizing dam breaches include predictive equations, comparative analysis, physical modeling, and erosion-based modeling (Wahl, 1998a; Wahl, 2010). For this application, predictive equations provided the best balance of data requirements, accuracy, and feasibility given economic and time constraints. In using such empirically derived equations it is essential to address the uncertainty in the estimates due to the amount of scatter in the case study data (Wahl, 1998a; Wahl, 2004). To do so, we first identified a range of feasible parameters based on the predictive equations. From there, we performed a sensitivity analysis to evaluate how model output (principally, peak magnitude of the flood wave) responded over the range of the most uncertain input parameters. Lastly, we used best-judgment to choose a single scenario that provides a conservative, yet realistic representation of a dam failure.

Failure Mode

Dam breaches initiate by the embankment overtopping or by piping. Embankment failures typically happen as the spillway is overtopped thereby initiating erosion on the downstream toe of the embankment which advances upstream through the remainder of the embankment (Dodge, 1988). As such, it would be unlikely for an embankment failure to occur anywhere other than at the Van Norden Dam spillway since it is the low point along the embankment. Piping failures initiate as seepage erodes a gradually widening hole through the dam embankment, which eventually breaches the dam crest and has a similar ultimate shape as an embankment failure. Outflow peaks by piping were found to be five percent less compared to embankment failures having the same ultimate breach geometry. The failure mode is highly uncertain, so an embankment failure was adopted since it provides a more conservative outflow hydrograph.

Embankment failures were idealized as trapezoidal notches gradually increasing in size. The notch is defined by its physical properties (bottom width, height, and side slopes), as well as time-related parameters. A literature review by Wahl (1998b) summarized ten different studies relating breach parameters to case study data through regression equations. The parameters describe the ultimate shape of the breach; the size of the breach during development varies as a percentage of the ultimate shape. It is worth noting that nearly all data used to develop the equations came from dams and reservoirs much larger than Van Norden. However, for lack of other guidance, these are used as the basis for parameterization herein.

Breach Side Slopes

Side slopes of the breach vary based on the method of construction and the angle of repose of the materials used to construct the dam. In the case study data side slopes ranged from 0.25 to 2.0 (horizontal : vertical), and side slopes greater than 1.0 were uncommon. A side slope of 1.0 was chosen for all breach scenarios, which is consistent with the angle of repose for large boulders (Julien, 2010). We anticipate the side slopes to be controlled by large clasts from the gradual erosion of the concrete spillway. A side slope steeper than 1.0 would yield a less conservative (i.e. smaller magnitude) peak outflow since the cross sectional area would be smaller.

Breach Depth

The is no guidance for selecting a breach depth because it is common to assume that the dam erodes vertically to its base. However, we do not believe it is likely that Van Norden Dam would erode its maximum height as measured from the downstream toe based on the bathymetry just upstream of the dam. It is improbable that the dam would erode to an elevation less than 6,748.6 feet (NGVD29) because the depressional areas below that elevation are small and setback from the spillway. There is not enough storage in these areas to generate the tractive forces for erosion. For this reason, we have set the height of the breach at 6 feet (bottom elevation of 6,748.6 feet).

Breach Formation Time

The only time-related parameter meaningful to this analysis is the time of failure (length of time it takes for the breach to fully develop). We are not routing an inflow hydrograph to Lake Van Norden so the water surface elevation is constant during the time leading up to the dam breach. Case study data agree that time of failure is typically between 0.25 and 1.0 hours, and that it rarely exceeds 1.0 hours. Balance Hydrologics and USFS managers agreed that a Van Norden Dam failure would happen slowly due to the low head in the reservoir and the large volume of material in the spillway that would need to erode (personal communication, Stephen Romero, August 12, 2013) so the time of failure was set at the upper bound of 1.0 hours. The progression of the breach was modeled as a sine wave. This is not a significant deviation from the linear method, and was chosen because it abets numerical stability in the model. The model was not found to be sensitive to a linear versus sine wave progression; everything else held equal, peak outflow varied by three percent at most.

Breach Bottom Width

Thus far, the parameters selected for defining the breach began as a range of possible values based on case study data, and were narrowed to a single value based on scientific reasoning and intuition from familiarity of the study site. It is more difficult, however, to imagine the ultimate width of the breach. Most case studies in the literature review by Wahl (1998b) used a linear relation to correlate the final bottom width of the breach to the height of the dam. Coefficients ranged from 0.5 to 5 with a mode of 3. No specific guidance on where to the measure the height of the dam is provided. Even by using USFS definitions for dam height (USFS, 2011) ambiguities remain given the eroded condition of the dam toe and lack of detailed topographic data just downstream of the dam. Depending on where elevations are measured on the spillway, adjacent ground, and scour pool at the downstream toe of the spillway, the height of Van Norden Dam is between 6 and 16 feet. From the regression equations, a minimum estimate for the final bottom width of the breach is 3 feet and a maximum is 80 feet. This range varies by more than an order of magnitude, and underscores the importance of sensitivity analyses and use of best judgment.

A series of sunny day dam breach models were run with varying bottom widths and holding all other breach parameters constant. The peak outflow from the breach ranged linearly from 290 cfs for a bottom width of three feet to 2,240 cfs for a bottom width of 80 feet. We feel a breach at either extreme of that range is improbable. Just after full formation the velocity of flow through a three-foot bottom width breach is over five feet per second; a cursory incipient motion calculation indicated the tractive forces at this point are enough to mobilize a clast over 3 feet in diameter (assuming a Shields parameter of 0.06). For this reason we think it is likely that a breach of this size would continue to develop. At the opposite end of the range, a breach having a bottom width of 80 feet would mean more than the entire width of the spillway eroding. The top of the spillway is approximately 100 feet long, and the side slopes on either end are over 20 feet long each. Given the amount of storage in Lake Van Norden it is improbable that a flow capable of mobilizing large concrete clasts would persist for enough time to erode an 80-foot wide notch through over 100 feet of concrete.

A bottom width of 20 feet was selected for all failure scenarios. The outflow from a breach of this size is near the middle of the range identified above (880 cfs), and we feel is a conservative, yet realistic representation of a potential failure at Van Norden Dam.

