PRELIMINARY HYDRAULIC ASSESSMENT OF 18 HHCTCP STREAM CROSSING SITES AND A COASTAL FLOOD ZONE

Draft

Prepared for:

Parsons Brinckerhoff Inc.
1001 Bishop Street, Suite 2400
Honolulu, HI 96813
808-531-7094

Prepared by:

Northwest Hydraulic Consultants Inc.
16300 Christensen Road, Suite 350
Seattle, WA 98188
206-241-6000

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Executive Summary

Preliminary Hydraulic Assessment of 18 HHCTCP Stream Crossing Sites and a Coastal Flood Zone

The proposed Honolulu High-Capacity Transit Corridor Project (HHCTCP) is an elevated rail line that will provide public rapid transportation service for the City and County of Honolulu on the island of Oahu, Hawaii. Parsons Brinckerhoff (PB), the General Engineering Consultant (GEC) for the project, retained Lyon Associates, Inc (LAI), who in turn subcontracted with Northwest Hydraulic Consultants (NHC) to perform preliminary hydraulic evaluations of 18 stream crossing sites and a coastal flood zone located along the HHCTCP corridor alignment. These preliminary evaluations are not comprehensive. They are intended to identify and address potential floodplain hydraulic issues and floodplain regulation requirements that may require further consideration during final project design. Results from the 19 individual Site reports are provided herein for review by the PB design team, the City and County of Honolulu Department of Planning and Permitting (DPP), and the Hawaii Department of Transportation (HDOT). This report is being provided in order to obtain feed back on the preliminary results and to facilitate discussions between PB and these agencies regarding additional information they may require.

Northwest Hydraulic Consultants (NHC) recently completed detailed hydraulic investigations and reporting for the Pearl Highlands Station and Park and Ride Facilities and the Waipahu Transit Center Station Facilities (NHC, December 2009). In addition to those two stream crossing sites, the HHCTCP guideway will encounter 18 additional stream crossings and will pass through an area that has been identified and mapped by FEMA as a coastal flood hazard zone. DPP requires that floodplain development regulations are met at each crossing and within mapped flood hazard zones. Table E-1 lists the site number, stream name or floodplain area name, identifies the HHCTCP design segment, and indicates whether the stream crossing or study site is located in a mapped FEMA flood zone. Figure E-1 identifies the guideway design segment and shows where all of the 19 study sites are located. Several sites are located outside FEMA flood zones, but will need to comply with HDOT drainage requirements.

Information for each of the 18 stream crossings and the coastal flood area was obtained from FEMA, PB, US Army Corps of Engineers (USACE) or other sources in order to determine if any proposed project features would be located in the water or within mapped FEMA flood zones. Using this information, staff from NHC, PB and LAI then visited all 19 sites where access was available. Particular care was taken to understand how proposed guideway support columns or other project facilities will be positioned along each stream crossing and whether guideway or project features will be located within a mapped flood zone or interact with flood waters. Following the site inspections that were completed on September 22, 2009, different levels of analyses were conducted for each site depending on whether the individual crossing will have HHCTCP project features located in a mapped FEMA flood zone or located within the active stream channel or floodplain.

Individual summary reports were prepared for each of the 18 stream crossings and the coastal flood zone area listed in Table E-1 and shown in Figure E-1. All 19 individual summary reports are compiled in consecutive order from west to east, and are contained in this overall summary report.

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Table E-1. Stream and Floodplain Sites Evaluated Along the HHCTCP Corridor Alignment

Site	Stream	HHCTCP Design Segment	FEMA Zone	Features that May Interact with Flood Waters
1	Kaloi Drainage Channel – East Kapolei Station	B		Columns in Channel; located above 100-year water surface. Verify Column & Other Feature Locations Inside and Outside of Channel
2A	Kaloi Drainage Channel – UH West Oahu Station	B		Columns in Channel; located above 100-year water surface. Verify Column & Other Feature Locations Inside and Outside of Canal
2B	Kaloi Drainage Channel – Guideway Channel Crossing North of UH West Oahu Station	B		Three Guideway Support Columns in Channel
3	Honouliuli Stream	B	A	None
4	Hoaeae Stream	C		Columns in Median of Existing Highway
5	Panakauahi Gulch	C	AE	Columns in Gulch
6	Pearl City Stream	D		Columns in Median of Existing Highway
7	Waiau Springs	D		Columns in Median of Existing Highway
8	Waimalu Stream	D		Columns in Median of Existing Highway
9	Kalauao Springs	D		Columns in Median of Existing Highway
10	Kalauao Stream	D	AE	None
11	Aiea Stream	D	A	None
12	Halawa Stream	J		Columns in Median of Existing Highway
13	Aolele Ditch	J		Columns Along Ditch, but Outside of Channel Banks
14	Moanalua Stream	J	AE	Two Columns in FEMA Floodway
15	Kalihi Stream	E	AO	Columns and Station in FEMA AO Zone
16	Kapalama Canal Stream	E		Columns in Median of Existing Highway & Station
17	Nuuanu Stream	G		Two Columns in Channel
18	Ala Mona Coastal Zone	G	A	Multiple Features Located in Mapped Coastal A Zone

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Each report describes the site, its location and provides photos, maps and/or figures needed to support results from the preliminary analyses and reporting conducted for each site. Each report provides a brief summary of the site's 100-year regulatory flow conditions, existing FEMA flood hazard characteristics, and the hydraulic analyses that were conducted for the site. Each report briefly discusses regulatory compliance and conclusions, including mitigation proposed from each site evaluation.

Brief Summary of Findings for Each of the 19 Sites

Following are brief summaries of the key findings for each of the sites that were evaluated.

Site 1, Kaloi Drainage Channel, East Kapolei Station – This site is not located in a mapped FEMA flood zone but it is in the jurisdiction of the Hawaii Department of Transportation (HDOT). Some HHCTCP project features will be constructed adjacent to and slightly inside the Kaloi Drainage Channel; however, all of the project features will be designed to be above the 100-year design water level. Therefore, the station entrance buildings and guideway support structures will not affect the present 100-year flows or water levels. However, results from the design HEC-RAS model indicate that 100-year flow conditions in this portion of the channel are such that periodic channel maintenance after significant flow events will be required to repair channel scour caused by high velocity flows. Therefore, final project design should ensure that base elevations for the HHCTCP columns and foundation support structures are set above 100-year water levels and sufficiently away from potentially problematic scour processes.

Site 2A, Kaloi Drainage Channel, UH West Oahu Station – This site is not located in a mapped FEMA flood zone but it is in the jurisdiction of the Hawaii Department of Transportation (HDOT). Entrance Building A will straddle the drainage channel to allow passengers to enter from the western and eastern sides of the channel and Entrance Building B will be located on the west bank of the Kaloi Drainage Channel. Some project features may be located inside the channel, but all of the project features will be designed to be above the 100-year design water surface. Therefore, the station and entrance buildings and guideway support structures will not affect the present 100-year flows or water levels. However, results from the design HEC-RAS model indicate that 100-year flow conditions in this portion of the channel are such that periodic channel maintenance after significant flow events will be required to repair channel scour caused by high velocity flows. Therefore, final project design should ensure that base elevations for the HHCTCP columns and foundation support structures are set above 100-year water levels and sufficiently away from potentially problematic scour processes.

Site 2B, Kaloi Drainage Channel, Guideway Channel Crossing North of UH West Oahu Station – This site is not located in a mapped FEMA flood zone but it is in the jurisdiction of the Hawaii Department of Transportation (HDOT). Three guideway support columns are located in the active flow portion of the channel and are likely to alter local flow conditions. The design HEC-RAS model indicates that 100-year flow conditions in this portion of the channel are such that periodic channel maintenance after significant flow events will be required to repair channel scour caused by high velocity flows. Placement of the guideway support columns in the high velocity flows will likely cause a rise in water levels and contribute to local scour processes. PB and Lyon Associates designed scour countermeasures to mitigate

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for likely scour at the base of each column located in the flow. However, channel maintenance and scour repairs may still be required after large flood events. Therefore, likely needs for periodic channel maintenance and repairs at Site 2B should be addressed during final design.

Site 3, Honouliuli Stream – One of the proposed HHTCTCP elevated guideway columns is located within a mapped FEMA Zone A. Examination demonstrated that the column is actually on high ground, above flood elevations, and will not impact 100-year flood levels.

Site 4, Hoaeae Stream – No mapped FEMA Flood Hazard exists for this site. HHCTCP columns are located on highway median, outside channel boundaries. We do not anticipate any impact on flood levels at this site.

Site 5, Panakauahi Gulch – Columns are located within mapped FEMA floodplain, but outside of the FEMA floodway. Site hydraulic conditions suggest that the columns will have insignificant impact on 100-year flood levels.

Site 6, Pearl City Stream – No mapped FEMA Flood Hazard exists for this site. HHCTCP columns are located on highway median, outside channel boundaries. We do not anticipate any impact on flood levels at this site.

Site 7, Waiau Springs – No mapped FEMA Flood Hazard exists for this site. HHCTCP columns are located on highway median, outside channel boundaries. We do not anticipate any impact on flood levels at this site.

Site 8, Waimalu Stream – No mapped FEMA Flood Hazard exists for this site. HHCTCP columns are located on highway median, outside channel boundaries. We do not anticipate any impact on flood levels at this site.

Site 9, Kalauao Springs – No mapped FEMA Flood Hazard exists for this site. HHCTCP columns are located on highway median, outside channel boundaries. We do not anticipate any impact on flood levels at this site.

Site 10, Kalauao Stream – One HHCTCP column is located within the mapped FEMA Floodway. Examination of FEMA Base Flood Elevations, Flood Profiles, and new topographic information demonstrate that the column is outside channel boundaries and above mapped flood elevations, therefore we do not anticipate any impact on flood levels at this site.

