FINAL REPORT:
Simulation of Kalihiwai Reservoir Dam-Break Flooding

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October 17, 2006

Qualifications

FlowSimulation, LLC is a single-member limited liability company of California operated by Brett F. Sanders, Ph.D. Flow Simulation, LLC offers highly specialized expertise related to computer modeling of water motion in rivers, harbors and estuaries including studies of flooding, erosion, circulation and water quality. Clients have included major ports, city and county governments, judicial offices, film production companies and other private entities.

Dr. Sanders is internationally renowned expert in water flow modeling and the author or co-author of over 30 archival journal publications on water research topics. He is a full-time faculty member of the Department of Civil and Environmental Engineering at the University of California, Irvine where he is appointed Associate Professor, teaches courses in hydraulics and numerical methods, and leads a research group focused on water modeling topics. Full-time faculty members of the University of California are permitted to consult up to one day per week, and FlowSimulation, LLC was founded by Dr. Sanders to facilitate such activity.

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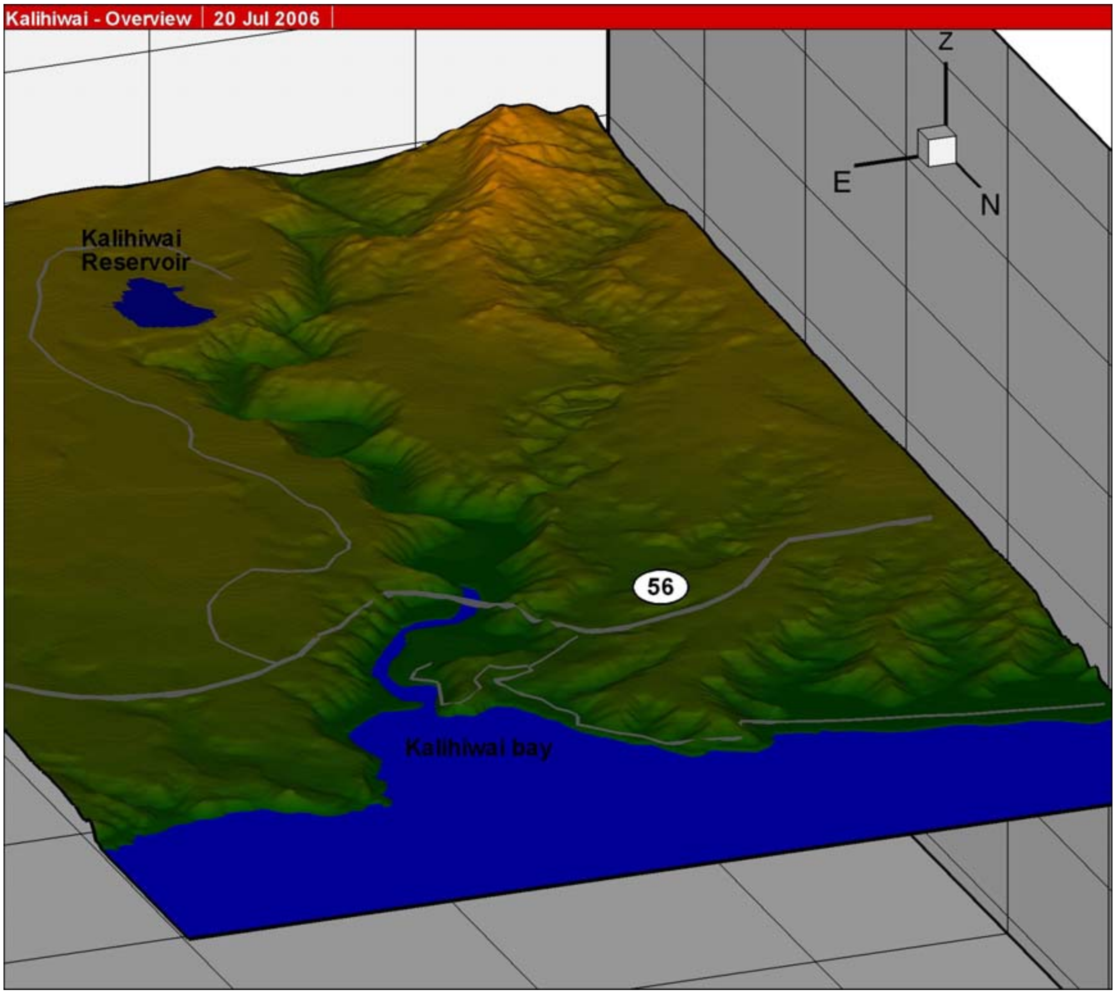


Figure 1: Kalihiwai Reservoir on the island of Kauai in the State of Hawaii.

Introduction

A computer modeling study was carried out to estimate flooding that would likely result following catastrophic failure of Kalihiwai Dam on the Hawaiian Island of Kauai. As shown in Fig. 1, Kalihiwai Reservoir sits on a ridge above a deep gorge (Kalihiwai Valley) that is drained by Kalihiwai River and terminates at Kalihiway Bay. Kalihiwai Lagoon is a prominent feature in the lower reaches of Kalihiwai Valley. From Kalihiwai Bay, the lagoon extends inland roughly 1.5 km. The lagoon passes under the Highway 56 Bridge (Big Bridge) and into a relatively flat, low-lying area known locally as the Taro Patch.

For this study, no attempt was made to evaluate the integrity of the Kalihiwai Dam and/or examine the likelihood of dam failure. All questions regarding dam safety should be directed to the State of Hawaii Department of Land and Natural Resources, which operates the State's Dam Safety Program. The most recent visual inspection report is dated March 20, 2006 (DLRN 2006). In addition, no attempt was made to evaluate or estimate the potential impact of dam-break flooding on the Big Bridge.

Inundation resulting from a dam-break flood event can be expected to vary depending on a number of factors such as the volume of water in the reservoir at the time of failure, the location and size of the dam breach, topography downstream of the dam, the condition of the flood path at the time of failure (e.g., a dry or flowing stream channel and the presence or absence of other water bodies), and factors that control flow resistance including vegetation and structures in the flood path. It is impossible to know, *a priori*, the conditions of an actual dam-break event. Consequently, the utility of any single model prediction, which must adopt particular settings to represent each of these controlling factors, is of limited value. However, by performing multiple model simulations that adopt a range of model settings, it is possible to estimate the range of inundation patterns that are likely to occur and therefore narrow down the possible range of flooding outcomes. This strategy is adopted for the present study.

Model simulations in this study utilized: two possible water volumes in the reservoir at the time of failure, a single dam-failure configuration involving a slot in the dam, two different water levels in the lagoon at the outlet of the flow path, and two different estimates of a parameter that scales the magnitude of bed resistance used by the model. These factors were all selected with the aim of bracketing the “worst case scenario” relative to flood inundation in Kalihiwai Valley. For this study, the “worst case scenario” represents the largest extent of flooding that can realistically be expected to occur based on all factors known at the time of this study. However, due to the unpredictable variability of natural phenomenon it is impossible to completely rule out the possibility of even greater flooding than that characterized by the “worst case scenario.”

Lastly, it is noted that a mixture of metric and U.S. customary units are cited in this report based on the origin of the data. USGS terrain data used for this study adopt the former, while historical maps and local survey data adopt the latter. All model results are reported in metric units. In addition, horizontal distances reported here are in meters and adopt the Universal Transverse Mercator (UTM) system of projection.

Selection of Water Volumes

Water volumes in the reservoir equal to 40 and 80 million gallons of water were adopted. The smaller of these two volumes corresponds to the reservoir at a normal operating level, i.e., at the level of the spillway or 17 feet relative to the scale used on an old Kilauea Sugar Plantation (KSP) map of the reservoir (Appendix). The larger of these two volumes (80 million gallons) represents the largest volume of water that could realistically be stored in the reservoir at the time of failure. It corresponds to a condition whereby water is overtopping the dam by a height of one foot, assuming the dam height is 20 feet according to the KSP scale. This represents a severe case of overtopping that an earthen dam would not be designed to withstand. No attempt was made to estimate the risk of such a condition based on hydrologic and meteorological factors. Furthermore, it is not known whether such a condition could or would ever develop.

The values of 40 and 80 million gallons were estimated based on the *ca.* 1958 KSP map of the reservoir (Appendix) and the associated table which lists water volumes at various water levels. Based on correspondence with Mr. Sterling Chisholm, a life-long resident of the area who supplied this and other maps, the 19 foot level corresponded to the spillway height prior to the 1980s and the 17 foot level roughly corresponds to the present spillway height (Chisholm 2006). That is, the spillway was reportedly lowered during the 1980s. To obtain the volume associated with water at the 21 foot level (one foot above the dam), United States Geologic Survey (USGS) one-third arc-second terrain data were utilized to determine the volume between the 19 and 21 foot levels.

According to the State of Hawaii, Department of Land and Natural Resources, Visual Dam Safety Inspection Sheet dated March 20, 2006, the normal and maximum storage of Kalihiwai Reservoir are 90 and 140 million gallons, respectively (DLNR 2006). These volumes were not used for this study because they did not appear credible based on the surface area of the reservoir, the height of the dam, and the volumes reported in KSP documentation (see Appendix 1). DLNR staff did not respond to an email inquiry about this discrepancy.

Dam-failure Configuration

For flood simulation purposes it is necessary to assume that the dam fails through a breach of some kind so the model can predict the resulting inundation. For this study, a single “worst case scenario” was adopted. It was assumed that the dam instantaneously fails creating a roughly 200 foot (61 m) wide and 20 foot (6 m) deep slot in the dam along its eastern half. The eastern side was found to be slightly lower than the western side, based on a recent survey (Appendix), which motivated the placement of the breach in this location. The depth of the opening was selected so the entire volume of the reservoir would eventually drain over the course of the model simulation. That is, using a more shallow breach a fraction of the reservoir volume would remain in the reservoir after failure and this would lead to less inundation downstream. The width was selected to be roughly one third the length of the dam. This value is large compared to formulae in the literature which suggest a breach width roughly 3 times the depth of the dam (Wahl 2004), consistent with the “worst case scenario” approach adopted for this study.

Flow Resistance

Flow resistance in the model is scaled by a parameter called the Manning coefficient (e.g. Chow 1959), and simulations were run using a Manning coefficient of either 0.05 or 0.1. There is presently very little research to support a precise characterization of this parameter for dam break inundation studies. Based on channel flow studies, these values correspond to a range of vegetated conditions and this motivated their selection. As a point of reference, the simulation of flooding recently prepared by the authors for Ka Loko Reservoir was based on a Manning coefficient equal to 0.05.

Flood Simulation Method

Flood simulations were carried out using BreZo flood modeling software (FlowSimulation, LLC, Irvine, CA). BreZo predicts water motion by solving the shallow-water equations using a Godunov-type finite volume scheme. A description of the theory and numerical scheme adopted by BreZo has been published the *ASCE Journal of Hydraulic Engineering* (Begnudelli and Sanders, 2006).

BreZo runs on an unstructured mesh of triangular cells, and for this study mesh generation was accomplished using a Delaunay mesh generator. A variable mesh resolution was used. In the path of flood waters, a grid resolution of roughly 3-5 meters (10-16 ft) was adopted while a coarser resolution was used outside the flow path. The mesh included roughly 80,000 cells, and a convergence test was done with a mesh containing roughly 120,000 cells. Fig. 2 shows the computational grid around the Big Bridge (left) and lagoon outlet (right).