Model Validation

A number of regression equations have been developed from the dam breach case study data that relate peak discharge to dam height, head above spillway, storage volume, volume above spillway, or some combination thereof. The hydrology methods used in this study limit which relationships may be used since routing a steady hydrograph through Lake Van Norden—while a conservative practice—does not provide a reliable estimate of the changing head and volume above the spillway during the breach event. Similar to the breach parameter predictors, there is significant scatter in the peak outflow data. Nevertheless, insight can be gained from certain relationships to at least gauge whether the breach simulation is reasonable. Plots in Figures 13, 14, and 15 from Wahl (1998b) show peak outflows vary by up to two orders of magnitude, but suggest that the model results are within reason.

RESULTS

The following sections present the results from routing the outflow hydrograph from the dam breach through the South Yuba River under three scenarios: a sunny day breach under existing conditions, a breach concurrent with the design flood under existing conditions, and a breach concurrent with the design flood under the proposed notched configuration. Additionally, the methods for estimating the largest flood that poses no measureable risk to life and property with no dam breach are presented. This flood is herein referred to as the design flood. The magnitude of this event is central to the foregoing discussion because it sets the target flood stage against which to compare the magnitudes of events that could happen concurrent with a dam breach and still pose no threat to life and property.

Sunny Day Breach

The sunny day breach is defined as a dam failure scenario with no concurrent inflow flood to the reservoir. The reservoir stage was set at the invert elevation of the spillway, and the dam was breached per the parameters described in the previous section. The peak of the outflow hydrograph was 880 cfs; a natural event of this magnitude has a return period of roughly 8 years. It is common in flood routing for the peak flow to decrease in the downstream direction as additional floodplain storage becomes available; however, there was very little variation in the magnitude of the hydrograph peak over the study reach because the channel is steep, dominated by boulders and glacial till, with limited room for lateral adjustments and floodplain development. Aside from the Soda Springs Road bridge, the other three crossings in the reach can convey the flood pulse from the sunny day breach with minimal backwater effects.

Figure 2 shows the inundation extents for the maximum stage during the sunny day breach. No structures are affected at the maximum stage for this event, nor is Soda Springs Road overtopped.

Design Flood

The design flood was estimated at 3,200 cfs and has a recurrence interval of roughly 120 years based on regional regression equations for California (Gotvald et al., 2012). This figure was arrived at through an iterative process whereby a series of steady state hydraulic models were run until the flood stage reached a point where lives or property was threatened. The study site presented a challenge in identifying a threshold at which lives are threatened because many houses along the South Yuba River are situated on steep hillsides with portions of the structures cantilevered over the floodway. The hydraulic model indicated that the lowest portion of one structure is inundated by as little as 1,200 cfs (approximately the 10-year event). However, the bottom several feet of all structures along the study reach are either crawl spaces or storage areas. Inundating uninhabited portions of homes was considered inconsequential in the risk analysis. For this reason, the threshold stage at which lives are endangered was set at the lowest living space elevation.

Figure 3 shows the inundation extents for the maximum stage during the design flood. A flood of 3,200 cfs has a modeled stage of 6,680.6 feet at the first home upstream of the Donner Pass Road culvert where the living floor elevation was estimated at 6680.7 feet. The only other structure inundated by a flood of this magnitude is the second home upstream of the Donner Pass Road culvert where the bottom 3.5 feet of the foundation—but not the living space—is wetted.

The Soda Springs Road bridge is overtopped at the design flood which causes a temporary economic disruption for the Serene Lakes community. A local resident confirmed that the bridge has overtopped four or five times in the last 50 years, and that there has been water three feet deep above the bridge deck following the largest events (Norman Sayler, personal communication, December 20, 2013). The bridge has not suffered damage in this time frame, so as soon as flood levels recede, the road is passable. Nevertheless, conveyance under the bridge is significantly less than the other three crossings within the study reach, and would be a good candidate for replacement regardless of the future configuration of Van Norden Dam.

Dam Breach Concurrent with Design Flood

The peak of the outflow hydrograph for a dam breach concurrent with the design flood under the existing dam configuration is modeled to be 4,570 cfs; the maximum inundation extents for this scenario are shown in Figure 4. The incremental risk for this scenario (compared to the design flood with no dam breach) is the two homes mentioned in the design flood section have their living spaces inundated by several feet of water thereby threatening the lives of the residents. The level of inundation may be enough to threaten the structures themselves, however, modeled velocities in these locations are fairly low (less than three feet per second) making destruction of the homes by moving water unlikely. Flooding in these locations is directly related to backwater created by the Donner Pass Road culvert; backwater extended as far as 700 feet upstream during the simulation. The foundation of an additional home in the vicinity of the Soda Springs Road bridge is inundated in this scenario by less than one foot of water, but there are several feet of freeboard between the water surface and the living space.

Loss of life was estimated for this scenario following the Bureau of Reclamation DSO-99-06 procedure. The threat posed to lives in this scenario represents the risk the Forest Service would assume if no changes are made to the dam configuration. The loss of life estimate is presented using the 11-steps of the DSO-99-06 procedure (USDHS, 2011) as a framework.

Selection of the dam failure scenario (Step 1) has already been discussed in detail in previous sections. Time categories were selected (Step 2) to depict a worst-case scenario where there are as many people as would be likely in the inundation extents. The population of Soda Springs is extremely seasonal (only 30% of homes are occupied full time), and is at a maximum during holidays and weekends. Time categories used for the loss of life estimate were: a weekend, during the Summer or Winter (high tourist season), and at night (when people would be at home). The area flooded has been identified (Step 3) in Figure 3. The number of people at risk (PAR; Step 4) was estimated through aerial photographs and US 2010 Census data. The total population of Soda Springs was 81 in 2010, and there were 136 households. Even if only occupied homes are considered (41 of the 136 households), the number of people per household falls below the national average of 2.6. We feel it is reasonable to assume there would be more people per household during a holiday weekend, so the average number of people per home was set at four. As mentioned earlier, there are two homes threatened by floodwaters from the selected dam failure scenario; therefore, the PAR is 8 people. Though it may not be reasonable to assume someone would be present to observe a breach initiating and issue a warning (Step 5), it is likely that local residents would be alerted by the threat from rising flood waters alone. That is, a design storm-magnitude event is rare for the region, and would be big news. If a warning of a dam failure were issued, it would not be meaningful because of the proximity of the PAR to the dam (Step 6), the lack of an emergency action plan, and a varied understanding of the flood severity by the local population (Step 7). Flood severity (Step 8) was defined as "low" because of the rise of the flood wave is anticipated to be slow from the slow formation time, and because no buildings are expected to be washed from their footings. The suggested fatality rate (Step 9) for a low-severity flood with ample (60 minute minimum) warning time is 0.0003, therefore the estimated loss of life (Step 10) is 0.0024 persons. This could be interpreted as, in the event of a breach concurrent with the design flood during a holiday weekend, there is an 0.24 percent chance that a fatality would occur. Aside from the uncertainty in selection of breach parameters and the routing of the flood wave, additional uncertainty in the loss of life estimate (Step 11) comes from our estimate of the PAR. Clearly, it is possible for more than eight people to be within the inundation zone, however, for a high degree of certainty in a breach event resulting in a fatality (i.e. loss of life equal to one), the PAR would need to be an order of magnitude greater. This is equivalent to the entire population of Soda Springs.