Site 11, Aiea Stream – Columns are located outside of the mapped FEMA Floodplain. We do not anticipate any impact on flood levels at this site.

Site 12, Halawa Stream – No mapped FEMA Flood Hazard exists for this site. HHCTCP columns are located on highway median, outside channel boundaries. We do not anticipate any impact on flood levels at this site.

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Site 13, Aolele Ditch – No mapped FEMA Flood Hazard exists for this site. HHCTCP columns are located along Aolele Ditch outside of the channel, above the top of the channel banks. We do not anticipate any impacts on flood levels at this site.

Site 14, Moanalua Stream – Several guideway support columns are located in mapped AE zones and there are two guideway support columns located in a regulatory floodway at this site. NHC analyzed the potential water level rise caused by the two columns located in the floodway using the NHC-revised HEC-RAS model obtained from the Honolulu District Corps of Engineers. This model is the closest model to the effective study model that is presently available. Model results indicate a modest rise of 0.08-ft in the floodway water level immediately upstream of the two proposed columns. Mitigation for this impact to the floodway or modification of the column layout may be required to comply with FEMA's and DPP's "no rise" requirements in floodways. Use of the NHC-revised HEC-RAS model to assess the effects of proposed guideway support columns in a regulatory floodway at Moanalua Stream will need to be approved by FEMA and DPP. Possible mitigation alternatives have been proposed, but they have not been assessed at this time.

Site 15, Kalihi Stream – Multiple HHCTCP support columns and structures are located within a mapped FEMA Floodplain (Zones AO and AE). These features will need to be adequately flood proofed to meet regulatory requirements.

Site 16, Kapalama Canal Stream – No mapped FEMA Flood Hazard exists for this site. HHCTCP columns are located on highway median, outside channel boundaries. We do not anticipate any impact on flood levels at this site. The adjacent station is located in low lying area that could experience shallow flooding should water exit the canal, and we recommend that structures be adequately flood proofed.

Site 17, Nuuanu Stream – Two HHCTCP columns are located within the Nuuanu Stream channel and would produce a negligible impact on flood levels. Modifying one of the column locations has been proposed to further reduce any impact of the project, though this is not in a mapped Flood Hazard District.

Site 18, Ala Moana Coastal Flood Zone – Approximately 4,800 linear feet of elevated guideway and two transit stations are located in a FEMA "A" zone. Preliminary analyses show that both station buildings are above the estimated "A" zone water level, so they will not affect the estimated 100-year water level. Three guideway support columns are located in shallow flooding areas. Water depths at the base of these columns are only 0.01 to 0.26 feet deep. Therefore, the displaced floodplain storage volume caused by the three columns is only 10.44 cubic feet which can be easily compensated for locally through minor project feature modifications or grading during final design. These results are based on currently publish BFEs and information presented in FEMA's September 2004 FIS. If FEMA adopts new mapping for the Ala Moana Coastal Flood Zone in the future, water levels, depths of inundation and volume compensation estimates reported herein may need to be revised based on new information and mapping made available at that time.

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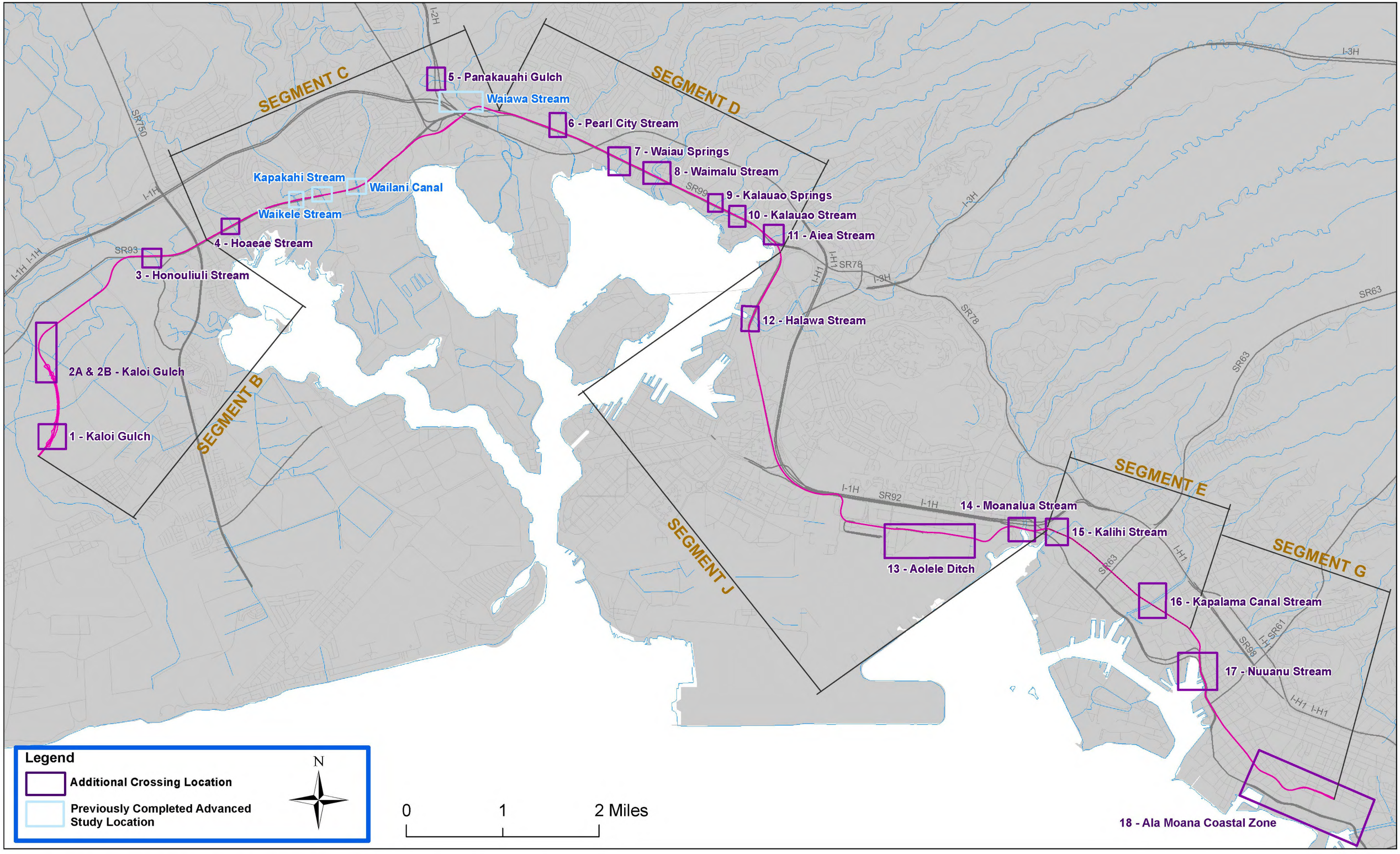

Figure E.1 Honolulu High Capacity Transit Corridor Project - Rail Crossing Locations

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1 - Kaloi Drainage Channel - East Kapolei Station

Site Description

East Kapolei Station is located at corridor station 398+00 near the south western-most end of Project Corridor Segment B (Figure 1.1). The Entrance Building for the East Kapolei Station will be located on the west bank of the newly constructed Kaloi Drainage Channel (Figure 1.2). Portions of the entrance building will be constructed at grade along the west bank of the earthen drainage channel. Also located along the west bank of the channel will be several 6 - foot square columns that support the elevated guideway and passenger loading platforms. Present design drawings show that portions of the station building foundation and several 30-inch diameter station building support columns will partially protrude into the channel, but they will be located above the 100-year design water surface profile. Representatives from NHC, PB and LAI briefly visited a portion of the Kaloi Drainage Channel near the proposed location of the East Kapolei Station site on September 22, 2009. They observed active construction activities related to Kualakai Parkway (North-South Road). Photos 1.1 to 1.4 show portions of Kaloi Drainage Channel near the site.

Hydrologic Analysis and FEMA Study

The Kaloi Drainage Channel was recently constructed and is in the jurisdiction of the Hawaii Department of Transportation (HDOT). The land areas and floodplains adjacent to the Kaloi Drainage Channel have not been analyzed by FEMA as part of the effective Flood Insurance Study (FIS) for Honolulu, HI. Therefore, no FEMA FIS study reports, floodplain maps or information are available for Site 1 at the East Kapolei Station. R.M. Towill Corporation provided the PB Study Team with their January 15, 2007 HEC-RAS model data files that were used to design the Kaloi Drainage Channel[1] (referred to herein as the "design HEC-RAS model"). According to the North-South Road Drainage Report[2] the channel was designed for ultimate built-out conditions and flows that can occur during a 100-year flood event. The 100-year design flows specified in the design HEC-RAS model are 5,800 cfs at the upstream end of the channel (which is located approximately 960 feet upstream from the proposed location for the Upper

[1] Email from PB containing R.M. Towill Corporation's design HEC-RAS model files for Kaloi Drainage Channel, January 25, 2007.

[2] HDOT/R.M. Towill Corporation, (July 2007), "Drainage Report for North-South Road Kapolei Parkway to Interstate Route H-1 Ewa, Oahu, Hawaii," prepared for Department of Transportation, Highways Division, Kapolei Hawaii, prepared by R.M. Towill, July 2007, 21 pages plus appendices.

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Connector Road) and increases gradually to 7,400 cfs near the downstream end of the channel at the stormwater retention basin. The 100-year design flow at Site 1 – East Kapolei Station is 6,200 cfs.