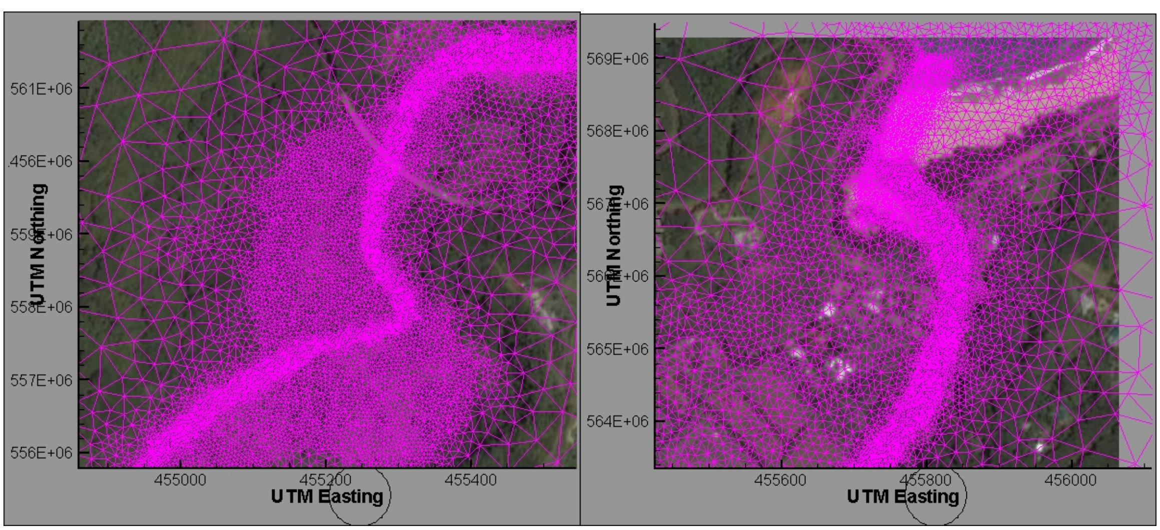


Figure 2: The model runs on a grid of triangular cells. Here the cells around the Big Bridge (left) and the Lagoon Outlet (right) are shown.

BreZo requires two key sets of input parameters: terrain data and resistance data. The primary source of terrain data was the one-third arc-second National Elevation Data (NED) supported by the United States Geologic Survey (USGS) and available on-line at http://seamless.usgs.gov. These data were selected because NED can be obtained for free on-line, because no better source of data was available, and because USGS data such as this has been previously used by the US Army Corps of Engineers and other agencies for dam-break modeling studies. The resolution of these data is roughly 10 meters in the horizontal direction. The stated vertical accuracy of the NED is 7 m, but the USGS states that this accuracy varies with location. No specific information for the accuracy of the data at this site is available. NED elevations are reported relative to North American Vertical Datum 1988 (NAVD 88) according to metadata distributed by the USGS. However, this is illogical because Hawaii is not in North America. Follow up discussions with Drs. Dan Roman and Ed Carlson of the NOAA National Geodetic Survey clarified that the vertical control is Mean Sea Level (MSL) at the Waimea tidal bench mark of 1964. Three additional sources of data were utilized to improve the mapping of terrain in the vicinity of Kalihiwai Lagoon, where there are buildings. These data sources included aerial imagery from http://maps.google.com, ground survey data and Interferometric Synthetic Aperature Radar (IfSAR) data collected by Intermap Corp. and made available at no-cost by the NOAA Coastal Services Center. These data sources were utilized to improve the mapping of terrain in the vicinity of Kalihiwai Lagoon. In addition, tidal data were obtained and reviewed to establish water levels in the lagoon.

Ground surveying was conducted by Mr. Sterling Chisholm (a retired building contractor with many years of surveying experience) to estimate the depth and width of the channel, ground elevation along lagoon banks, ground elevation at buildings, and the height of buildings (several of which are built on elevated platforms to guard against coastal flood hazards such as tsunamis). Surveying was conducted in three phases: (1) depth of lagoon and height of lagoon banks, (2) width of the lagoon and (3) ground elevation and height of buildings. Survey results are included as an Appendix and Fig. 3 shows buildings that were surveyed. These are labeled A-X. Survey heights were measured in feet relative to the lagoon level. The outlet of the lagoon was reported partially blocked at the time of the surveys by Mr. Chisholm, a condition typical of the summer season, and the water level in the lagoon was found to be equal to the high tide level in Kalihiwai Bay. Note that tidal amplitudes on Kauai are small, roughly 0.3 m.

IfSAR data were collected *ca.* 2005 by flying an aircraft and scanning the ground below with a radar sensor. In general, IfSAR data are of limited use for flood modeling for two reasons. First, the sensor will measure the heights of treetops and buildings instead of the ground surface when such features cover the ground surface. In addition, surfaces mapped by IfSAR will appear wavy even if they are flat, and the amplitude of this waviness is roughly 1 meter and this waviness can obscure real flow paths. Nevertheless, there are open fields along the lagoon and the IfSAR data were obtained with the aim of improving the representation of terrain here. IfSAR data were also reported relative to NAVD 88.

Figure 3: Aerial imagery of terrain along western bank of lagoon. Buildings labeled A-X were surveyed for ground elevation and building height above the ground.

Tidal bench mark data for Kauai, including measurements at Nawiliwili, Port Allen, and Waimea indicate that Mean Higher-High Water (MHHW) is roughly 0.3 m above mean sea level, the highest recorded tides are at most 1.3 m (4.3 ft) above mean sea level.

Ground survey data, measured relative to the lagoon level, were first converted to elevations comparable to the NED by assuming a lagoon level of 0.3 m. However, NED and ground survey measurements were inconsistent based on this approach. For example, the NED predicted lagoon bank elevations of roughly 4 meters while ground survey data predicted elevations of roughly 1 meter. To bring the data in better agreement, MHHW (and the level of the lagoon) was taken to be 3 m and the ground survey data were again converted to elevations. For example, an elevation measured to be 2 ft above the lagoon level was assigned an elevation of 3.0 m + 2 ft * (0.3048 m/ft) = 3.6 m. Lagoon depths were converted to ground elevation in a similar way, but by subtraction. For example, a depth of 10 ft was assigned an elevation of 3.0 m – 10 ft * (0.3048 m/ft) = -0.05 m.

Next, Tecplot visualization software (Tecplot Inc., Bellevue, WA) was used to superimpose elevation contours based on NED, IfSAR, survey measurements, and aerial imagery in the vicinity of the lagoon and to create a computational grid that leveraged the best available information. Aerial imagery was used to define the width and path of the lagoon channel, the width and path of the channel through the Taro Patch, and the overbank area on the western side of the lagoon where buildings were observed. The channel widths defined by aerial imagery were compared to survey measurements, and a very good agreement was achieved (within 5%). To prepare the computational grid, bed elevation at all nodes was first defined by NED and then updated as follows: (a) the lagoon channel path and width was defined based on aerial imagery and the elevation was interpolated from survey measurements by an inverse-distance methodology (b) the Taro Patch channel path and width was defined based on aerial imagery and the elevation was set 1 m below the bank height, (c) overbank elevations west of the lagoon were interpolated by an inverse-distance methodology using survey data and additional hypothetical survey points. These hypothetical survey points were established using the IfSAR data. First, a set of 16 elevations were randomly sampled from a roughly 50 x 50 m swath and averaged to filter out waviness in the IfSAR. This process was repeated for several other open-field swaths (stations) including a station where a survey measurement had been taken (Survey Point "S", see Appendix I). This station served to ground-truth all of the hypothetical stations. The final, synthesized topographic dataset used in the model is shown in Fig. 4, along with topography based on NED for comparison purposes. Note that exclusive use of NED would cause underestimation of flood inundation for two reasons: first, the NED overestimates the width (and conveyance capacity) of the lagoon channel, and second, the NED overestimates the height of terrain where buildings are present.

The approach used here to define terrain elevations made best use of all available information, but it is not ideal. A better approach is to carry out a comprehensive ground survey or Light Detection and Ranging (LiDAR) survey (e.g. http://www.lidar.org). However, it is understood that a survey is not possible at this time because it is cost prohibitive.

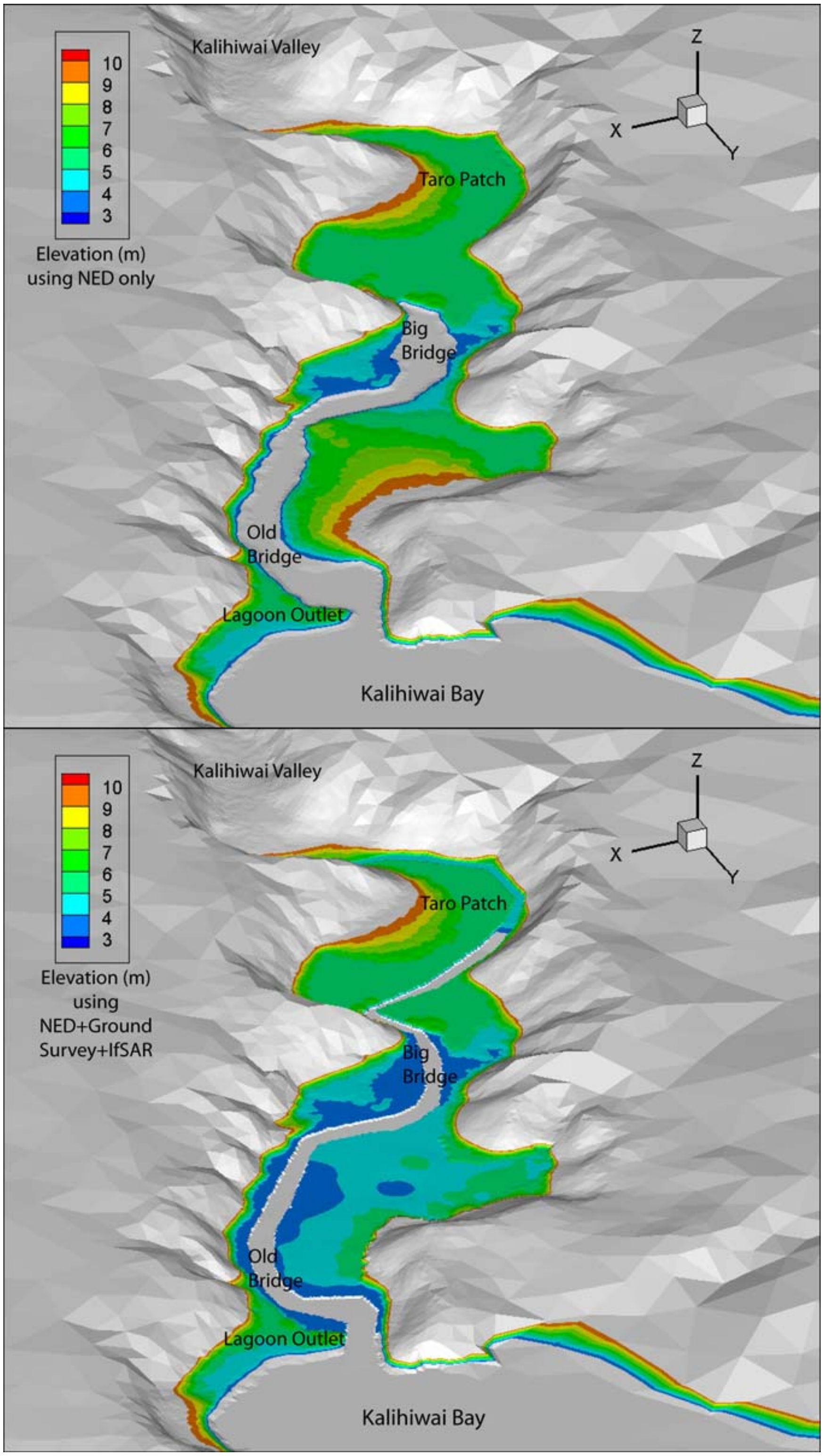


Figure 4: Model depiction of terrain based on NED data only (top) and synthesized data: NED+IfSAR+ground survey data (bottom). Flood modeling performed with synthesized dataset.