In conclusion, the estimated loss of life for a dam breach concurrent with the design flood is 0.0024 lives meaning a fatality is possible, though not probable. This figure was arrived at by estimating the number of people within the inundation extents for a dam breach concurrent with the design flood (the PAR), and multiplying by the fatality rate as determined by USBOR guidance. In evaluating risk, it is appropriate to consider the probability of events applied to the loss of life analysis. One such metric used by the USBOR is annualized loss of life:

$$Annualized\ loss\ of\ life = Estimated\ loss\ of\ life \times Annualized\ failure\ probability$$

$$Annualized\ failure\ probability = P_{design\ storm} \times P_{failure\ during\ design\ storm}$$

The USBOR uses a guideline of 0.001 fatalities per year to trigger the need to better understand or reduce the risk of a dam breach (USBOR, 2011). No guidance for estimating the probability of a failure during a design storm was available, so the annualized failure probability is presented as a range. The probability of the design storm in a given year is 0.0083 (inverse of the recurrence interval), and the probability of a failure during the design storm may lie between 0.01 and 1.0 (1 to 100 percent). The annualized failure probability would be between 0.000083 and 0.0083, and the annualized loss of life between 0.0000002 and 0.00002. This is well below the USBOR threshold since in a given year, a fatality from a dam breach is highly improbable. These figures do not take into account the probability of the design flood and dam breach occurring on a weekend versus weekday, holiday versus regular time of year, or during the night versus day. The PAR is sensitive to these factors, and would decrease (along with the annualized loss of life) with less conservative (i.e. non-holiday, non-weekend) assumptions.

The low probability for loss of life was derived for a hypothetical scenario where a dam failure occurs concurrent with an extreme flood event at a time when there would be an above average number of people in the inundation area. Historically, very few lives have been lost from failures of small dams. A summary of case study data by Graham (1999) showed on over 20 dam failures from 1960 to 1998 where there was loss of life, only two percent of the lives lost were from dams less than 20 feet high.

FUTURE DAM CONFIGURATION

The following section presents a series of possible alternatives for future configurations of the Van Norden Dam spillway. Each configuration is discussed in terms of risk to the downstream community along the South Yuba River according to the hazard potential classification, impacts to Lake Van Norden, and flood control potential. Definitions for hazard potential classifications were adopted from USFS (2011) guidelines. It is important to note that this study does not intend to quantify flood control benefits under the various configurations, as this would require a detailed hydrologic analysis of the Lake Van Norden watershed. Rather, alternatives are qualified as either having the potential for flood control or not.

None of the alternative dam configurations alleviate the flooding between Van Norden Dam and Soda Springs Road as seen in Figures 3 and 4. The flooding here is a consequence of the limited hydraulic capacity of the bridge. Furthermore, field evidence suggests that the channel may have been realigned upstream of Soda Springs Road, and that a portion of the former floodway was filled to build the parking area along the right bank. If true, it is likely that this channel reconfiguration also contributes to flooding problems in the vicinity.

No Action

The no action alternative is leaving the dam in its current configuration, but fortifying the existing spillway with like-for-like repairs. The risk for loss of life (0.002 to 0.00002 percent chance per year) would be the same as discussed under the dam breach concurrent with the design flood section. The hazard potential classification for this scenario is significant. The loss of life is not probable, and the potential for economic loss exists from both damaging homes and temporarily disrupting the Soda Springs Road crossing. No changes would occur to the current lake footprint or the flood control characteristics.

Reduce Van Norden Lake to 50 acre-feet

Lowering the spillway elevation to a level at which the dead storage in Lake Van Norden is less than 50 acre-feet has been identified by the Truckee Donner Land Trust and stakeholders as a viable alternative because (1) it would result in a non-jurisdictional designation for the dam, and (2) it is a compromise between having a lake and removing the dam entirely. Based on the bathymetric survey, the invert elevation of the spillway would need to be lowered by 2.3 feet to an elevation of 6,752.3 feet. The footprint of Lake Van Norden under this configuration is shown in Figure 5. A notched configuration for the spillway has been proposed where the spillway would function as a two-stage weir. The lower stage would have an invert of 6,752.3 feet thereby bringing the storage below 50 ac-ft, and the upper stage would have the same invert elevation of the existing spillway.

A series of hydraulic models were run with trapezoidal notches in the spillway of varying widths. The dam was breached concurrent with the design flood for each configuration. The breach parameters were the same as under the existing conditions model, with the exception of the breach width. The breach width was reduced from 20 to 15 feet to reflect the reduced height of the dam. Peak flows were lower for all notch widths compared to existing conditions, and decreased with increasing notch width. Maximum flood stages at the homes just upstream from the Donner Pass Road culvert were also lower, but not enough to avert the risk to loss of life or property damage. These patterns are a result of the steady state inflow hydrograph used in the analysis. Intuitively, the opposite effects might be expected, however the results are not unreasonable for a prolonged duration of the peak flow. Even if the entire width of the spillway were widened (e.g. would no longer be a two-stage weir), the first home upstream from Donner Pass Road would be inundated by one foot of water. The threat to loss of life may be less under this configuration because the maximum flood stage is over two feet lower than a breach under the existing configuration, however, property damage is still probable. For this reason, the hazard potential classification remains as significant for a non-jurisdictional spillway configuration with a 2.3 foot deep notch of any width, as it is for the dam in its current configuration.

Flood control benefits would still exist for a spillway with an invert of 6,752.3 feet. A narrower notch intrinsically provides more flow metering, but has the potential to result in a more severe outflow hydrograph in the event of a dam breach. Lowering the entire spillway width by 2.3 feet with no notch may provide some attenuation, but the spillway is sufficiently wide that the inflow and outflow hydrographs for Lake Van Norden would be very similar. Flood control in this configuration would be similar to that in the existing configuration, in which flood control benefit is largely a function of the antecedent lake level.