Hydraulic Analysis

The Kaloi Drainage Channel is a steep (slope ~ 0.012) earthen channel (approximately 150-foot top width, 60-foot bottom width, and 2:1 side slopes) with 2- to 6-foot-high, rock-covered drop structures (Photo 1.4) spaced approximately 80 to 300 feet apart to reduce velocities and provide a milder water surface slope. NHC used the design HEC-RAS model to generate and plot the 100-year design water surface profile between corridor stations 396+00 to 399+50 at Site 1. Figures 1.3A and 1.3B show the plan and profile for the channel adjacent to the East Kapolei Station together with the 100-year HHCTCP West Oahu Farrington Highway Design Build Contract Design Water Surface Extent and Profile[3], respectively (referred to as the WOFH water surface extent and profile in Figure 1.3). Figures 1.3A and 1.3B also show the approximate location and dimensions of the existing bridge culvert for the East-West Road (Photos 1.2 and 1.3). The upstream face of the bridge culvert is located approximately 65 to 70 feet downstream from an existing rock-covered drop structure and only 20 feet downstream from the southern-most end of the East Kapolei Station entrance building. Figures 1.3A and 1.3B show the locations of several of the 6-foot square guideway and 30-inch diameter entrance building support columns.

Figures 1.3A and 1.3B also show the 100-year HHCTCP WOFH Design Water Surface (**blue** line) and the design HEC-RAS 100-year water surface profile (**black** line) along the channel in the vicinity of the East Kapolei Station. Top-of-channel bank profiles were developed from contour data prepared by PB from Segment B LiDAR, which includes the Kaloi Drainage Channel. Figures 1.3A and 1.3B show the east and west top-of-channel bank profiles along the Kaloi Drainage Channel as the **green** and **red** line profiles, respectively. Comparison of the **red** or **green** line top-of-bank profiles with the **blue** or **black** line 100-yr water surface profiles provides an estimate of the in-channel freeboard during the 100-year event. Similarly, comparison of the **blue** or **black** line water surface profiles with the channel invert (bottom) profile provides an estimate of flow depth during the 100-yr flood event.

Figures 1.4A and 1.4B show cross section views developed from the design drawings for project features that protrude into the channel along with the HHCTCP WOFH Design Water Surface (**blue** line). Figures 1.3 and 1.4 show that all of the HHCTCP support columns and support structures are above the 100-year (WOFH) design water level and should not interfere with flood flows. This conclusion should be verified during final design.

Design 100-year flow conditions for the Kaloi Drainage Channel are listed in Appendix F in the North-South Road Drainage Report[4]. The 100-year flows in this portion of the channel result in average

[3] HDOT/R.M. Towill, 2007

[4] HDOT/R.M. Towill, 2007, Appendix F

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channel velocities of 7.25 to 13 fps with super-critical flow conditions and average velocities of approximately 10 fps to 20 fps near the drop structures. These flow conditions are likely to generate significant turbulence, bed shear stress and standing waves, especially in the vicinity of the drop structure and near the East-West Road Culvert Bridge located just downstream from the East Kapolei Station. High-energy flows with these characteristics may cause localized channel bed and bank scour and require periodic channel maintenance to repair sour-related damage. Therefore, base elevations for the HHCTCP columns and foundation support structures in final design should be set above 100-year water levels and be sufficiently protected from potentially problematic scour processes.

Regulatory Compliance and Conclusions

The Kaloi Drainage Channel is not located in a mapped FEMA flood hazard zone. The Hawaii Department of Transportation (HDOT) is responsible for operating and maintaining the Kaloi Drainage Channel. Therefore, channel hydraulic conditions are subject to HDOT design criteria. According to available information (100-year design water levels obtained from the design HEC-RAS model and PB's HHCTCP design drawings) the East Kapolei Station and guideway support structures do not intersect the HHCTCP WOFH 100-year design water level and will have no impact on the 100-year design flows or water levels in the Kaloi Drainage Channel. However, the 100-year design water level is close to the base elevation of the support column located at station 398+64.83 shown in Figure 1.3B. The vertical clearance distance at this column and all other HHCTCP features should be checked during final design.

100-year flow velocities and bottom shear stresses are quite high in the vicinity of the East-West Road Bridge and East Kapolei Station[5], which makes the channel vulnerable to bed, and bank erosion during high flows. Therefore, in order to safeguard HHCTCP features from the possible effects of channel erosion, NHC recommends that no HHCTCP project features be constructed in the active channel. We also recommend that flow conditions and channel erosion and scour potential in the vicinity of the Lower Connector Road Bridge and East Kapolei Station be carefully evaluated during final design so that the base elevations for the HHCTCP columns and foundation support structures have sufficient vertical clearance above anticipated 100-year water levels and be sufficiently protected from potentially problematic scour processes.

[5] HDOT/R.M. Towill, 2007, Appendix F

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References

Email from Paul Fujii at PB containing R.M. Towill Corporation's design HEC-RAS model files for Kaloi Drainage Channel; email dated January 25, 2007.

HDOT/R.M. Towill Corporation, (July 2007), "Drainage Report for North-South Road Kapolei Parkway to Interstate Route H-1 Ewa, Oahu, Hawaii," prepared for Department of Transportation, Highways Division, Kapolei Hawaii, prepared by R.M. Towill, July 2007, 21 pages plus appendices.

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Photos

Photo 1.1: Intersection of Kualakai Parkway (Sta. 39+17) with East-West Road; View to north

Photo 1.2: View of downstream side of East-West Road Culvert Bridge; View to northeast

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Photo 1.3: Close-up of downstream outlet from East-West Road Culvert Bridge

Photo 1.4: Close-up of downstream drop structure (Sta. 36+00 +/-); View to southeast

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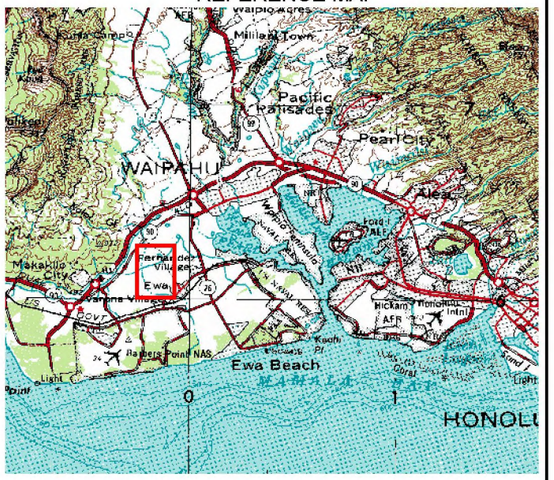
Figure 1.1 - Segment B
Site 1 Location Map

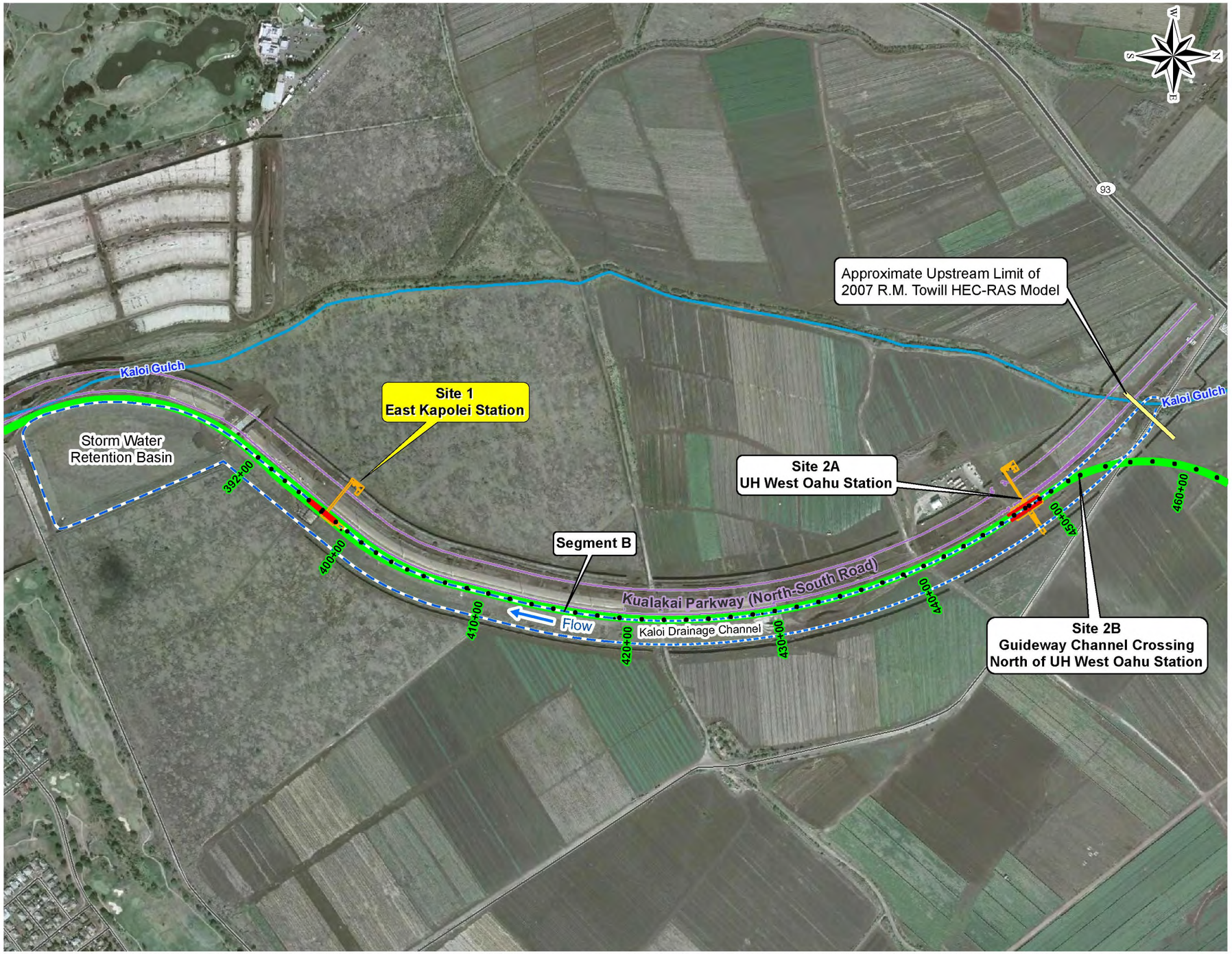
Scale - 1:8,400 1 inch = 700 feet

0 350 700 1,400 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50597	April 2010