BreZo also requires an initial condition, which in this case represents the volume of water initially in the reservoir and the river channel. As was noted earlier in this report, the reservoir volume was set to either 40 or 80 million gallons. To initialize the river channel, the model was pre-run to a steady state with water flowing at 28 cubic meters per second (1000 cubic feet per second or 7,500 gallons per second) from a point source upstream of Kalihiwai Reservoir along Kalihiway Valley. The steady state solution to the river-flow problem was saved as an initial condition for the dam-break simulation, and during the dam-break simulation water continued to flow from the previously mentioned point source so the effects of river water and reservoir water were additive during the simulation. The river flow rate was based on USGS stream flow records for Kalihiwai River (USGS Gage 16098000) which cover the period 1914-1923. The value used (1000 cfs) is slightly larger than the largest average daily flow rate on record (967 cfs; Sept. 25, 1914) again consistent with the “worst case scenario” approach adopted for this study.

The ocean level must also be specified to run BreZo, both to carry out the initialization procedure described above and to continue with the dam-break flood simulation. The principles of flood hydraulics dictate, for relatively flat terrain, that over-bank flooding along rivers is exacerbated by higher downstream water levels. Simulations were carried out using a downstream water level of 3.0 and 3.3 meters. Accounting for the datum offsets described earlier, these levels correspond to MHHW and one foot above MHHW, respectively.

Once BreZo was initialized, the prediction was advanced in time using a time step of 0.025 seconds. The model was run for 140,000 time steps, or a period of just over 58 minutes. The solution in all cells was saved at 2.5 minute intervals for animation purposes. Second, the solution in four selected cells roughly aligned with the Lagoon Outlet, Old Bridge, Big Bridge, and Taro Patch was saved every five seconds for time series analysis. The location of these stations is noted in Fig. 4. Third, the maximum depth at each location was also saved to a file. Visualization of BreZo results, including both static images and computer animations, was accomplished using Tecplot software (Tecplot Inc., Bellevue, WA).

BreZo has excellent conservation properties. In all simulations the global mass conservation error was negligible, roughly 10^{-15}%. BreZo simulations do not account for infiltration of water into the soil or changes in topography resulting from flow induced erosion or sediment deposition. In addition, BreZo simulations cannot account for channel flow at horizontal scales smaller than the grid resolution which in this study was set between 3 and 5 meters. Predictions of very shallow inundation (say, less than 10 cm) are therefore likely to overestimate the spreading of water and underestimate the peak velocity of water in sub-grid scale channels and rivulets.

Four simulations were carried out based on four different model configurations. Whereas all simulations adopted the same breach configuration and river base-flow as described above, factors that were varied included: the reservoir volume, the Manning coefficient of resistance and the downstream water level (ocean level). The precise configuration of each simulation is as follows:

Simulation 1 (Base Case): A reservoir volume of 80 million gallons, a Manning coefficient equal to 0.05 (n=0.05), a downstream water level of 3.0 meters.

Simulation 2 (Average Reservoir Volume): A reservoir volume of 40 million gallons, a Manning coefficient equal to 0.05 (n=0.05), and a downstream water level of 3.0 meters.

Simulation 3 (Greater Flow Resistance): A reservoir volume of 80 million gallons, a Manning coefficient equal to 0.1 (n=0.1), and a downstream water level of 3.0 meters.

Simulation 4 (Higher Ocean Level and Greater Flow Resistance): A reservoir volume of 80 million gallons, a Manning coefficient equal to 0.1 (n=0.1), and a downstream water level of 3.3 meters.

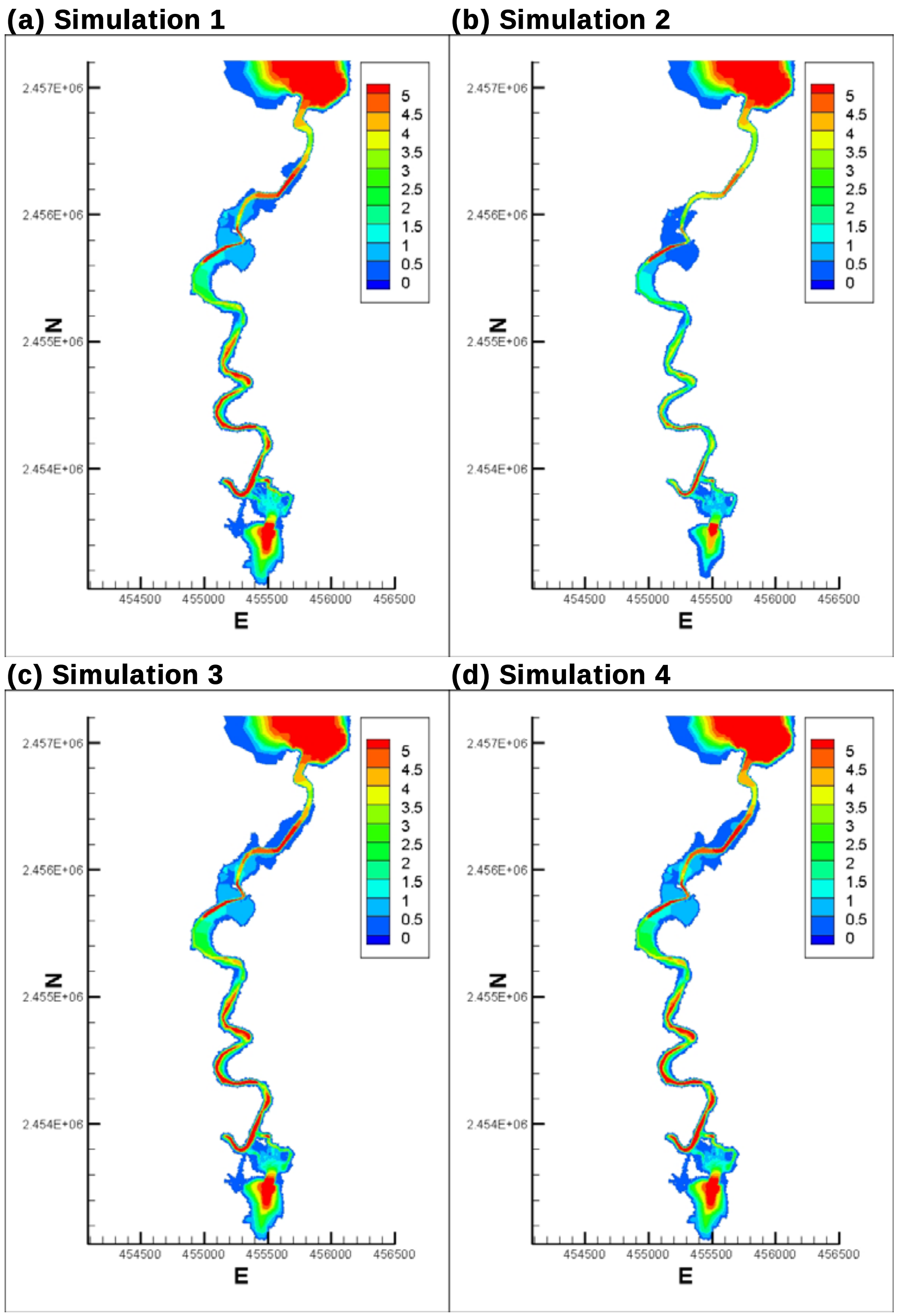


Figure 5a-d: Contours of maximum flood depth in meters for Sims. 1-4 showing the relative effect of reservoir volume, flow resistance and ocean level on flood inundation predictions. Note that all simulations show the Taro Patch completely inundated, and the extent of flooding along Kalihiwai Lagoon varies between simulations.

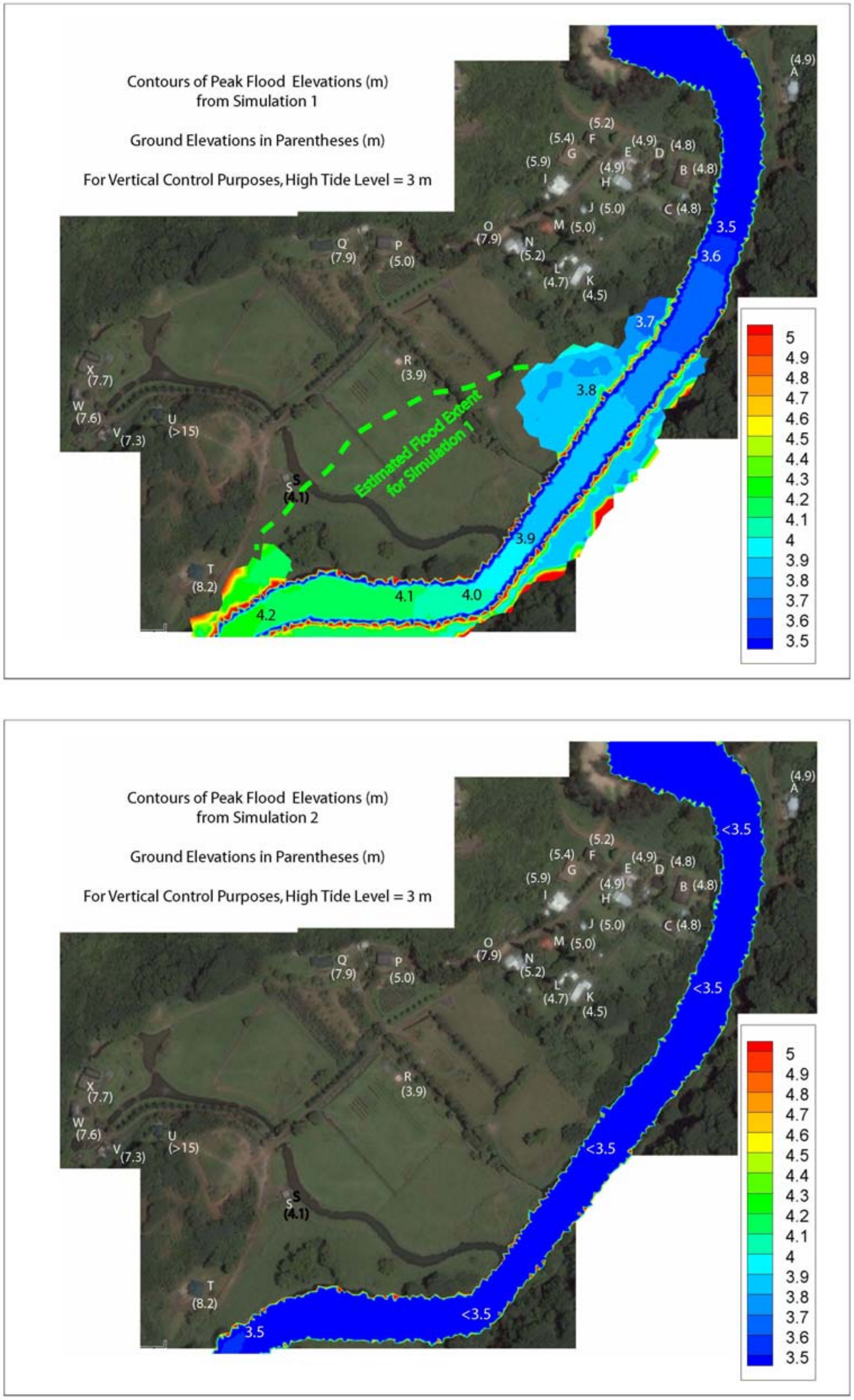
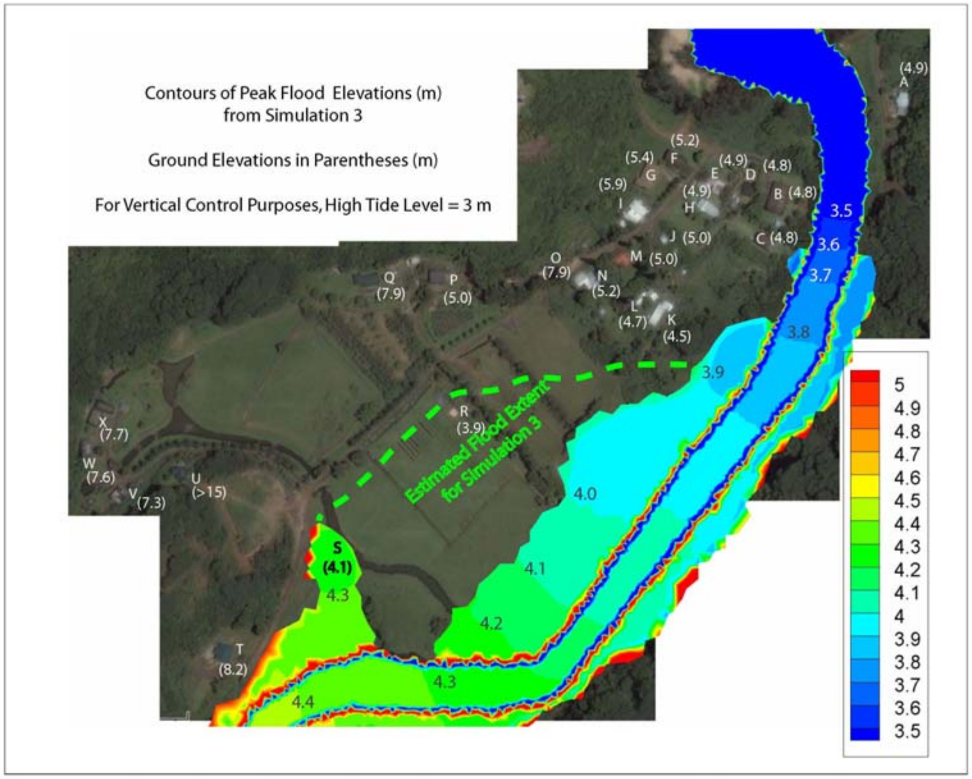