Low Hazard Potential Configuration

For Van Norden Dam to have a low hazard potential classification the spillway invert would need to be lowered below 6,752.3 feet which would result in a smaller lake size. We recognize the ecological value of the lake in its existing condition, and used an iterative process to maximize the lake size while achieving a low hazard rating. The objective was to identify a configuration in which the peak flow and flood depths resulting from a failure versus non-failure are essentially indistinguishable when estimating loss of life and within the level of error of the model.

The threshold where no loss of life would occur was defined as the stage at which no living spaces are inundated. To achieve this condition during a dam breach event, the entire width of the spillway invert would need to be lowered by 3.3 feet to an elevation of 6,749.6 feet. Under this configuration, the dead storage in Lake Van Norden would be 5 ac-ft, and the lake footprint is shown in Figure 6. The peak discharge from a dam breach concurrent with the design flood would be 3,250 cfs, and the flood stage at the first home upstream of Donner Pass Road would be 6,680.8 feet. (The living floor elevation at the same structure was measured to be 6,680.7 feet.) We believe the additional 0.1 feet of flooding to be indistinguishable to loss of life and property, and within the level of error for the hydraulic model, especially since no data were available to calibrate the model. The height of the dam is lower in this configuration, and the ultimate breach width was adjusted accordingly to 10 feet. All other breach parameters remained unchanged. The possibility for property damage under this scenario is the same as for the design flood. A low hazard potential classification is recommended for this configuration; loss of life is highly improbable, and property damage would be minor, if any.

Flood control benefit could be gained from this configuration by lowering the entire spillway to an elevation of 6,750.1 feet, and having a narrow notch with an invert of 6,749.6 feet. The model was minimally sensitive to small additions of cross sectional area along the spillway. The incremental storage in Lake Van Norden between 6,749.6 and 6,750.1 feet is roughly 4 ac-ft. The storage gained from a narrow, but shallow notch is minimal from a flood control perspective, and the incremental risk in the event of debris blockage would be low.

Remove Dam

The dam configuration identified for a low hazard rating may not be an attractive alternative to stakeholders because of the size of the remaining lake footprint may not be enough to rationalize keeping a low-height dam with unknown flood control benefits. Removing the dam entirely would be an alternate means of mitigating the risks to loss of life and property damage along the South Yuba River corridor because the incremental risk associated with a dam failure would not be an issue. The hydraulics for this scenario would essentially be the same as under the steady state results discussed in the design flood section. Base flows could be increased in the South Yuba River, and potential flood control benefits would be that of a typical subalpine Sierra meadow. If this alternative is pursued, dam removal planning be done in concert with a detailed hydrologic analysis to quantify the potential impacts on downstream flooding.

CLOSING

The preceding analysis presented the results from hydraulic modeling of Van Norden Dam and the South Yuba River under existing conditions and several alternatives for future configurations of the dam. The results were related to risk to life and property, hazard potential classification, flood control potential, and the impacts to Lake Van Norden's footprint. Table 2 provides a summary of key hydraulic parameters related to these criteria. The alternatives presented herein only represent four discrete conditions where some threshold criteria was crossed; a sliding scale exists between each of the alternatives.

We appreciate the opportunity to contribute to the formulation of this exciting project. Do not hesitate to contact Balance Hydrologics staff if you have questions or comments on our foregoing modeling efforts or strategies for future work described here.

Sincerely,

BALANCE HYDROLOGICS, Inc.

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Enclosures: Figures 1-6
Tables 1-2

References:

Dodge, R.A., 1988, Overtopping flow on low embankment dams, summary report of model tests: U. S. Bureau of Reclamation technical report REC-ERC-88-3, 28 p.

Gotvald, A.J., Barth, N.A., Veilleux A.G., and Parrett, C., 2012, Methods for determining magnitude and frequency of floods in California, based on data through water year 2006: U.S. Geological Survey Scientific Investigations Report 2012–5113, 38 p.

Graham, W.J., 1999, A procedure for estimating loss of life caused by dam failure: U.S. Bureau of Reclamation Dam Safety Office Report DSO-99-06, 43 p.

Jarrett, R.D., 1985, Determination of roughness coefficients for streams in Colorado: U.S. Geologic Survey Water Resources Investigations Report 85-2004, 60 p.

Julien, P.Y., 2010, Erosion and sedimentation, second edition: United Kingdom, Cambridge University Press, 371 p.

U.S. Bureau of Reclamation, 2011, Dam safety public protection guidelines, a risk framework to support dam safety decision-making: U.S. Bureau of Reclamation interim report, 31 p.

U.S. Department of Homeland Security, 2011, Estimating loss of life for dam failure scenarios: Dams Sector, 86 pp.

U.S. Census Bureau, 2010, available at www.census.gov, accessed December 13, 2013.

U.S. Forest Service, 2011, Water storage and transmission: Forest Service Manual 7500, Amendment No. 7500-2011-1, 18 pp.

Wahl, T.L., 1998a, Predicting embankment dam breach parameters, a needs assessment: Proceedings of the XXVIIth IAHA Congress, San Francisco, California, August 10-15, 1997, 7 p.

Wahl, T.L., 1998b, Predicting embankment dam breach parameters, a literature review and needs assessment: U.S. Bureau of Reclamation, Dam Safety Office, 72 p.

Wahl, T.L., 2004, Uncertainty of prediction of embankment dam breach parameters: Journal of Hydraulic Engineering, v. 130, no. 5, p. 389-397.

Wahl, T.L., 2010, Dam breach modeling, an overview of analysis methods: Proceedings of the Joint Federal Interagency Conference on sedimentation and hydrologic modeling, Las Vegas, Nevada, June 27-July 1, 2010, 7 p.