REFERENCE MAP



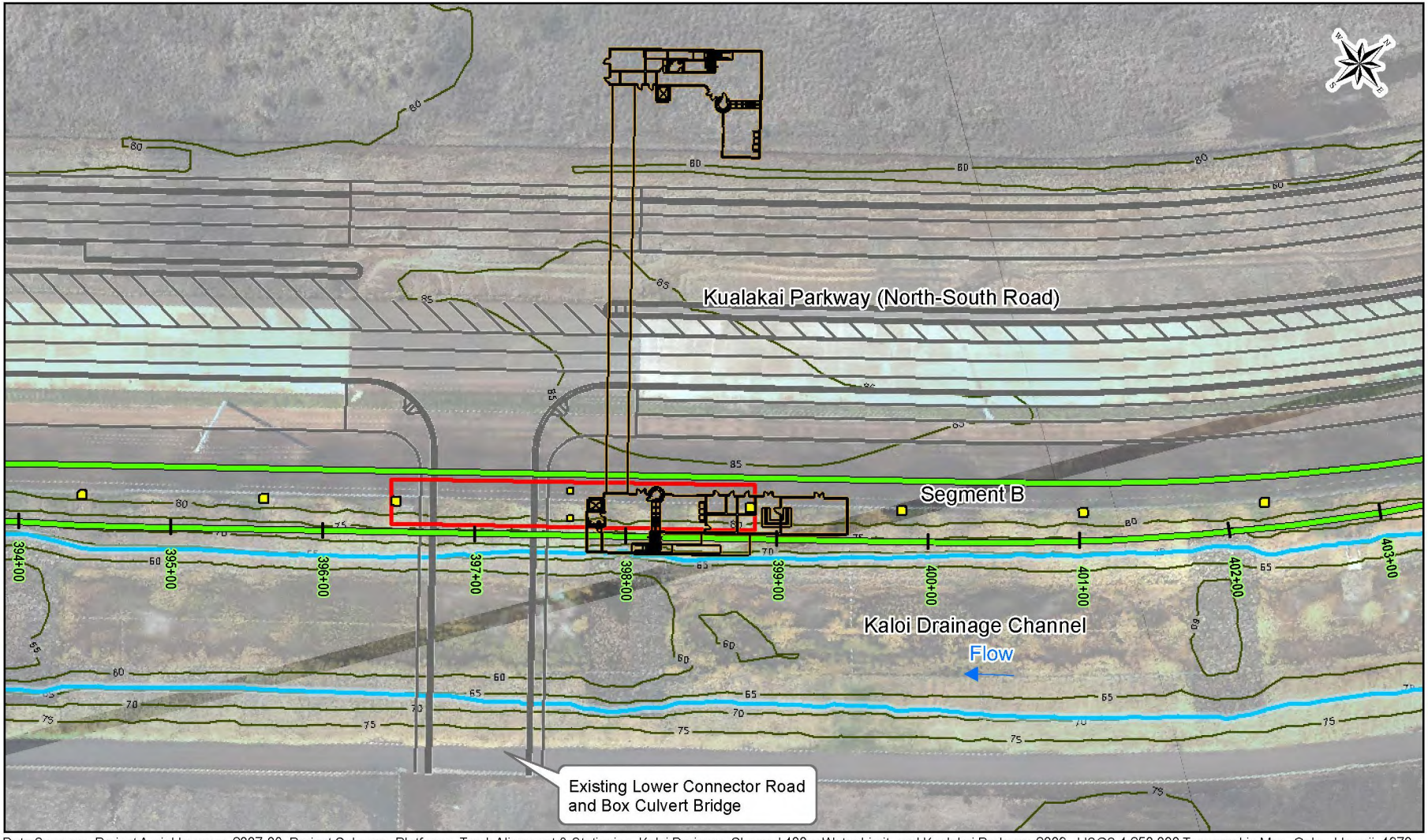


LEGEND

- Proposed Guideway Support Columns
- Proposed Platforms
- Proposed Track Alignments (with stationing)
- Kaloi Drainage Channel
- Roads
- Streams

Data Sources: Google Earth Imagery, Dec 2008. ESRI Roads and Streams, 2008. Project Columns, Platforms, Track Alignment, Channel, and Kualakai Parkway, 2009. USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

Figure 1.1 Segment B - Site 1 Location Map

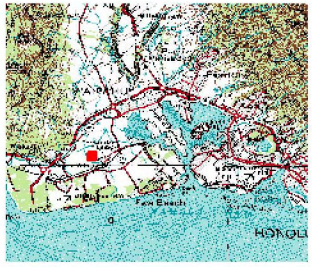


Data Sources: Project Aerial Imagery, 2007-08. Project Columns, Platforms, Track Alignment & Stationing, Kaloi Drainage Channel 100yr Water Limit, and Kualakai Parkway, 2009. USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

Legend

- East Kapolei Station Architectural Gound Level Plan
- Station Platform
- Guideway Support Columns
- 5ft Contours
- Track Alignment
- Kaloi Drainage Channel 100yr Water Limits
- West Bank Track Alignment Stationing

Oahu Reference Map



Honolulu High Capacity Transit Corridor

Site1 Kaloi Drainage Channel East Kapolei Station

Scale -1:1,200

50 25 0 50 100 Feet

coord. syst.: HI State Plane 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50597	Apr-2010

Figure 1.2

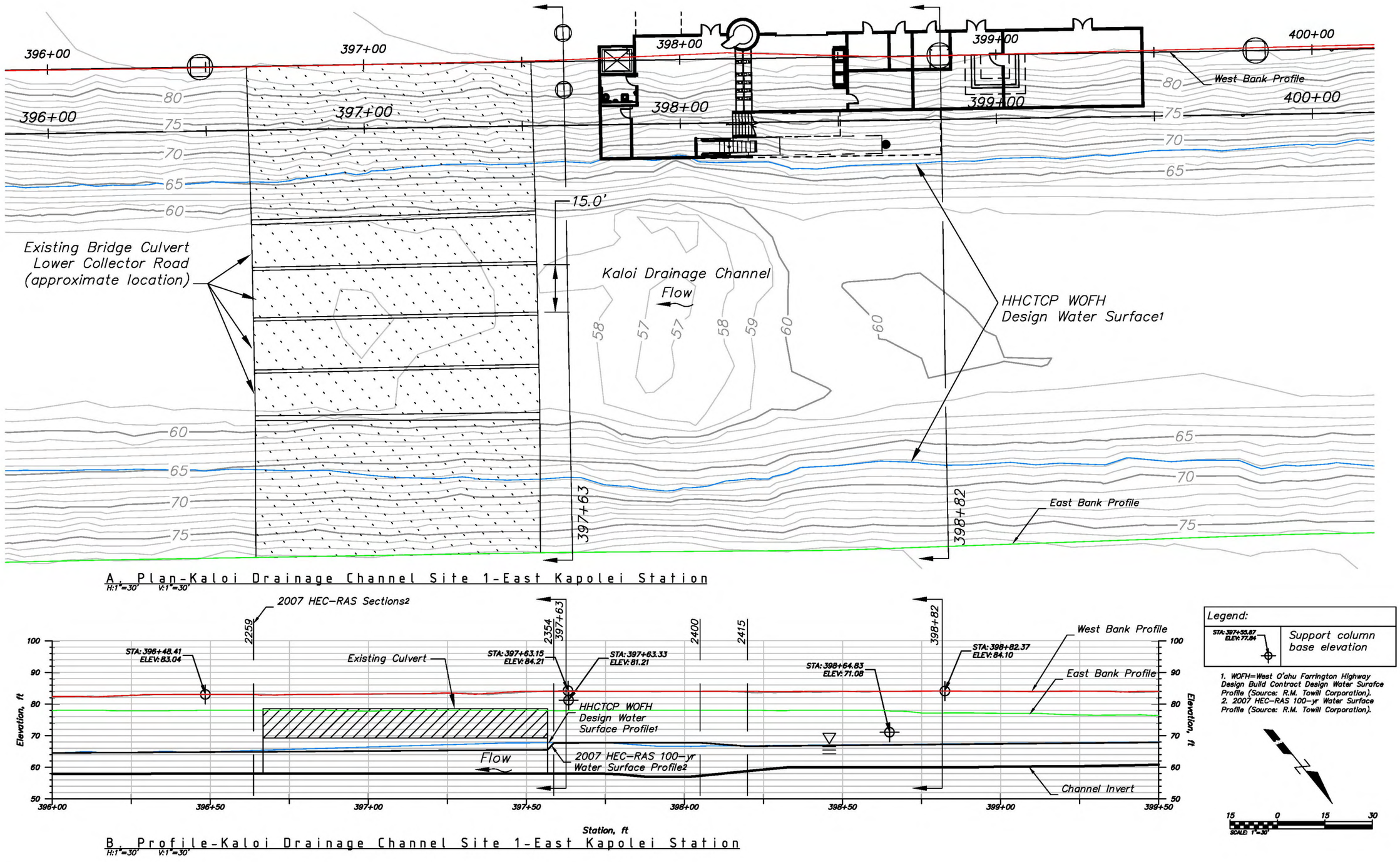
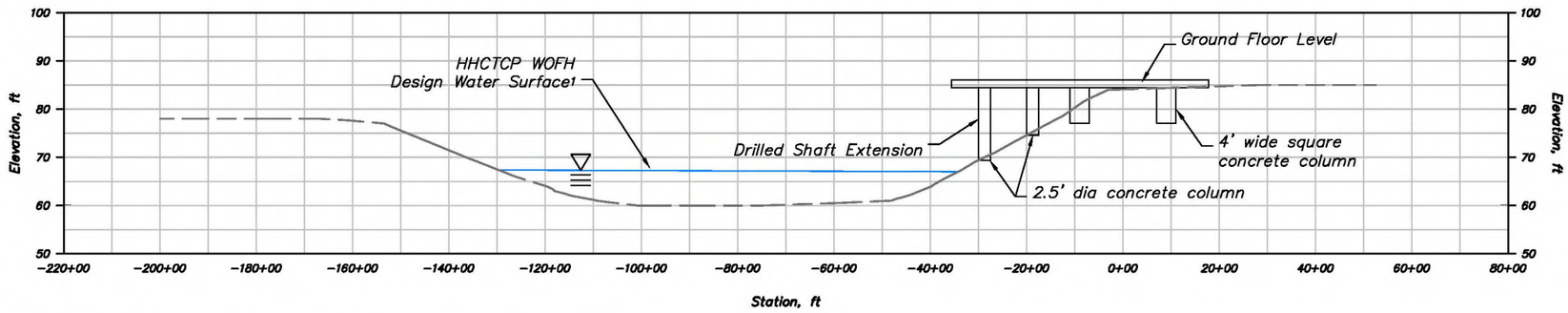
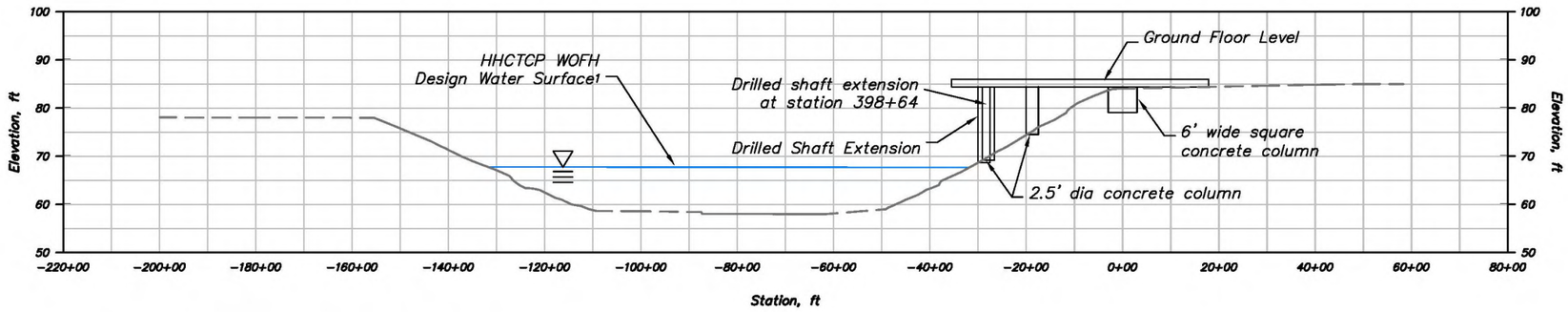