Figure 6a-b: Contours of flood elevations and ground elevations of buildings for Sims. 1 and 2. Ground elevations at buildings are shown in parentheses.



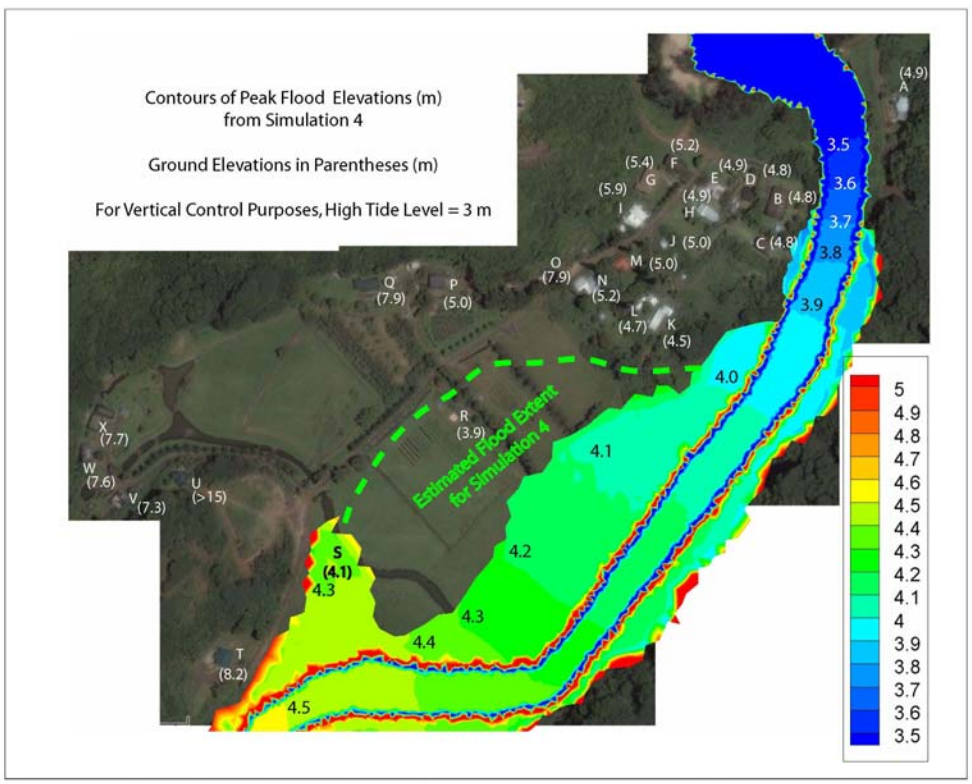


Figure 6c-d: Contours of flood elevations and ground elevations of buildings for Sims. 3 and 4. Ground elevations at buildings are shown in parentheses.

Flood Simulation Results

Fig. 5a-d presents contour plots of the maximum flood depth predicted by Sims. 1-4, respectively. This figure shows the sensitivity of model predictions to the reservoir volume, resistance factor and downstream water level. Fig. 6a-d presents contours of maximum flood elevations in the vicinity of the lagoon superimposed upon an aerial image of the site for geo-referencing purposes. Ground elevation is also shown to interpret the susceptibility of buildings to flooding. Note that several of these buildings are built on elevated platforms, 2-3 meters above the values reported in Fig. 5. All heights are reported in the Appendix.

Fig. 6 includes a dashed line that bounds an area labeled “Estimated Flood Extent”. This was added because close inspection of the final terrain map shown in Fig. 4 revealed a subtle ridge 0.1-0.2 m high running parallel the lagoon, like a levee that guarded point “R” (see Fig. 6) from flooding. However, this ridge is an artifact of the interpolation process, and not grounded in survey data. Recall that survey data were clustered around buildings, not open space areas such as this. Therefore, any flood protection that it seemingly offers cannot be justified. The “Estimated Flood Extent” represents the area of inundation that the model would have most likely predicted if the ridge was not present in the final terrain map used by the flood model.

Fig. 8 presents time series of flood heights (or stage) predicted at four stations: the Lagoon Outlet, the Old Bridge, the Big Bridge and the Taro Patch (at its western most point).

Trends in Results

All simulations depict flooding of the Taro Patch, the low-lying terrain south of the Big Bridge. As shown in Fig. 8, the model predicts water as deep as 3 m (10 ft) based on an 80 million gallon release and roughly 2 m (6 ft) based on a 40 million gallon release. Between the Big Bridge and the Old Bridge, model predictions are sensitive to the reservoir volume. Based on 40 million gallons, the model predicts the flood will be contained within the lagoon channel. But based on 80 million gallons, the model predicts over-bank flooding. The extent of lateral flood varies between Simulations 1, 3 and 4, with Simulation 4 showing the highest flood stages and the largest flood extent.

Sims. 2 and 3 (see Figs. 5 and 6) show that a doubling of the Manning coefficient leads to only a minor increase in flood depths and area of inundation predicted by the model. However, Fig. 8 shows that the larger Manning coefficient slows the speed of the flood wave leading to a longer travel time.

Sims. 3 and 4 (see Figs. 5 and 6) show that a 0.3 m (1 ft) increase in the ocean level will leads to more over-bank flooding along the lagoon between the Old Bridge and the Big Bridge.

The hydrographs (Fig. 8) indicate that the ocean level influence on flood stage diminishes with distance inland, while the dam-break influence on flood stage increases with distance inland. At the Lagoon Outlet, Fig. 8 shows that flood stage is primarily dependent on the ocean level which is to be expected based on the proximity of this station to the ocean. At the Old Bridge, Fig. 8 shows flood stage is also strongly affected by the ocean level and in addition, water levels are predicted to rise by as much as 0.65 m (2.1 ft) due to the dam-break. At the Big Bridge, flood stage is predicted to rise by as much as 1.9 m (6.2 ft) due to the dam-break, and in the Taro Patch flood stage is predicted to rise up to 3 m (10 ft).

Based on Fig. 8, the model predicts that the flood wave will reach the Taro Patch somewhere between 10 and 16 minutes after dam failure, depending on the amount of water in the reservoir and the amount of flow resistance encountered by the flood wave. The flood wave here could be characterized as a “wall of water”, because water levels are predicted to rise several meters over a period of 1-2 minutes. Flood waters are predicted to reach the Big Bridge between 11 and 18 minutes after failure and flood waters will continue to rise for another 10-15 minutes before slowly receding. Flood waters are predicted to reach the Old Bridge 15-20 minutes after dam failure and rise for 15-20 minutes before receding. Finally flood waters are predicted to reach the Lagoon Outlet 16-22 minutes after dam failure and rise for 15-20 minutes before receding. Overall, the model predicts that peak flooding will occur no later than 45 minutes after dam failure throughout the study site, and dam-break flood waters are predicted to recede over a similar time scale.

Two factors help to mitigate over-bank flooding along Kalihiwai Lagoon. First, the Taro Patch acts as a natural detention basin. The accumulation of flood waters here decreases the volumetric flow rate of water entering the lagoon area. Second, the conveyance capacity of the lagoon channel is substantial given its breadth and depth.

Vulnerability of Buildings to Flooding

Two buildings along the lagoon appear vulnerable to dam-break flooding, those labeled R and S in Fig. 6. These correspond to a “small illegal structure” and a “boat storage structure”, respectively, based on comments in the ground survey report (Appendix). Peak flood elevations exceed the ground elevation by roughly 0.2 m (0.7 ft) at both of these sites based on flood Simulation 4.

Residences along the lagoon are built on land that is least 0.5 m (1.6 ft) above peak flood stage predicted by the model, based on the residence labeled K in Fig. 6 which is the limiting case. The floor height of K is 1.0 m above the ground, so the clearance (difference between floor and flood height) is 1.5 m (4.9 ft). Residence L is built on slightly higher terrain, but the floor height is less than that of L and the clearance is 1.3 m. Other buildings appear close to flood waters based on Fig. 6, e.g. A, B, C and T.

However, ground elevation at these sites is more than 1.0 m (3.3 ft) above peak flood stage and there is at least 3.0 m (10 ft) of clearance. The clearance of buildings D and E is 1.5 m (4.9 ft), less than that of A, B, C and T.