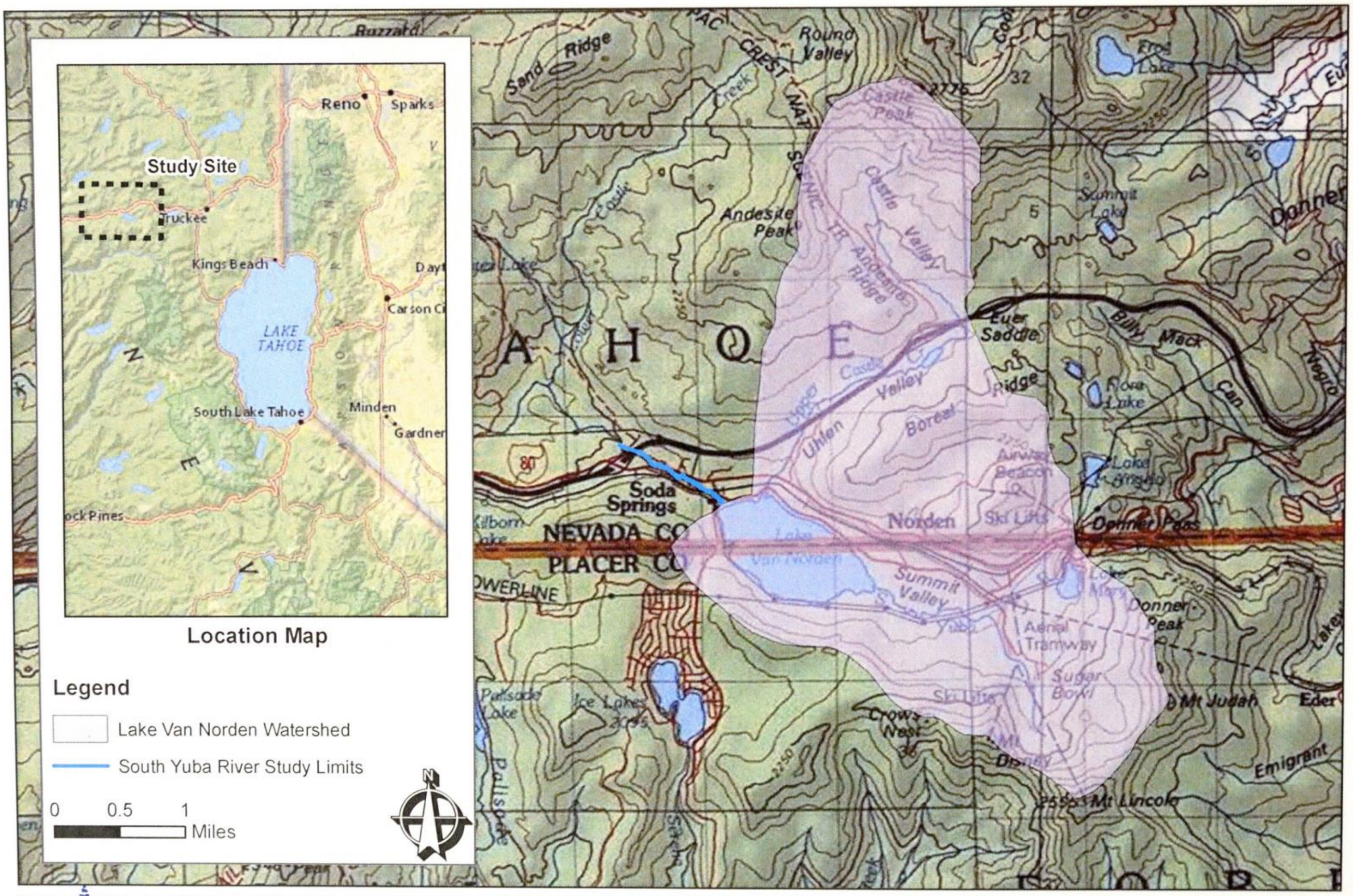

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Figure 1. Lake Van Norden Location Map and Watershed
Soda Springs, Placer and Nevada Counties, California

Data Source: USGS, National Geographic

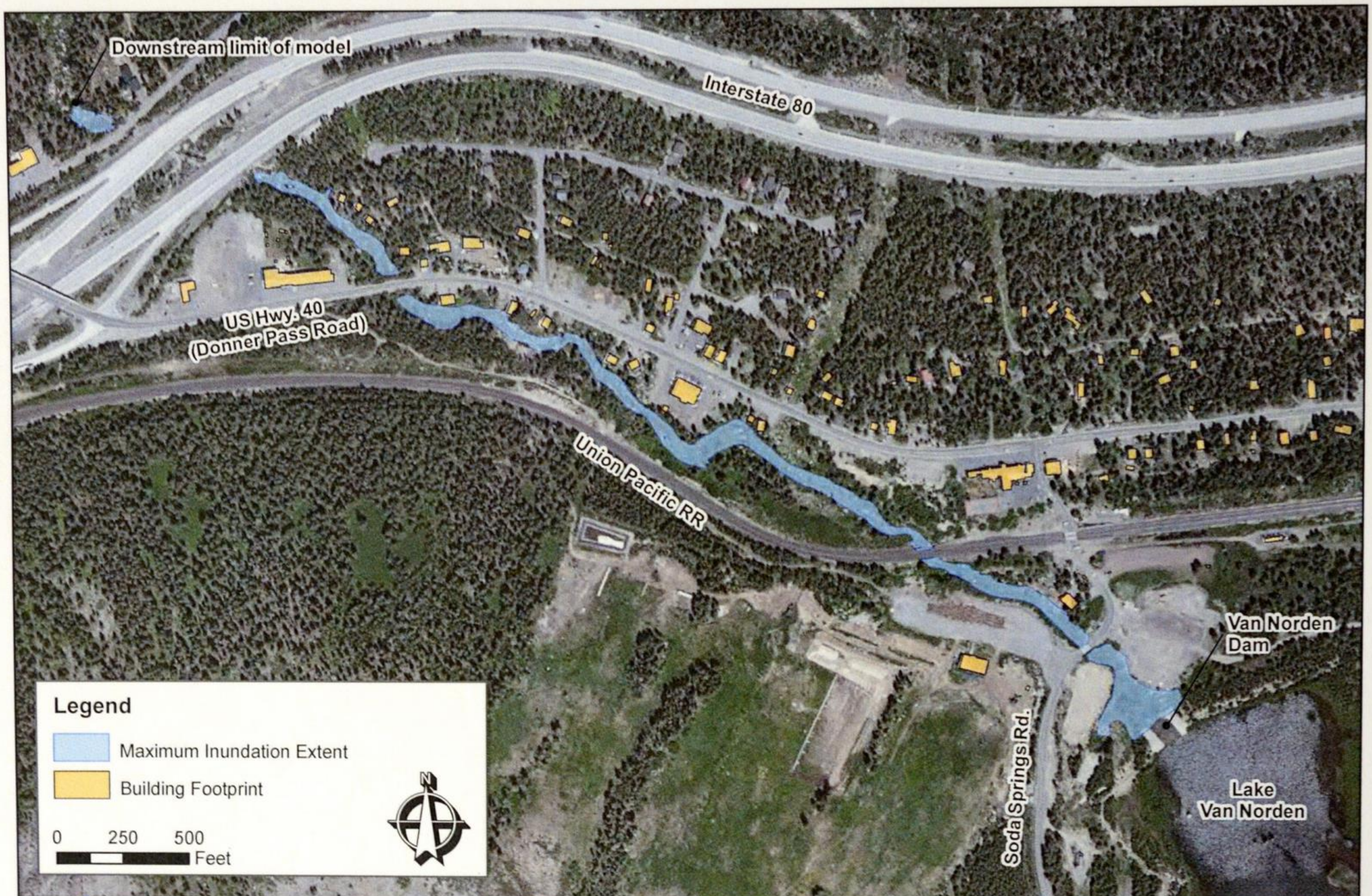

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Figure 2. Maximum Inundation Extents for a Sunny Day Breach of Van Norden Dam, Peak Flow = 880 cfs
South Yuba River, Placer and Nevada Counties, California