Figure 1.3 Kaloi Drainage Channel Site 1-East Kapolei Station Plan and Profile



A. SECTION 397+63 (looking downstream)
H:1"=30' V:1"=30'



B. SECTION 398+82 (looking downstream)
H:1"=30' V:1"=30'

1. WOFH=West O'ahu Farrington Highway Design Build Contract Design Water Surafce Profile (Source: R.M. Towill Corporation).

Figure 1.4 Kaloi Drainage Channel Site 1-East Kapolei Station Sections

2A – Kaloi Drainage Channel - UH West Oahu Station

Site Description

UH West Oahu Station is located at corridor station 448+50 in Corridor Segment B, north of the East Kapolei Station (Figure 2A.1). Entrance Building B of the UH West Oahu Station will be located on the west bank of the newly constructed earthen Kaloi Drainage Channel (Figure 2A.2). However, Entrance Building A will straddle the drainage channel from west to east to allow passengers to enter and exit from the western and eastern sides of the channel (Figure 2A.2). Also located along the west bank of the channel will be several 6 - foot square columns (yellow dots in Figure 2A.2) that support the elevated guideway and passenger loading platforms. Vertical retaining walls and 30-inch diameter station building support columns may be required along the west and east channel banks. The specific size and location of station and entrance building support structures will be identified during final design, however, all of these project features will be located sufficiently above the 100-year design water level. Representatives from NHC, PB and LAI briefly visited a portion of the Kaloi Drainage Channel (Kaloi channel) near the proposed location of the UH West Oahu Station site on September 22, 2009. They observed active construction activities related to Kualakai Parkway (North-South Road), however, construction of the UH West Oahu Station facilities had not been initiated at that time.

Hydrologic Analysis and FEMA Study

The Kaloi Drainage Channel was recently constructed and is in the jurisdiction of the Hawaii Department of Transportation (HDOT). The land areas and floodplains adjacent to the Kaloi Drainage Channel have not been analyzed or mapped by FEMA as part of the effective Flood Insurance Study (FIS) for Honolulu, HI. Therefore, no FEMA FIS reports, floodplain maps or information are available for Site 2A at the UH West Oahu Station. R.M. Towill Corporation provided the PB Study Team with their January 15, 2007 HEC-RAS model data files that were used to design the Kaloi Drainage Channel[1] (referred to herein as the "design HEC-RAS model"). According to the North-South Road Drainage Report,[2] the channel was designed for ultimate built-out conditions and flows that can occur during a 100-year flood event. The 100-year design flows specified in the design HEC-RAS model are 5,800 cfs at the upstream end of the channel (which is located approximately 960 feet upstream from the proposed location for the Upper

[1] Email from PB containing R.M. Towill Corporation's design HEC-RAS model files for Kaloi Drainage Channel, January 25, 2010.

[2] HDOT/R.M. Towill Corporation, (July 2007), "Drainage Report for North-South Road Kapolei Parkway to Interstate Route H-1 Ewa, Oahu, Hawaii," prepared for Department of Transportation, Highways Division, Kapolei Hawaii, prepared by R.M. Towill, July 2007, 21 pages plus appendices.

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Connector Road) and increases gradually to 7,400 cfs near the downstream end of the channel at the stormwater retention basin. According to the design HEC-RAS model, the 100-year design flow at Site 2A at UH West Oahu Station is 5,900 cfs.

Hydraulic Analysis

The Kaloi Drainage Channel is a steep (slope ~ 0.012) earthen channel (approximately 150-foot top width, 60-foot bottom width, and 2:1 side slopes) with 2- to 6-foot-high, rock-covered drop structures spaced approximately 80 to 300 feet apart to reduce velocities and provide a milder water surface slope. NHC used the design HEC-RAS model to generate and plot the 100-year design water surface profile between corridor stations 446+00 to 450+26 at Site 2A. Figures 2A.3A and 2A.3B show the plan and profile adjacent to the UH West Oahu Entrance Building together with 100-year HHCTCP West Oahu Farrington Highway Design Build Contract Design Water Surface Extent and Profile[3], respectively (referred to as the WOFH water surface extent and profile in Figure 2A.3). Figures 2A.3A and 2A.3B also show the approximate location and dimensions of the proposed bridge culvert for the Upper Connector Road that is included in the design HEC-RAS model and discussed in the North-South Road Drainage Report.[4] However, nothing exists at this location today. It is possible that the location and design of Upper Connector Road and box culvert will change in the future, which would affect water levels near Entrance Building A. Therefore, the location and possible effects of the proposed Upper Connector Road and box culvert should be addressed during final design of the UH West Oahu Entrance Building A.

Figure 2A.2 shows that Entrance Building A for the UH West Oahu Station will straddle the drainage channel. Passenger loading platforms will be located on the west bank and the southernmost end of the platforms will be elevated above the proposed Upper Connector Road near its intersection with the Kualakai Parkway (North-South Road). Figure 2A.3A shows the locations of several of the 6-foot square guideway support columns to be located along the west bank of the drainage channel. The cross section views in Figures 2A.4A, 2A.4B and 2A.4C show that all of the HHCTCP support columns and support structures are above the 100-year (WOFH) design water level and will not interfere with flood flows.

Figures 2A.3A and 2A.3B also show the 100-year HHCTCP WOFH Design Water Surface (**blue** line) and the 2007 design HEC-RAS 100-year water surface profile (**black** line) along the channel in the vicinity of the UH West Oahu Station. Top-of-channel bank profiles were developed from contour data prepared by PB from Segment B LiDAR, which includes the Kaloi Drainage Channel. Figures 2A.3A and 2A.3B show the east and west top-of-channel bank profiles along the Kaloi Drainage Channel as the **green** and **red** line profiles, respectively. Comparison of the **red** or **green** line top-of-bank profiles with the **blue** or **black** line 100-yr water surface profiles provides an estimate of the in-channel freeboard during the 100-year event. Similarly, comparison of the **blue** or **black** line water surface profiles with the channel invert (bottom) profile provides an estimate of flow depth during the 100-yr flood event.

[3] HDOT/R.M. Towill, 2007.

[4] HDOT/R.M. Towill, 2007.

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Figures 2A.4A, 2A.4B and 2A.4C show cross section views of the channel, ground floor elevations for the entrance buildings and the HHCTCP WOFH Design Water Surface (**blue** line) adjacent to the entrance buildings. Note that in the immediate vicinity of Entrance Building A, the 100-year water level is approximately three feet below the top of bank elevation in that area (Figures 2A.3B and 2A.4A).