Summary and Recommendations

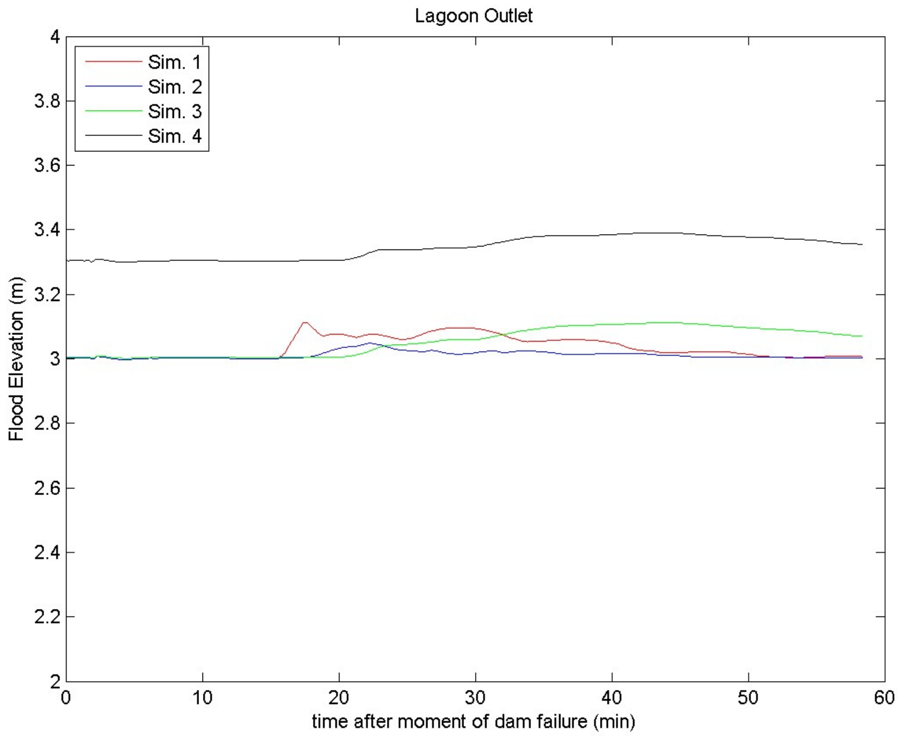
Modeling was carried out to map terrain in Kalihiwai Valley susceptible to flooding as a result of a catastrophic failure of Kalihiwai Reservoir dam. Model predictions are sensitive to many factors that relate to the physical setting and these cannot possibly be precisely known. Therefore, parameters were selected with the aim of bracketing the “worst case scenario” relative to dam break flooding. This involved the reservoir filled higher than the height of the dam (an overtopping condition), Kalihiwai River flowing with the highest flow rate on record, and the ocean at a high tide.

Under the “worst case scenario,” the model predicts complete inundation of the Taro Patch, the low-lying terrain south of the Big Bridge, with water as deep as 3 m (10 ft). Between the Big Bridge and the Old Bridge and along the lagoon, the model predicts over-bank flooding in the vicinity of buildings. Two non-residential structures characterized as a “boat storage structure” and a “small illegal structure” by surveyors are predicted to be inundated by as much as 0.2 m (0.7 ft) of water. No residential structures are predicted to be inundated. Flood stages would need to rise another 0.5 m (1.6 ft) to reach the foundation of residence K, 1.3 m (4.3 ft) to reach the first floor of residence L, and at least 3 meters to inundate residences built on elevated platforms (e.g. A, B, and C).

With the reservoir filled to its normal operating level, i.e. the height of the spillway, the model predicts that a dam-break flood would inundate the Taro Patch but that flood waters would be contained by the lagoon channel downstream of the Big Bridge. Hence, the model predicts that no buildings would be impacted by a dam-break flood based on the reservoir filled to its normal operating level.

That residences are built on elevated platforms serves as a reminder of coastal hazards such as tsunamis, storm surge, and even sea level rise. Surely the design of these stilted structures was motivated by the risk posed by coastal hazards and not the risk posed by a possible dam failure. Indeed, simulations presented in this report show the sensitivity of flood stages to the ocean level, particularly at the Lagoon Outlet and Old Bridge stations.

Were a dam-break flood ever to happen, two factors would help to minimize over-bank inundation along the lagoon. First, the Taro Patch acts as a natural detention basin. Second, the lagoon channel offers substantial flood conveyance capacity due to its breadth and depth.



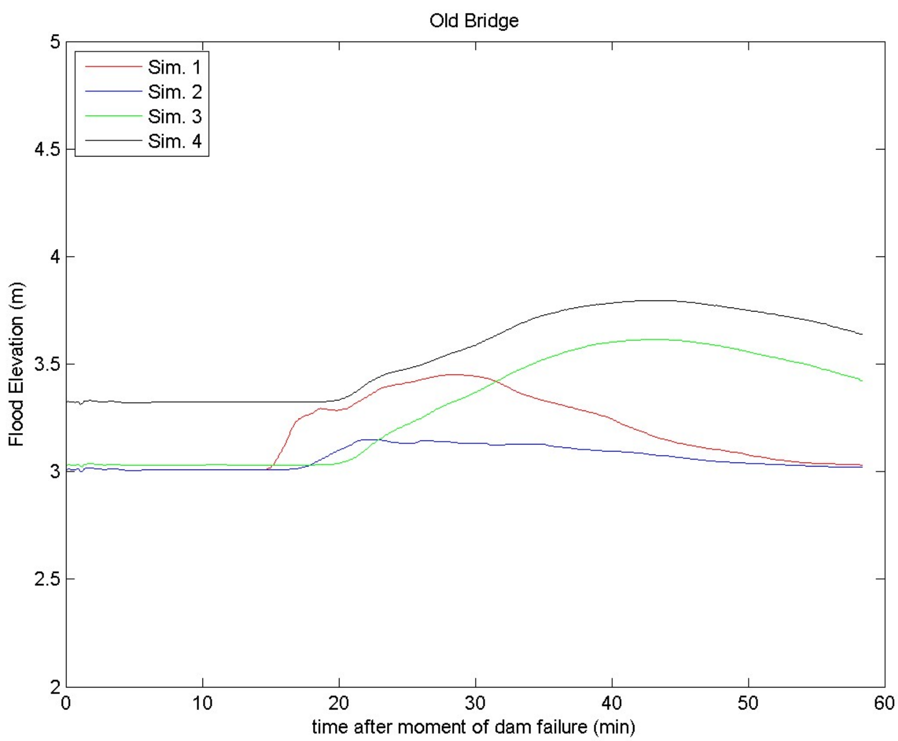


Figure 8a-b: Time series of flood elevation predicted at Lagoon Outlet (a-top) and Old Bridge (b-bottom). Flood elevation at time zero differs between simulations due to Manning coefficient value and ocean water level.

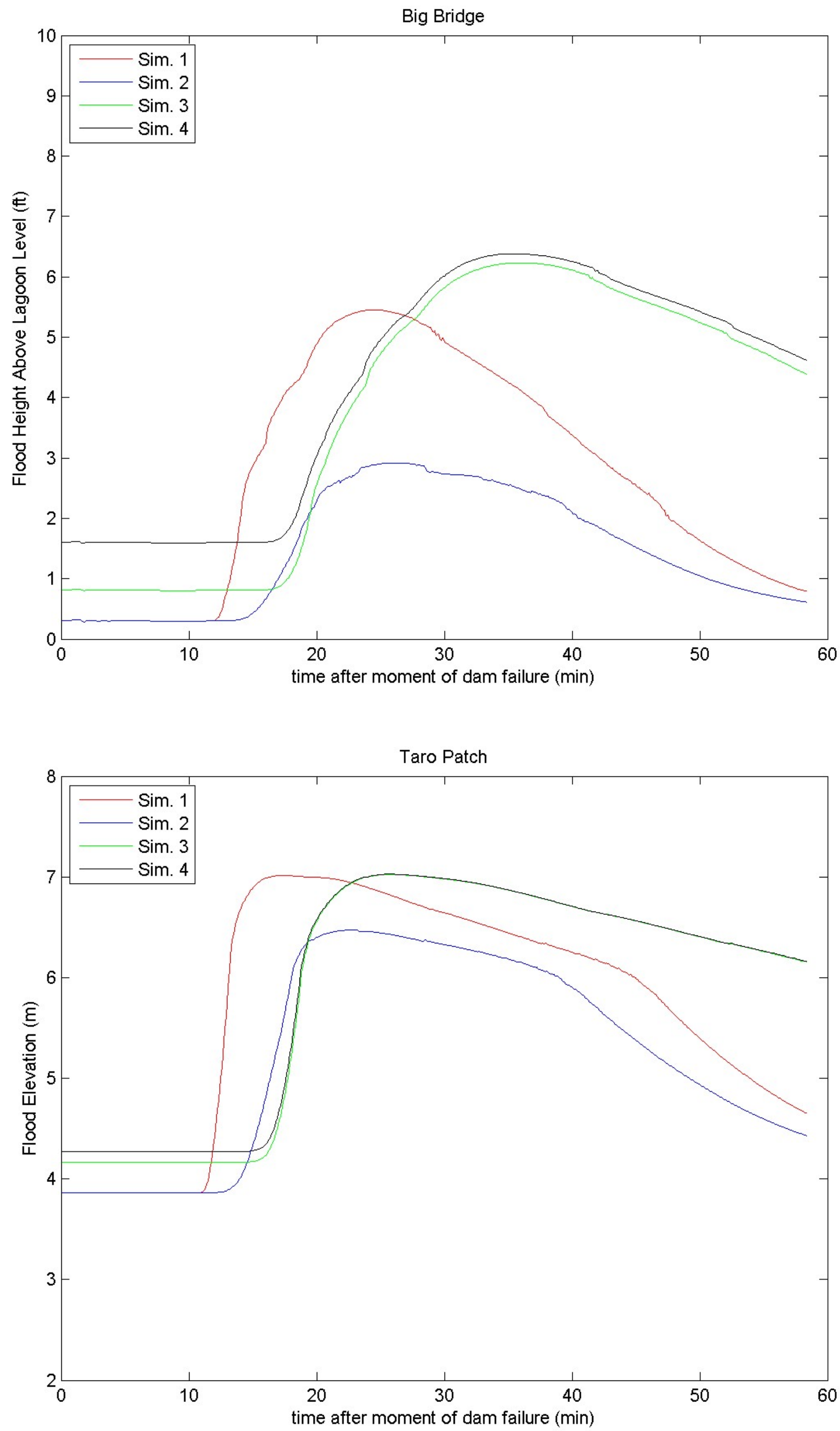


Figure 8c-d: Time series of flood elevation predicted at Big Bridge (c-top) and Taro Patch (d-bottom). Flood elevation at time zero differs between simulations due to Manning coefficient value and ocean water level.

References

Begnudelli, L. and Sanders, B.F. (2006) Unstructured Grid Finite Volume Algorithm for Shallow-water Flow and Transport with Wetting and Drying, *ASCE Journal of Hydraulic Engineering*, 132(4), 371-384.

Chisholm, S. C. (2006) Letter to B.F. Sanders dated July 21, 2006.

Chow, V.-T. (1959) *Open-Channel Hydraulics*, McGraw-Hill, 197 p.