Aerial Photo Source: ESRI

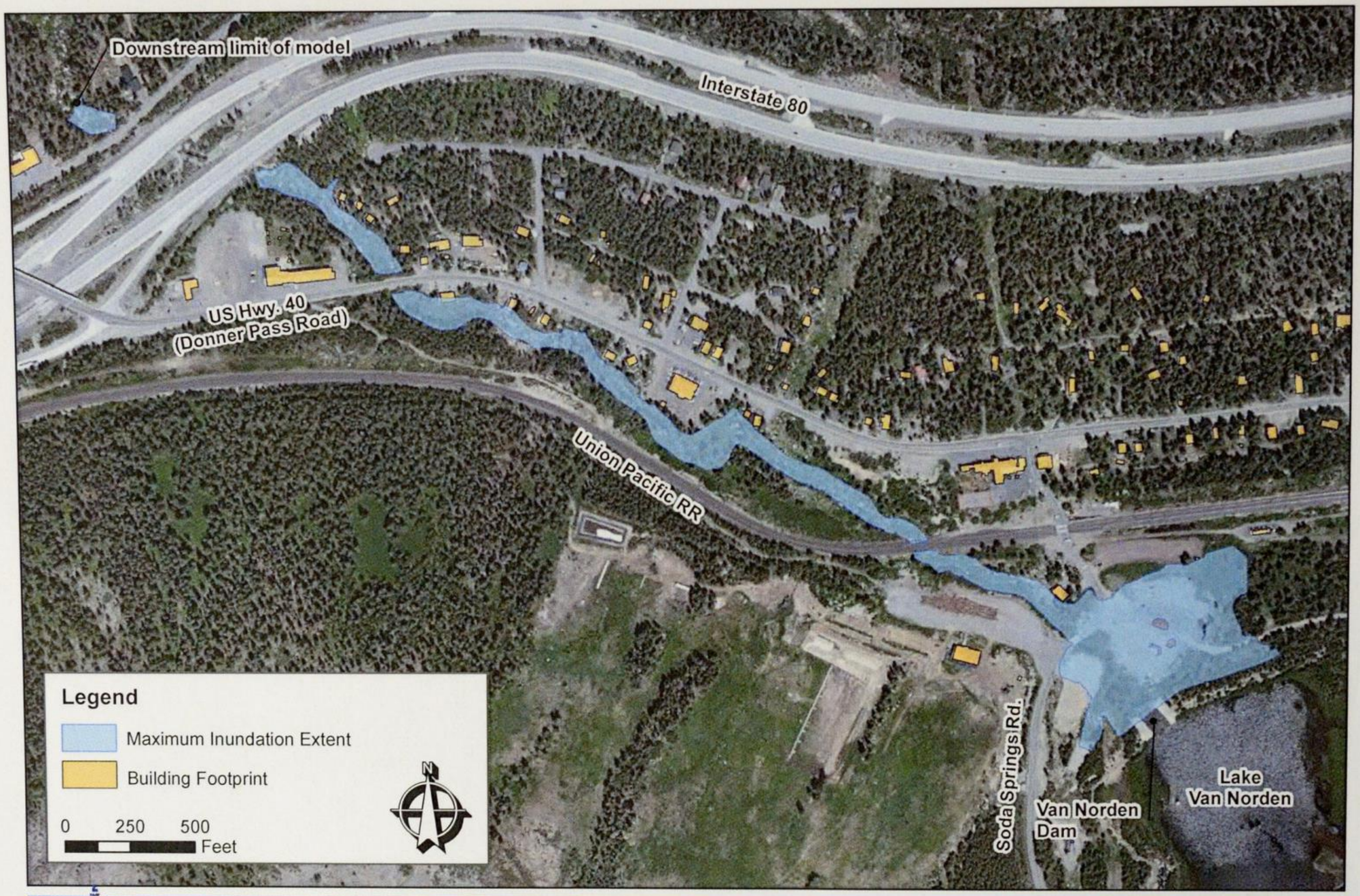

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Figure 3. Maximum Inundation Extents for the Design Flood (3,200 cfs)
South Yuba River, Placer and Nevada Counties, California