Design 100-year flow conditions for the Kaloi Drainage Channel are listed in Appendix F in the North-South Road Drainage Report[5]. The 100-year flows in this portion of the channel result in average channel velocities of approximately 6 to 10 fps with super-critical flow conditions and average flow velocities of approximately 10 fps to 20 fps near the drop structures. These flow conditions are likely to generate significant turbulence, bed shear stress and standing waves, especially in the vicinity of the drop structures and the proposed Upper Connector Road Culvert Bridge. High-energy flows with these characteristics may cause channel bed and bank scour and require periodic maintenance to repair scour-related damage. Therefore, base elevations for the HHCTCP columns and foundation support structures in final design should be set above 100-year water levels and be sufficiently protected from potentially problematic scour processes.

Regulatory Compliance and Conclusions

The Kaloi Drainage Channel is not located in a mapped FEMA flood hazard zone. The Hawaii Department of Transportation (HDOT) is responsible for operating and maintaining the Kaloi Drainage Channel. Therefore, hydraulic conditions must comply with HDOT design criteria. According to available information (100-year design water levels obtained from the design HEC-RAS model and PB's HHCTCP design drawings) the UH West Oahu Station and guideway support structures do not intersect the HHCTCP WOFH 100-year design water level and will have no impact on the 100-year flows or design water levels in the Kaloi Drainage Channel. However, the 100-year design water level is close to ground floor elevations at Entrance Building A (Figures 2A.3 and 2A.4). This vertical clearance distance should be checked during final design.

The proposed Upper Connector Road and box culvert included in the design HEC-RAS model do not exist at this time. It is possible that the location and the design of these important features will change in the future, which would affect presently defined flow conditions and water levels near Entrance Building A. Therefore, the location and possible effects of the Upper Connector Road and box culvert should be addressed during final design of the UH West Oahu Entrance Building A.

High-energy flows during flood events are likely to cause localized channel bed and bank scour and require periodic channel maintenance to repair scour-related damage. Therefore, in order to safeguard HHCTCP features from the possible effects of channel erosion, NHC recommends that no HHCTCP project features be constructed in the active channel at Site 2A. Also, flow conditions and channel erosion and scour potential in the vicinity of the Upper Connector Road Culvert Bridge and the UH West

[5] HDOT/R.M. Towill, 2007, Appendix F

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Oahu Station should be carefully evaluated during final project design. Base elevations for the HHCTCP columns and foundation support structures should be set above 100-year water levels and be sufficiently protected from potentially problematic scour processes.

References

Email from Paul Fujii at PB containing R.M. Towill Corporation's design HEC-RAS model files for Kaloi Drainage Channel; email dated January 25, 2010.

HDOT/R.M. Towill Corporation, (July 2007), "Drainage Report for North-South Road Kapolei Parkway to Interstate Route H-1 Ewa, Oahu, Hawaii," prepared for Department of Transportation, Highways Division, Kapolei Hawaii, prepared by R.M. Towill, July 2007, 21 pages plus appendices.

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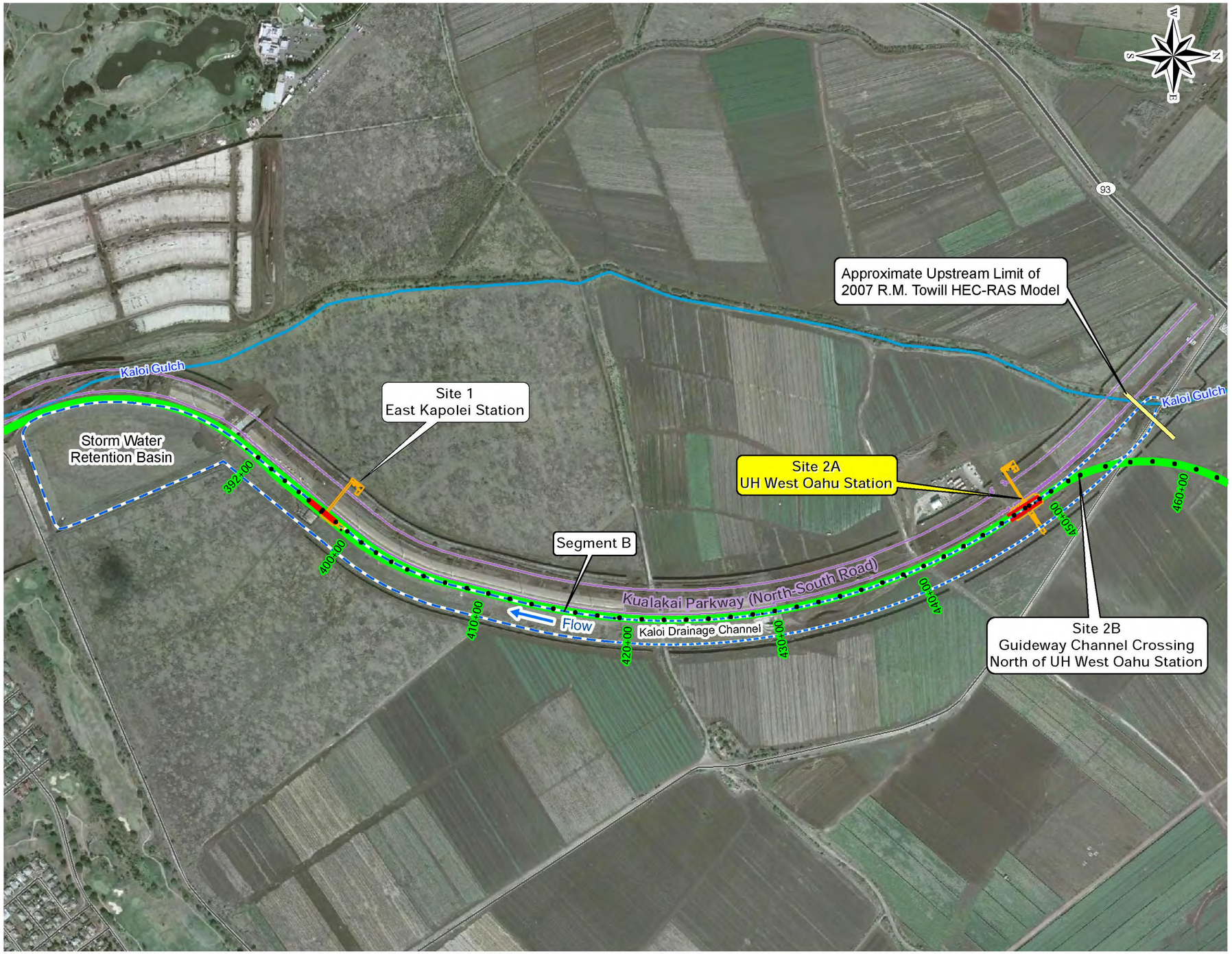
Figure 2A.1 - Segment B
Site 2A Location Map

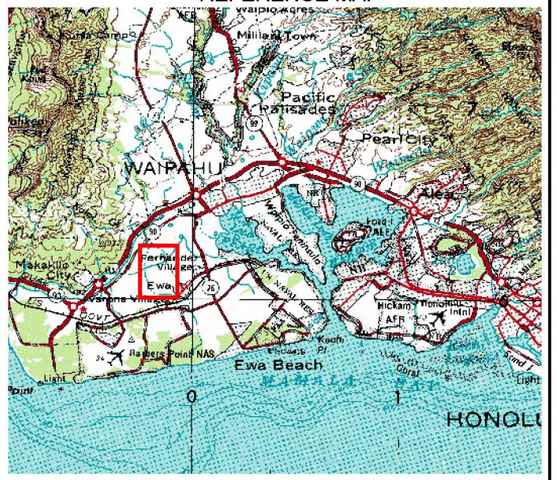
Scale - 1:8,400 1 inch = 700 feet

0 350 700 1,400 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50597	April 2010