Department of Land and Natural Resources, State of Hawaii (2001) Table 5.21 posted at http://www.state.hi.us/dlnr/cwrm/data/facts.htm

Department of Land and Natural Resources, State of Hawaii (2006) Visual Dam Safety Inspection Sheet for Kalihiwai Reservoir dated 20 March, 2006. http://www.hawaii.gov/dlnr/reports/dam-inspections/Kalihiwai.pdf

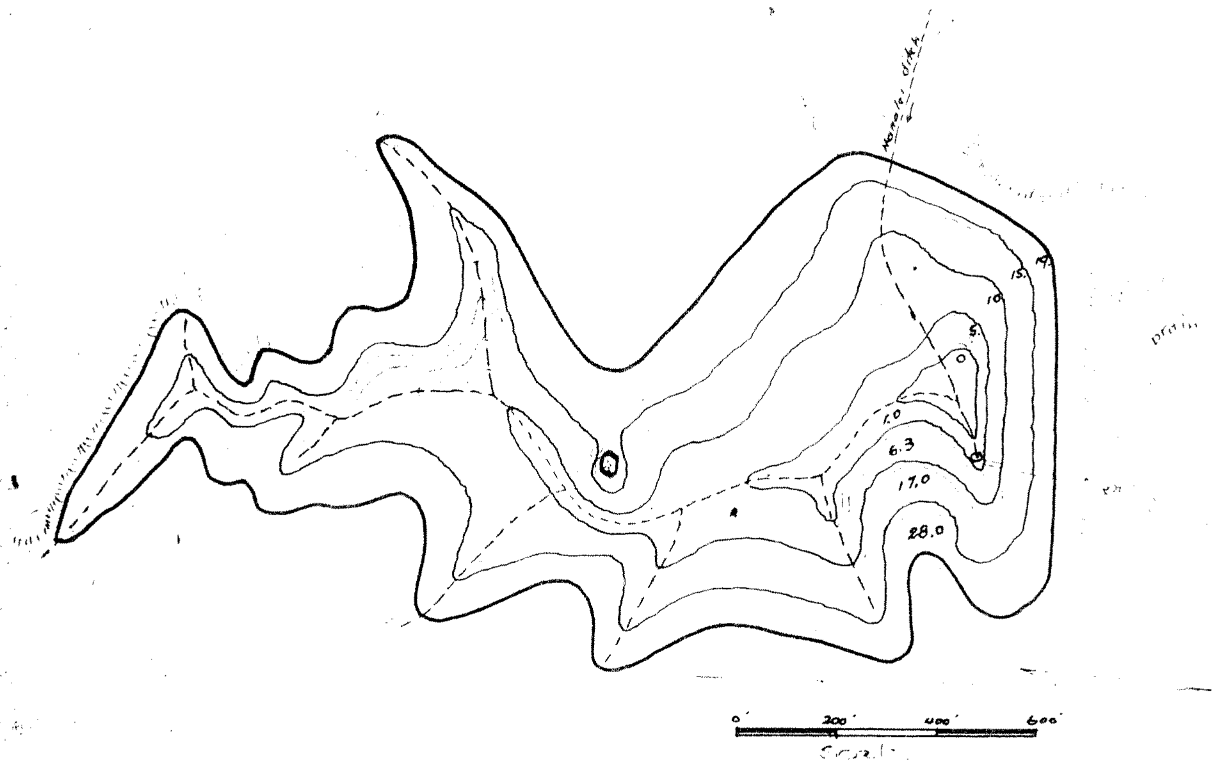
Kilauea Sugar Plantation (1958) Exhibit entitled "Kalihiwai Reservoir, Kilauea, Kauai" and dated Nov, 1958. Exhibit includes a sketch of the reservoir bathymetry and a table of reservoir storage versus water level in units of acre-inches and feet, respectively. Exhibit supplied to B.F. Sanders by S.C Chisholm.

Wahl, T.L. (2004) Uncertainty of Predictions of Embankment Dam Breach Parameters, *ASCE Journal of Hydraulic Engineering*, 130(5), 389-397.

Appendix

Part 1 of 5: Kilauea Sugar Plantation Maps of Kalihiwai Reservoir and Storage Tables

EXHIBIT - 6



KALIHIWAI RESERVOIR

Kilauea Kauai

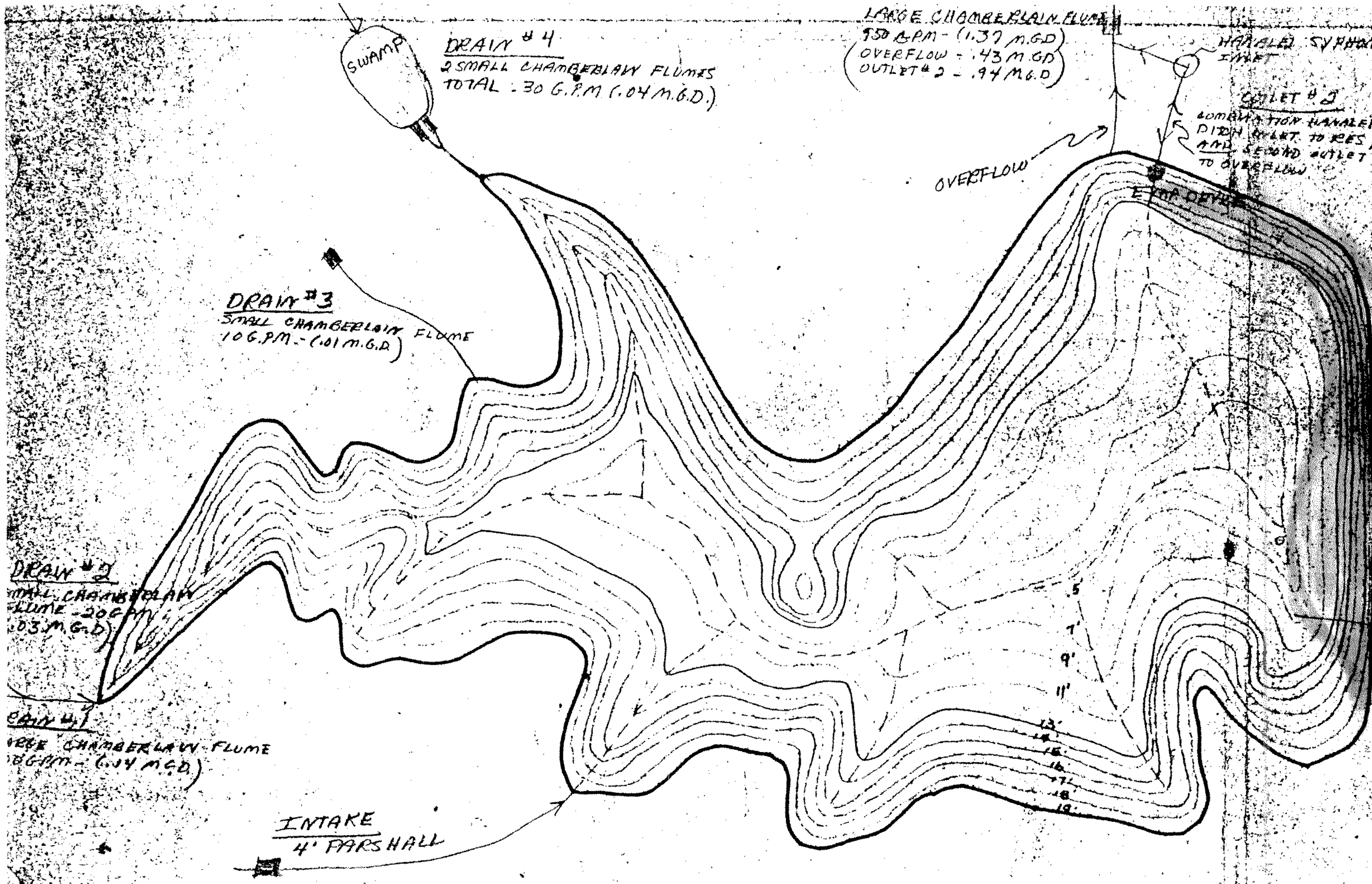
1958

No. Ft.	Area	Acre Inches Per Foot	Total Acre Inches
19	28.00	336.89	2119.31
18	25.00	300.79	1782.42
17	22.50	270.71	1481.63
16	19.70	237.02	1210.92
15	17.00	204.54	973.90
14	14.50	174.46	769.36
13	12.00	144.38	594.90
12	9.70	116.71	450.52
11	7.60	91.44	333.8[illegible]
10	6.30	75.43	242.3[illegible]
9	5.00	60.16	166.[illegible]
8	3.70	44.52	106.7[illegible]
7	2.40	28.88	62.2[illegible]
6	1.40	16.84	33.3[illegible]
5	1.00	12.03	16.54
4			
3			
2			
1			
0	.50		

NOV, 1958

KALIHIWAI RESERVOIR

Kilauea Kauai



Ft	Area	MG per Ft.	Total MG
19	28.0	9 147600	4 4953290
18	25.0	8 167500	3 5805690
17	22.5	7 350750	2 7638190
16	19.7	6 435990	2 0287440
15	17.0	5 553900	1 3851450
14	14.5	4 737150	6297550
13	12.0	3 920400	3920400
12	9.7	3 168990	1 2110570
11	7.6	2 482420	8041580
10	6.3	2 048210	6458660
9	5.0	1 633500	4410450
8	3.7	1 208790	2776950
7	2.4	784080	1568160
6	1.4	457380	784080
5	1.0	326700	326700
0	.5	122512	

date ?

KILAUEA AGRONOMICS, INC.
P.O. Box 80, Kilauea, HI 96754
Telephone: (808)828-1761 Fax: (808)828-1880
A C. Brewer Company

1958 data

KALIHIWAI RESERVOIR
Kilauea, Kauai, HI

NO. FT.	AREA	ACRE IN/FT	TOTAL ACRE IN
19	28.0	336.89	2119.31
18	25.0	300.79	1782.42
17	22.5	270.71	1481.63
16	19.7	237.02	1210.92
15	17.0	204.54	973.90
14	14.5	174.46	769.36
13	12.0	144.38	594.90
12	9.7	116.71	450.52
11	7.6	91.44	333.81
10	6.3	75.43	242.37
09	5.0	60.16	166.34
08	3.7	44.52	106.78
07	2.4	28.88	62.26
06	1.4	16.84	33.38
05	1.0	12.03	16.54
04	0.8		

HD#8 KAL'RES

KALIHIWAI RESERVOIR
Kilauea, Kauai, HI

NO. FT.	AREA	ACRE IN/FT	TOTAL ACRE IN
19	28.0	336.89	2119.31
18	25.0	300.79	1782.42
17	22.5	270.71	1481.63
16	19.7	237.02	1210.92
15	17.0	204.54	973.90
14	14.5	174.46	769.36
13	12.0	144.38	594.90
12	9.7	116.71	450.52
11	7.6	91.44	333.81
10	6.3	75.43	242.37
09	5.0	60.16	166.34
08	3.7	44.52	106.78
07	2.4	28.88	62.26
06	1.4	16.84	33.38
05	1.0	12.03	16.54
04	0.8		

Appendix

Part 3 of 5: Survey of Lagoon Depth and Bank Elevation by Mr. Sterling C. Chisholm

2006 Survey

3/25/06

LAKE LOT DAM ADJUSTMENTS

SPILLWAY TO BE ZERO

STATION				
1 - ~~BLUE~~ BASE MARBLE TREE (TROY'S PEN.)	+ ~~1.66~~ 1.44	FEET	ABOVE	SPILLWAY
2 - TOP OF DAM	+3.84	"	"	"
3 - TOP OF DAM.	+3.07	"	"	"
4 - TOP OF DAM	+2.76	"	"	"
5 - TOP OF DAM	+4.14	"	"	"
6 - TOP OF DAM.	+4.71	"	"	"
7 - TOP OF DAM.	+4.32	"	"	"
8 - TOP OF ~~SPILLWAY~~ THROW AWAY GATE	-3.~~8~~1	"	BELOW	SPILLWAY
8A. BOTTOM OF THROW AWAY	-6.21	"	"	"
9 TOP OF SPILLWAY	0	"	—	—
10 PVC COUPLINGS	-1.93	"	BELOW	SPILLWAY
11 LAKE LEVEL 3/25/06	-3.47	"	"	"
12 EDGE OF GRASS	+0.16	"	ABOVE	SPILLWAY
12A 35' FROM GRASS - EARL.	-1.23	"	BELOW	"
13 ISLAND (REEF)	-4.03	"	BELOW	SPILLWAY
14 STERLING'S POINT METAL POST.	-2.84	"	"	"