Aerial Photo Source: ESRI

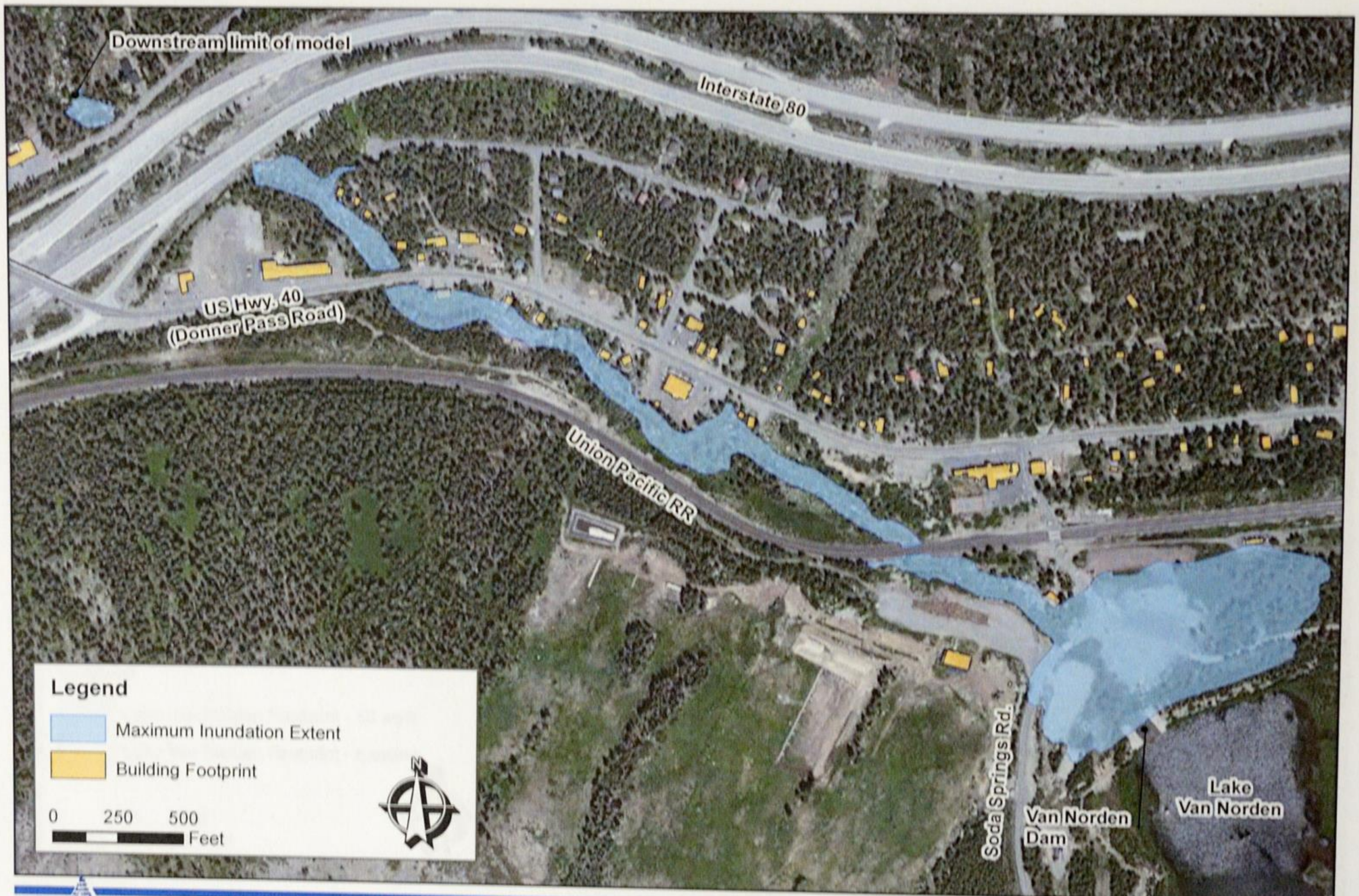

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Figure 4. Maximum Inundation Extents for a Dam Breach of Van Norden Dam Concurrent with the Design Flood, Peak Flow = 4,570 cfs South Yuba River, Placer and Nevada Counties, California

Aerial Photo Source: ESRI

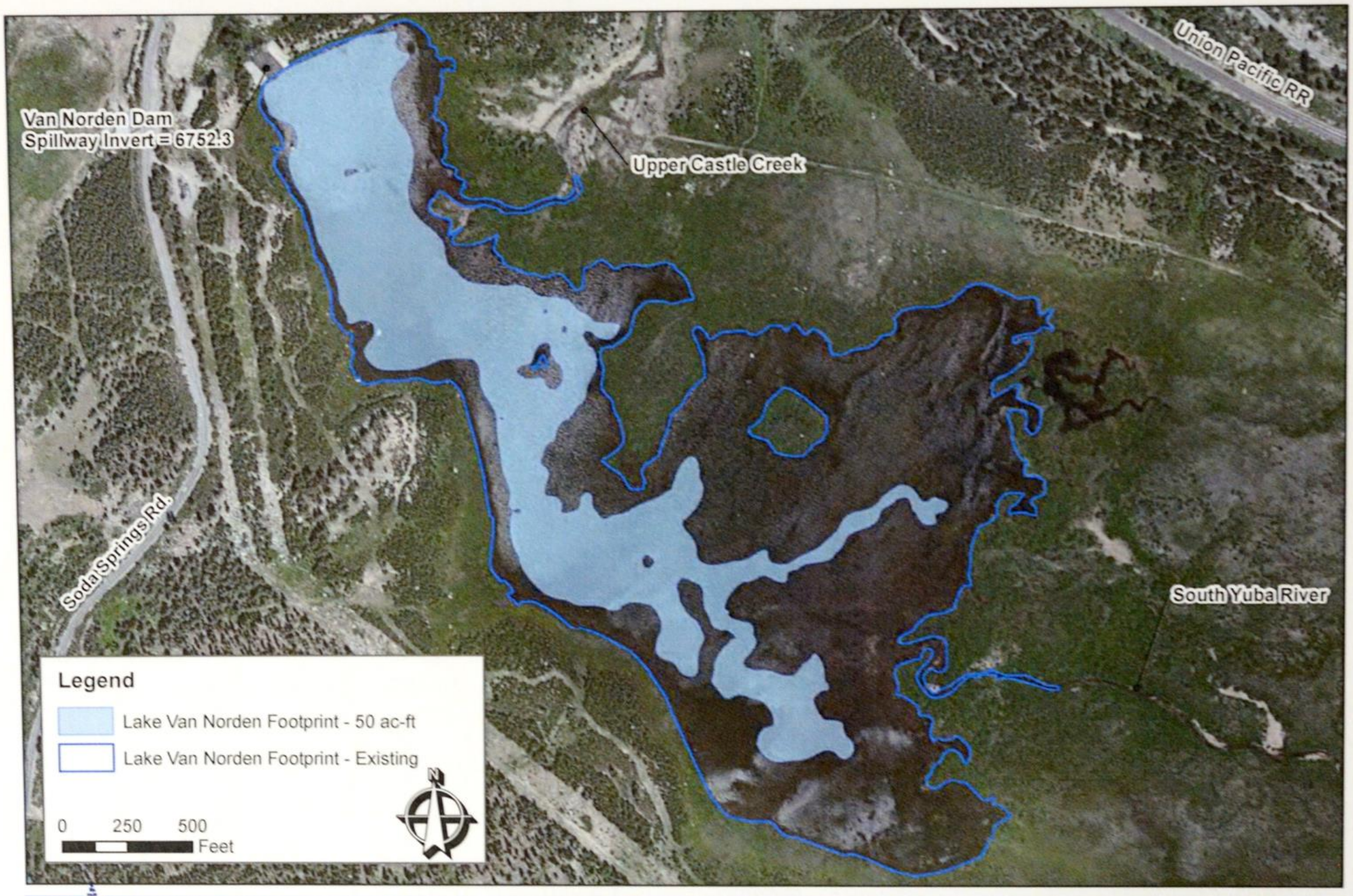

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Figure 5. Lake Van Norden Footprint at 50 ac-ft of Storage
Soda Springs, Placer and Nevada Counties, California