REFERENCE MAP

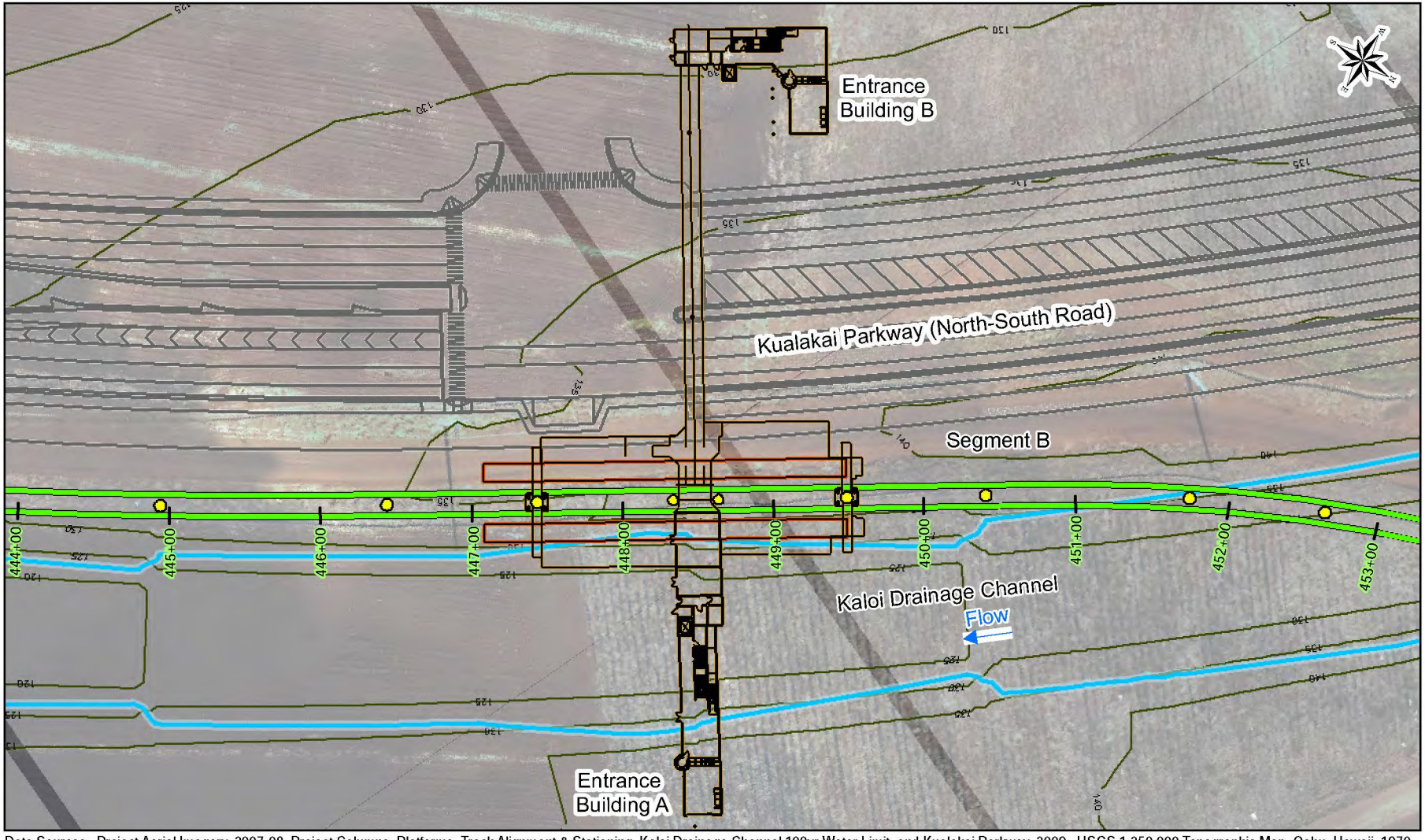


LEGEND

- Proposed Guideway Support Columns
- Proposed Platforms
- Proposed Track Alignments (with stationing)
- Kaloi Drainage Channel
- Roads
- Streams

Data Sources: Google Earth Imagery, Dec 2008. ESRI Roads and Streams, 2008. Project Columns, Platforms, Track Alignment, Channel, and Kualakai Parkway, 2009. USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

Figure 2A.1 Segment B - Site 2A Location Map

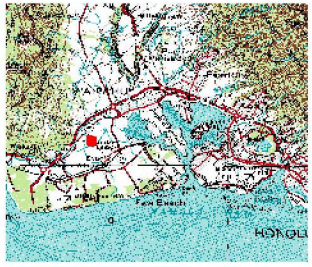


Data Sources: Project Aerial Imagery, 2007-08. Project Columns, Platforms, Track Alignment & Stationing, Kaloi Drainage Channel 100yr Water Limit, and Kualakai Parkway, 2009. USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

Legend

- UH West Oahu Station Architectural Gound Level Plan
- Station Platform
- Guideway Support Columns
- 5ft Contours
- Kaloi Drainage Channel 100yr Water Limits
- Track Alignment
- West Bank Track Alignment Stationing

Oahu Reference Map



Honolulu High Capacity Transit Corridor

Site2A Kaloi Drainage Channel UH West Oahu Station

Scale -1:1,200

50 25 0 50 100 Feet

coord. syst.: HI State Plane 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50597	Apr-2010

Figure 2A.2

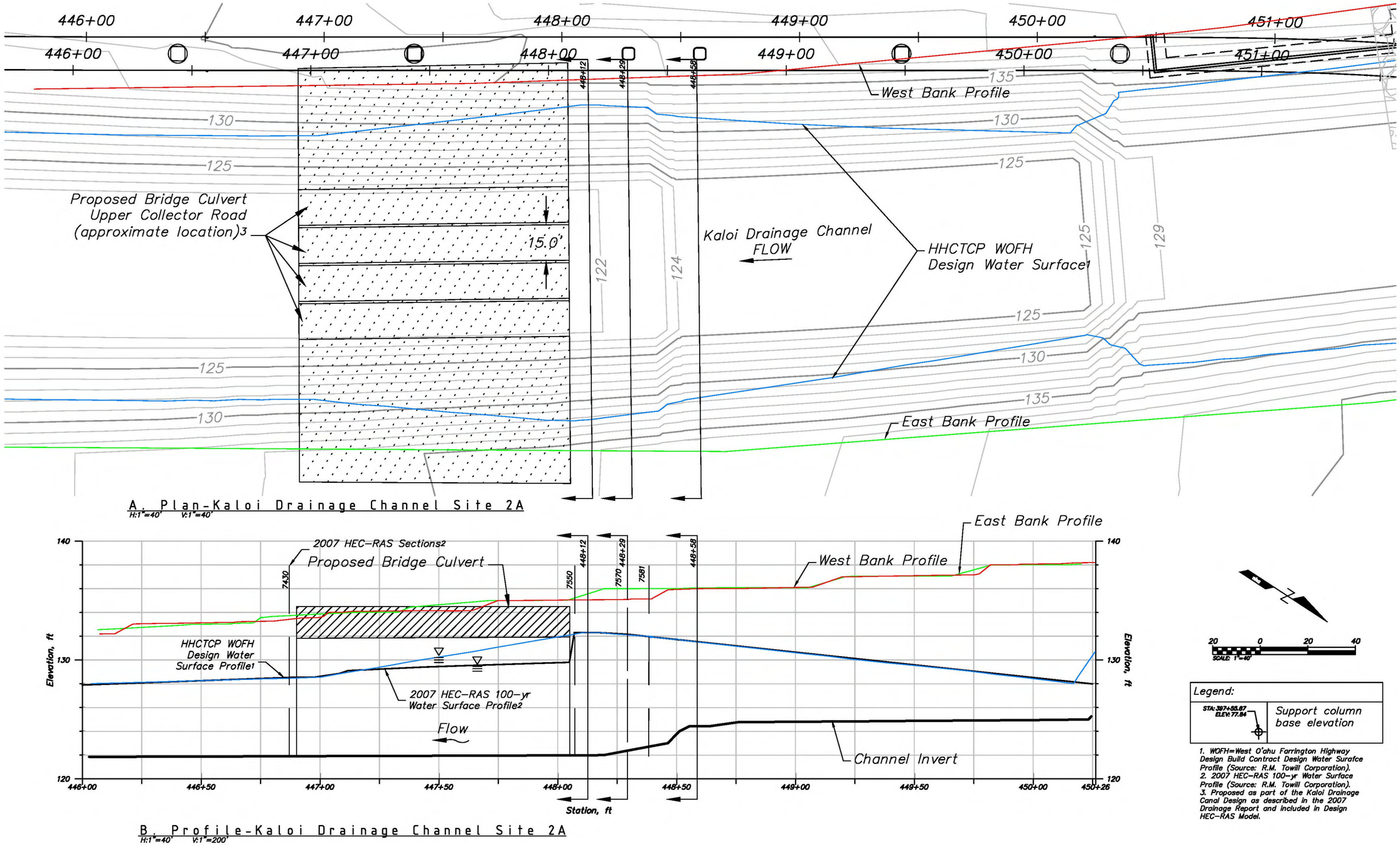
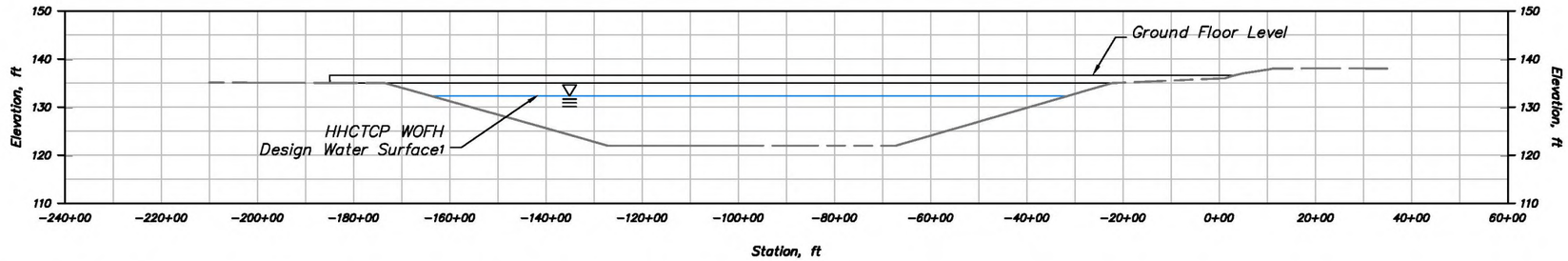
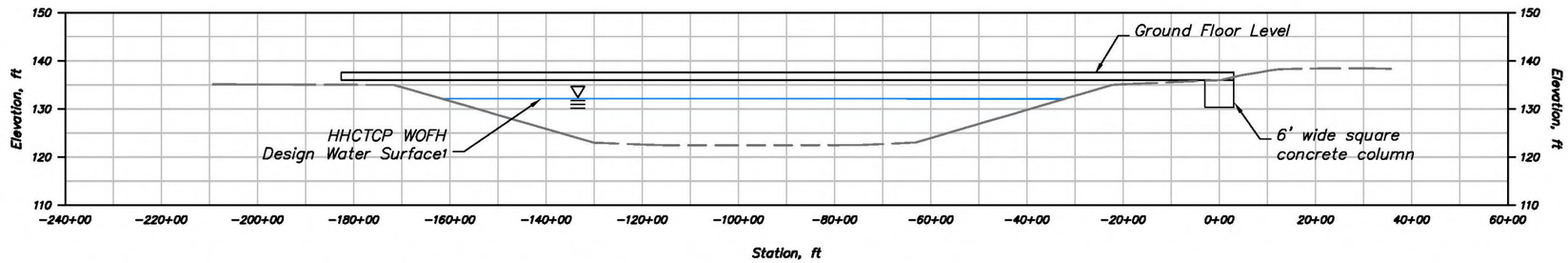