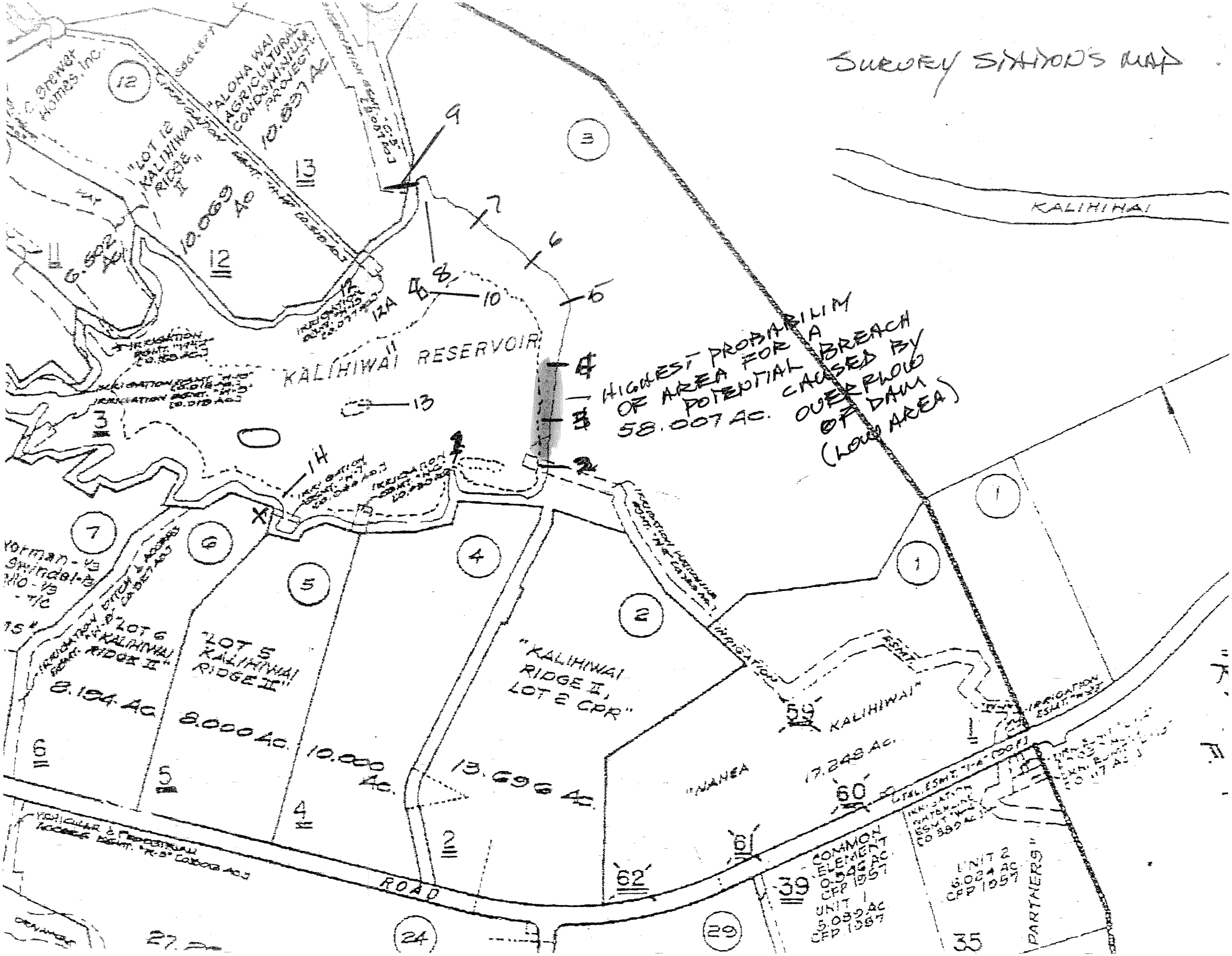
ALL SHOTS TAKEN FROM POINT ON LOT 6

SEE ATTACHED MAP

3½" FROM SURFACE TO TOP OF SPILLWAY

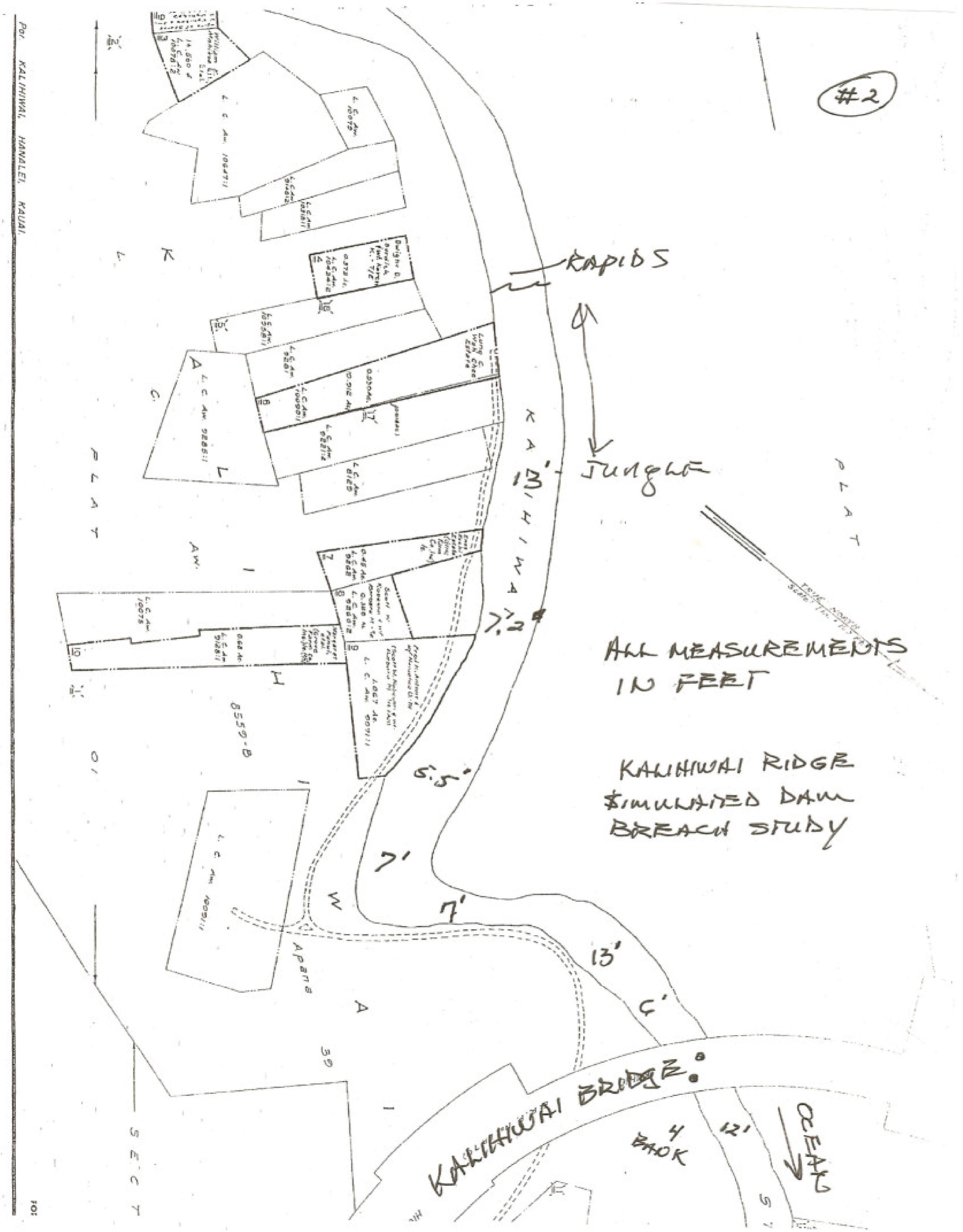
17 DEPTH FROM TOP OF SPILLWAY @ OUTLET - 20' MAX