Aerial Photo Source: ESRI

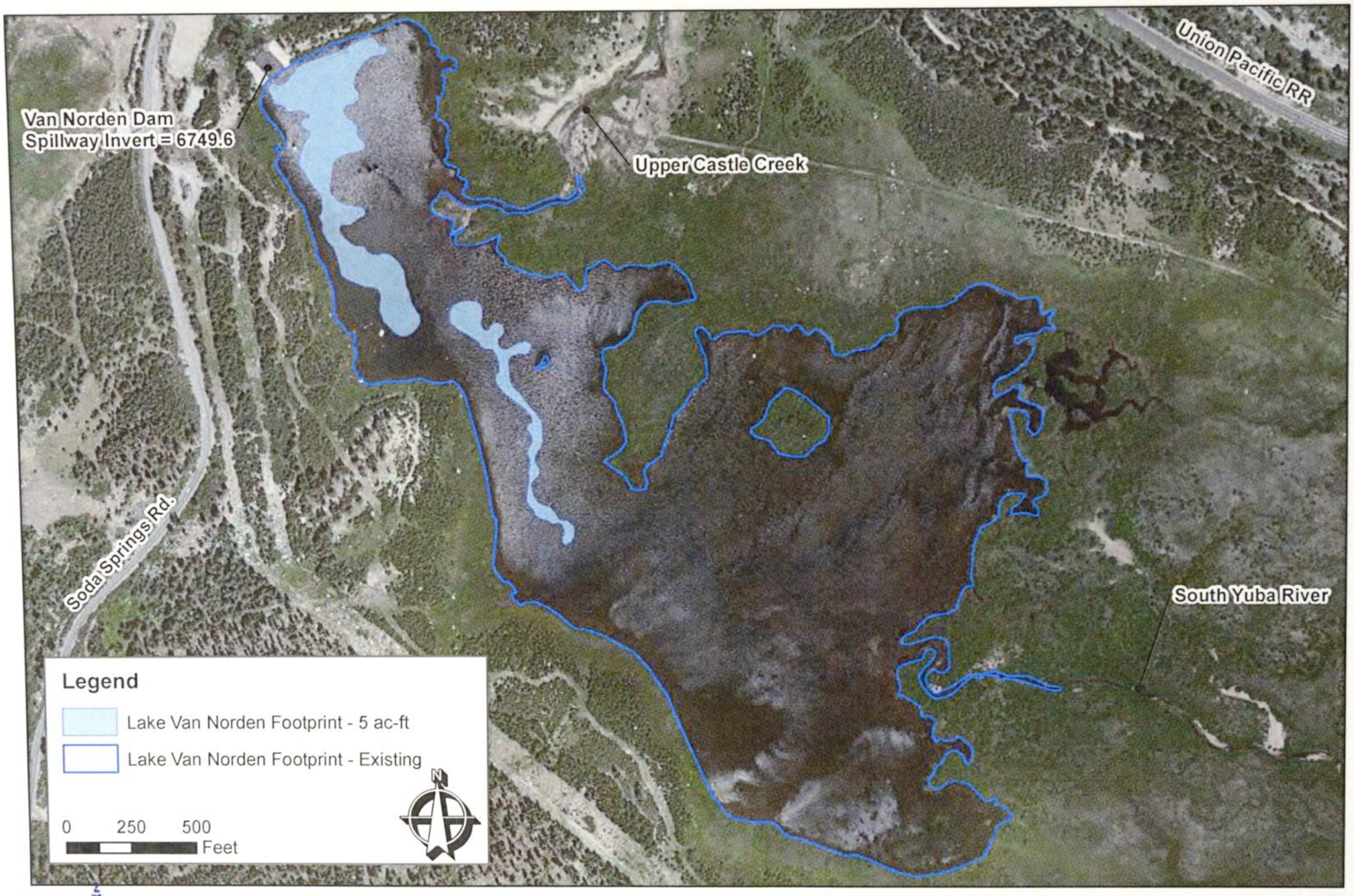

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Figure 6. Lake Van Norden Footprint at Low Hazard Rating Configuration, 5 ac-ft of Storage
Soda Springs, Placer and Nevada Counties, California

Aerial Photo Source: ESRI

Table 1. Selected Flood Frequencies, Lake Van Norden Watershed Placer and Nevada Counties, California

Percent Chance Exceedance	Recurrence Interval	Estimated Peak Discharge		
		Van Norden Dam	South Yuba River above Lake Van Norden	Upper Castle Creek above Lake Van Norden
(percent)	*(years)*	*(cfs)*	*(cfs)*	*(cfs)*
50	2	378	215	164
20	5	783	451	344
10	10	1,179	685	522
4	25	1,794	1,051	803
2	50	2,404	1,414	1,081
1	100	3,111	1,835	1,406
0.5	200	3,918	2,317	1,778
0.2	500	5,182	3,074	2,365

Note:

1. Peak discharge estimates based on regional regression equations for the Sierra Nevada region of California (Gotvald et al., 2006)

Table 2. Summary of Future Van Norden Dam Configuration Alternatives
Placer and Nevada Counties, California

Future Dam Configuration	Peak Discharge for Dam Breach Concurrent with Design Flood[1]	Maximum Flood Stage at first home upstream of Donner Pass Road[2]	Loss of Life	Flood Control Potential
	(cfs)	*(ft; NVGD 29)*	*(# of people)*	
No Action	4,570	6,684.6	0.0024	Low, depends of antecedent lake level
Reduce Lake Van Norden to 50 ac-ft	4,200[3] - 3,980[4]	6,683.8[3] - 6,683.0[4]	0.0024	Low to moderate, depends of width of notch
Low Hazard Potential Configuration	3,250	6,680.8	0	Marginal, depends of width of notch
Remove Dam	3,200	6,680.6	0	Limited to meadow floodplain attenuation

Notes:

1. The design flood has a magnitude of 3,200 cfs
2. The living floor elevation is 6,680.7.
3. Result for a trapezoid notch with a 10 foot bottom width and 1:1 side slopes.
4. Result for lowering the entire spillway width.