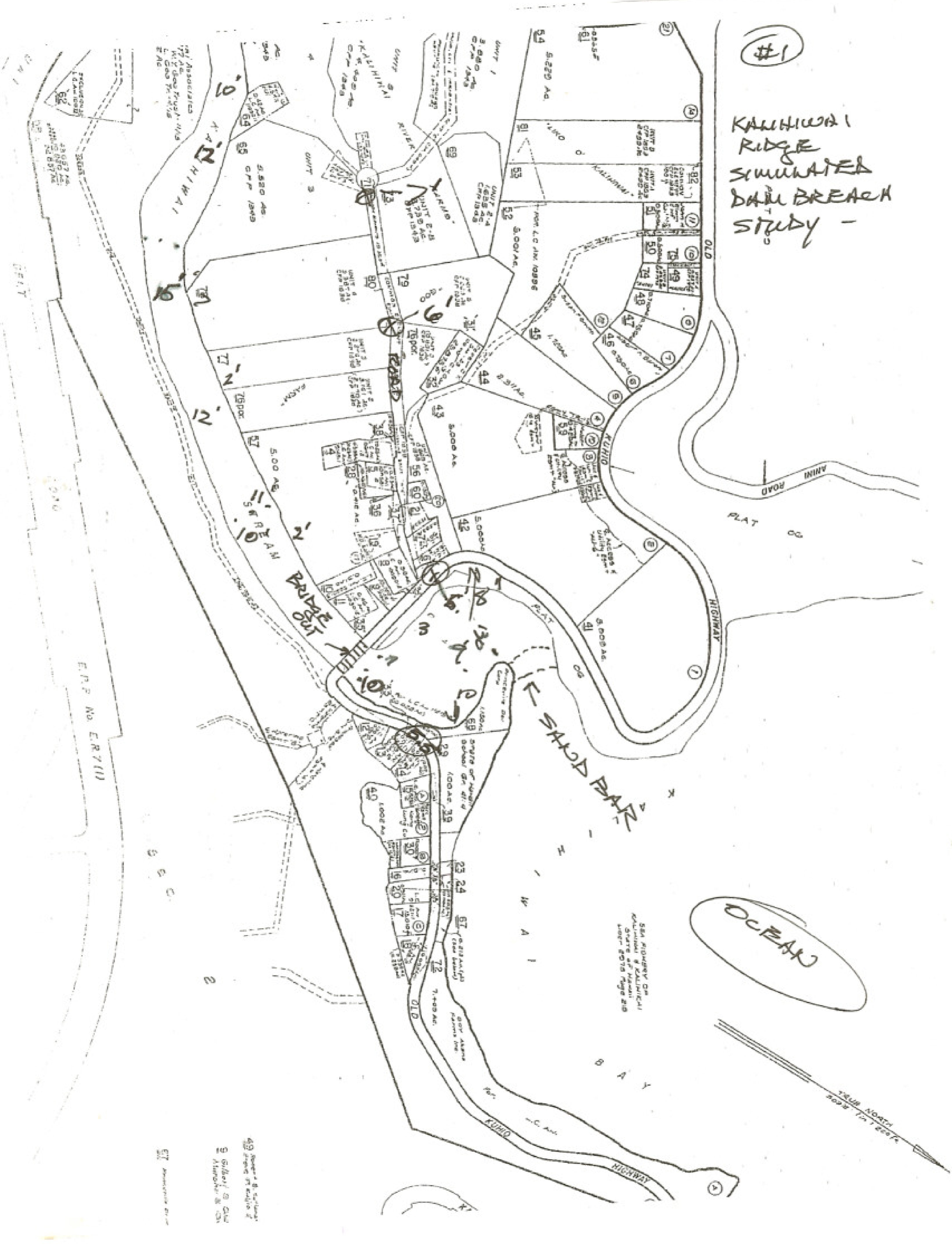
SURVEY STATION'S MAP



Appendix

Part 2 of 5: Survey of Kalihiwai Dam by Mr. Sterling C. Chisholm

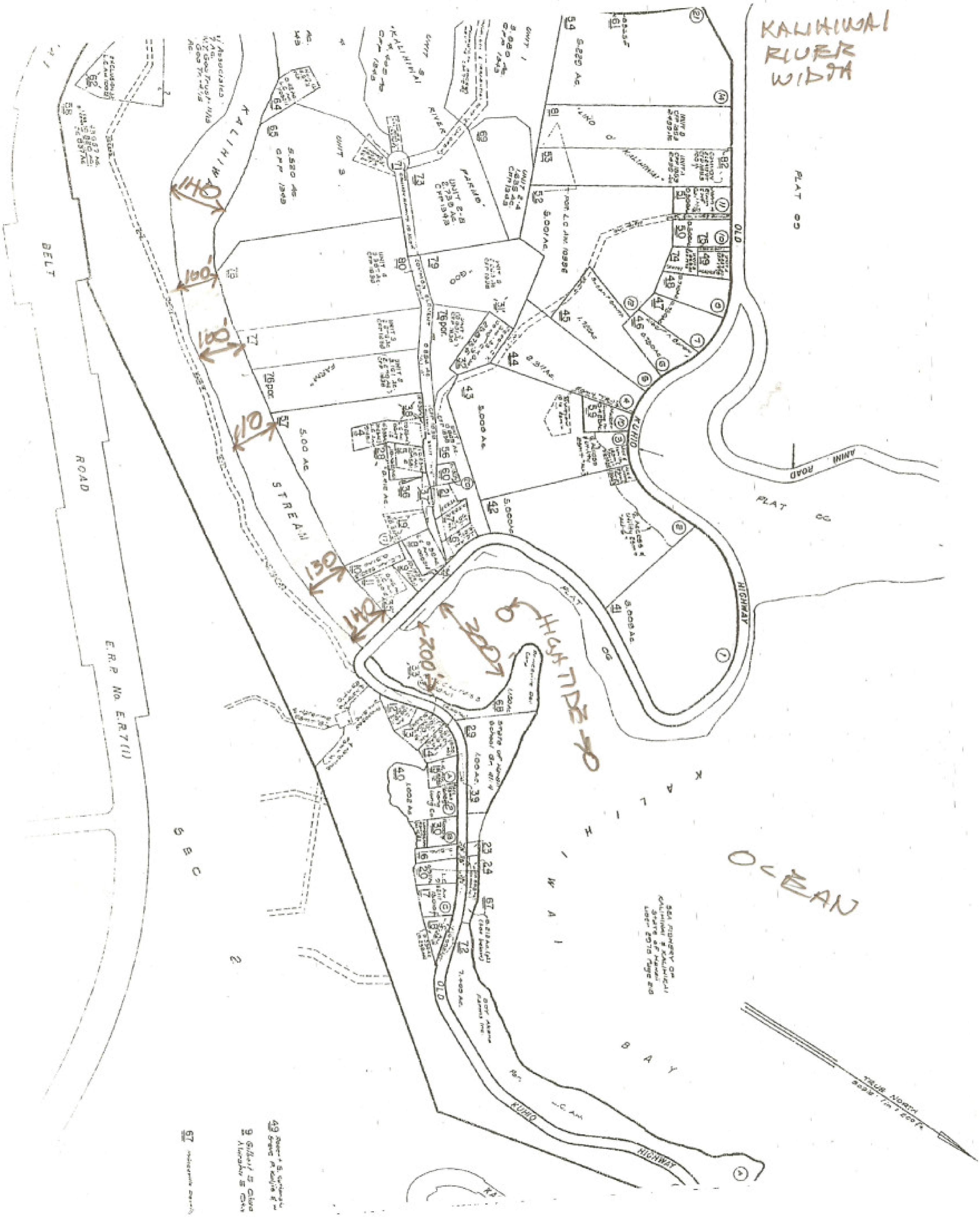

#2
RAPIDS
JUNGLE
13'
7.2'
ALL MEASUREMENTS IN FEET
KALIHIWAI RIDGE SIMULATED DAM BREACH STUDY
5.5'
7'
7'
13'
6'
KALIHIWAI BRIDGE
12'
OCEAN
Por. KALIHIWAI, HANALEI, KAUAI

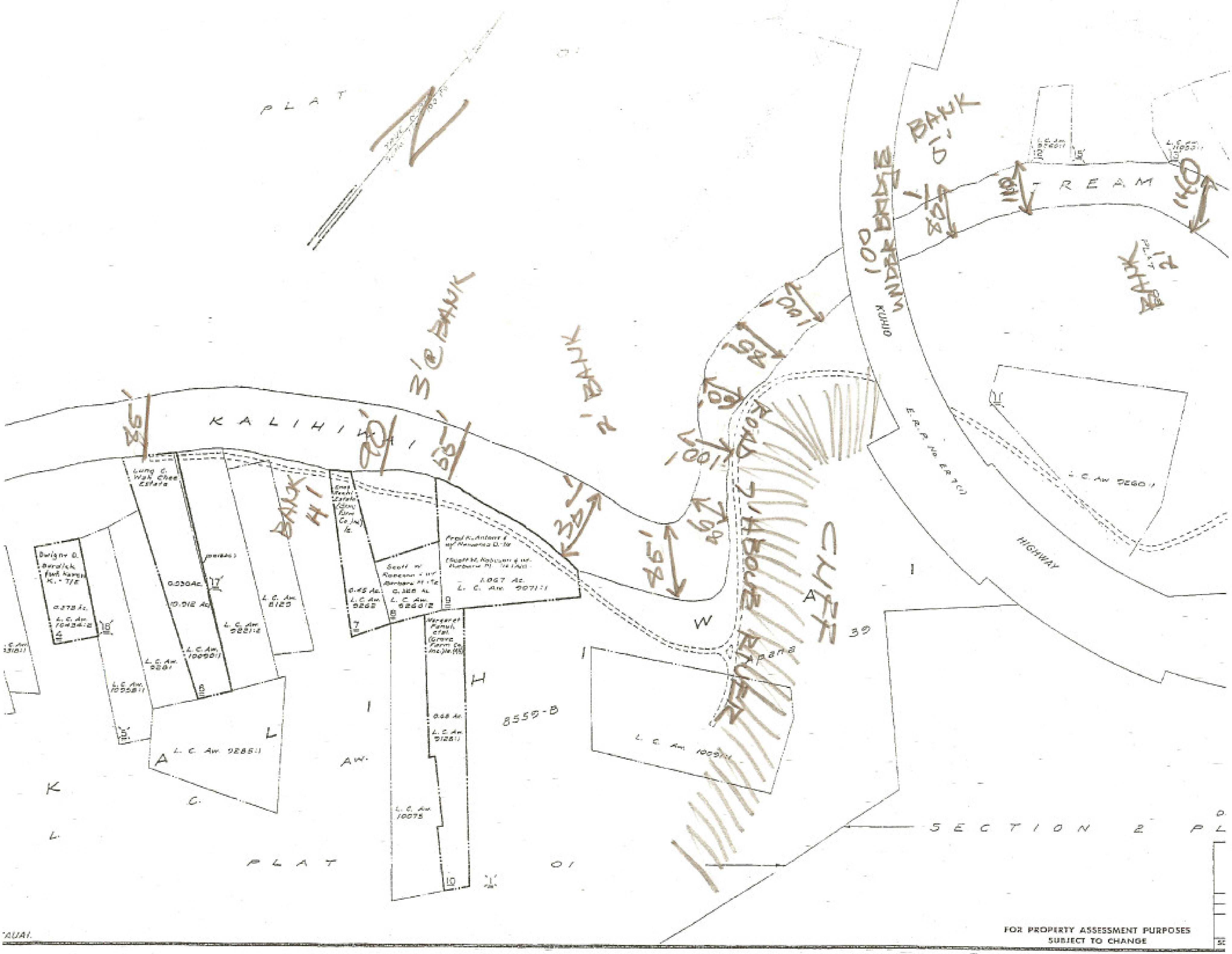


#1
KALIHIWAI RIDGE SIMULATED DAM BREACH STUDY -
ROAD
BRIDGE OUT
SAND BAR
OCEAN

Appendix

Part 4 of 5: Survey of Lagoon Widths by Mr. Sterling C. Chisholm


KALIHIWAI
RIVER
WIDTH
140'
100'
100'
110
130
140
200'
200'
HIGH TIDE
OCEAN
KALIHIWAI STREAM
KALIHIWAI BAY
BELT ROAD
KUHIO HIGHWAY
OLD KUHIO HIGHWAY
ANINI ROAD
PLAT 06
TRUE NORTH



FOR PROPERTY ASSESSMENT PURPOSES
SUBJECT TO CHANGE

Appendix

Part 5 of 5: Survey of Buildings by Mr. Sterling C. Chisholm

Memorandum

To: Brett Sanders

From: Sterling C. Chisholm

Date: October 8, 2006

Re: Survey of Buildings

Today, Sunday, October 8, 2006, beginning at 10:00 a.m. my brother, Robert Bruce Chisholm and I began the elevation shots of the various homes in the Kalihiwai Valley from the ocean area up to the Kalihiwai Bridge and according to the lettered outline on your aerial photograph sent to me via e-mail on October 2, 2006. The weather was clear with the temperature in the mid 80ies F. The Kalihiwai River was at it's standard non flood level and moving at an almost un-noticeable pace. We used the elevation of the Kalihiwai River as our zero mark and took all ground elevations from that benchmark point. All ground elevations elevations are in feet and inches above the level of the Kalihiwai River.. The elevation of the various structures are from the ground level to the joist level of the first habitable floor and not from the river level. Some of the homes in this area have been constructed to meet Federal Food Zone requirements, however there are some structures that were built prior to these flood zone requirement elevations have simply been "grandfathered" in according to our County of Kauai building guidelines. The following is an account of our measurements:

Location ref map 10/2/06	Ground Elevation	Joist Height	Comments
A	6'-4"	9'	
B	6'-0"	8'	
C	6'-0"	8'	
D	5'-10"	1' to 0'	Old structure
E	6'-2"	2'	Old Structure
F	7'-4"	9'	
G	8'-0"	8'	
H	6'-2"	2'	Old Structure
I	9'-5"	2'	Old Structure
J	6'-5"	2"-5"	House on moving blocks
K	5'-0"	3'-5"	Old house
L	5'-6"	2'	Old House
M	7'-2"	3'	Old House
N	7'-0"	2'	Old House
O	16' plus est.		Elevation Estimated No Access
P	6"-5"	3'	Old House
Q	16' plus est.	0'	Pit-bulls in yard
R	3'-0"	2"	Small Illegal Structure
S	3'-9"	0'	Boat Storage Structure
T	16'-18' plus est.	8' est.	Not in Flood Zone
U	40' plus est.	0'	" " " "
V	14" est	0'	Old Structure
W	15'	0'	Old Structure
X	15'-7"	0'	Old Structure