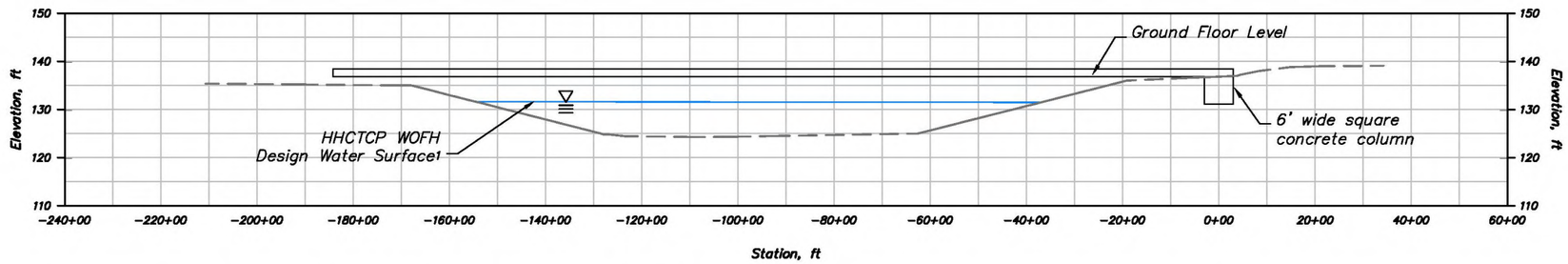
Figure 2A.3 Kaloi Drainage Channel Site 2A Plan and Profile



A. SECTION 448+12 [RAS 7180] (looking downstream)
H:1"=30' V:1"=30'



B. SECTION 448+29 [RAS 7168] (looking downstream)
H:1"=30' V:1"=30'



C. SECTION 448+58 [RAS 7197] (looking downstream)
H:1"=30' V:1"=30'

1. WOFH=West O'ahu Farrington Highway Design Build Contract Design Water Surafce Profile (Source: R.M. Towill Corporation).

Figure 2A.4 Kaloi Drainage Channel Site 2A Sections

2B - Kaloi Drainage Channel - Guideway Channel Crossing North of UH West Oahu Station

Site Description

Site 2B is located northeast of the UH West Oahu Station, between corridor station 450+00 to 456+00 in Corridor Segment B (Figure 2B.1). Site 2B includes a section of the elevated guideway that crosses from west to east over the earthen Kaloi Drainage Channel located in Segment B of Honolulu High Capacity Transit Corridor Project (HHCTCP). Once constructed, three 6-foot square guideway support columns will be located inside the Kaloi Drainage Channel (Figure 2B.2). Representatives from NHC, PB and LAI briefly visited the Kaloi Drainage Channel near Site 2B on September 22, 2009. They observed active construction activities related to Kualakai Parkway (North-South Road), however, construction of the Site 2B guideway support columns in the Kaloi Drainage Channel had not been initiated at that time.

Hydrologic Analysis and FEMA Study

The Kaloi Drainage Channel was recently constructed and is in the jurisdiction of the Hawaii Department of Transportation (HDOT). The land areas and floodplains adjacent to the Kaloi Drainage Channel have not been analyzed or mapped by FEMA as part of the effective Flood Insurance Study (FIS) for Honolulu, HI. Therefore, no FEMA FIS reports, floodplain maps or information are available for Site 2B. R.M. Towill Corporation provided the PB Study Team with their January 15, 2007 HEC-RAS model data files that were used to design the Kaloi Drainage Channel[1] (referred to herein as the "design HEC-RAS model"). According to the North-South Road Drainage Report,[2] the channel was designed for ultimate built-out conditions and flows that can occur during a 100-year flood event. The 100-year design flows specified in the design HEC-RAS model are 5,800 cfs at the upstream end of the channel (which is located approximately 625 feet upstream from Site 2B) and increases gradually to 7,400 cfs near the downstream end of the channel at the stormwater retention basin. According to the design HEC-RAS model, the 100-year design flow at Site 2B, where the guideway crosses the Kaloi Drainage Channel, is 5,800 cfs.

[1] Email from PB containing R.M. Towill Corporation's design HEC-RAS model flies for Kaloi Drainage Channel, January 25, 2010.

[2] HDOT/R.M. Towill Corporation, (July 2007), "Drainage Report for North-South Road Kapolei Parkway to Interstate Route H-1 Ewa, Oahu, Hawaii," prepared for Department of Transportation, Highways Division, Kapolei Hawaii, prepared by R.M. Towill, July 2007, 21 pages plus appendices.

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Hydraulic Analysis

The Kaloi Drainage Channel is a steep (slope ~ 0.012) earthen channel (approximately 150-foot top width, 60-foot bottom width, and 2:1 side slopes) with 2- to 6-foot-high, rock-covered drop structures spaced approximately 80 to 300 feet apart to reduce velocities and provide a milder water surface slope. NHC used the design HEC-RAS model to generate and plot the 100-year design water surface profile between corridor stations 448+50 to 455+50 at Site 2B. Figures 2B.3A and 2B.3B show the plan and profile of the Kaloi Drainage Channel and the location where the guideway will cross the channel. Figures 2B.3A and 2B.3B also show the location of the guideway support columns and the 100-year HHCTCP West Oahu Farrington Highway Design Build Contract Design Water Surface Extent and Profile[3] (referred to as the WOFH water surface extent and profile in Figure 2B.3). The base elevations of the guideway support columns are noted in Figure 2B.3B.

Figures 2B.3A and 2B.3B also show the 100-year HHCTCP WOFH Design Water Surface (**blue** line) and the 100-year design HEC-RAS water surface profile (**black** line) along the channel at Site 2B. Top-of-channel bank profiles were developed from contour data prepared by PB from Segment B LiDAR data, which includes the Kaloi Drainage Channel. Figures 2B.3A and 2B.3B show the east and west top-of-channel bank profiles along the Kaloi Drainage Channel as the **green** and **red** line profiles, respectively. Comparison of the **red** or **green** line top-of-bank profiles with the **blue** or **black** line 100-yr water surface profiles provides an estimate of the in-channel freeboard during the 100-year event in the vicinity of the guideway crossing. Similarly, comparison of the **blue** or **black** line water surface profiles with the channel invert (channel bottom) profile provides an estimate of flow depth at each column during the 100-yr flood event.

Figures 2B.4A, 2B.4B, 2B.4C and 2B.4D show cross section views of the channel, guideway support columns and the HHCTCP WOFH Design Water Surface (**blue** line) at each cross section. Figure 2B.4A shows the base elevation of the guideway support column and design water surface (**blue** line) located at station 450+39. Note that the column at station 450+39 is located adjacent to an existing 4-foot drop structure (Figure 2B.3A) and its base elevation is approximately 2.5 feet above the HHCTCP WOFH design water level at that cross section. There are three additional guideway support columns located at corridor stations 451+72, 452+62, and 454+44 that penetrate below the HHCTCP WOFH design water level and may alter local flow conditions. The two guideway support columns at stations 452+62 and 454+44 straddle an existing 2.5-foot high drop structure located in the Kaloi Drainage Channel. The guideway support column at station 452+62 is located just downstream from the drop structure while the guideway support column at station 454+44 is located just upstream from the drop structure. Figures 2B.1 and 2B.2 show that the three columns are located in a slightly curved section of the Kaloi Drainage Channel where flows bend slightly from left to right in the downstream direction.

Design 100-year flow conditions for the Kaloi Drainage Channel are listed in Appendix F in the North-South Road Drainage Report. The 100-year flows in this portion of the Kaloi Drainage Channel result in

[3] HDOT/R.M. Towill, 2007.

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average channel velocities of approximately 7 to 11 fps with super-critical flow conditions and average flow velocities of approximately 11 fps to 22 fps near the drop structures. Therefore, current baseline design conditions without addition of the proposed HHCTCP guideway support columns show that high-energy flows with these characteristics are likely to cause localized channel bed and bank scour and require periodic channel maintenance to repair scour-related damage.

PB and Lyon Associates, Inc designed scour countermeasures to mitigate for likely additional localized channel scour that could be caused by placement of the three columns in the flow. These designs call for construction of 50-foot-long by 60-foot-wide by 6-foot-deep riprap scour protection pads around the base of each column located in the active channel (Figures 2B.3A, and 2B.4A through 2B.4D). Details for the column scour protection measures are found in "PB / Lyon Associates, Inc., West Oahu/Farrington Design Build Guideway Drainage Details & Sections".[4] Analyses and scour protection design calculations are found in the "Preliminary Drainage Report for the Honolulu High Capacity Transit Corridor Project West Oahu/Farrington Design-Build".[5] While these project-detailed scour countermeasures are designed to prevent localized scour at the base of the proposed columns, likely needs for periodic channel maintenance and scour repairs at Site 2B should be addressed during final design.

Regulatory Compliance and Conclusions

The Kaloi Drainage Channel is not located in a mapped FEMA flood hazard zone. The Hawaii Department of Transportation (HDOT) is responsible for operating and maintaining the Kaloi Drainage Channel. Therefore, hydraulic conditions are subject to HDOT design criteria. 100-year flood flow velocities and bottom shear stresses are quite high in the Kaloi Drainage Channel at Site 2B[6], which makes the channel vulnerable to bed and bank erosion during these high flows. High-energy flows during large flood events are likely to cause localized scour along the channel bed and banks and require periodic channel maintenance to repair scour-related damage. Site 2B will have three guideway support columns that protrude into the 100-year design water surface. Construction of guideway support columns in the active flow portion of the Kaloi Drainage Channel will cause a slight rise in water levels and contribute to local scour processes. Scour countermeasures designed by PB and Lyon Associates, Inc are intended to mitigate for likely scour at the base of the three columns located in the flow. However, channel maintenance and scour repairs may still be required after large flood events. Therefore, likely needs for periodic channel maintenance and scour repairs at Site 2B should be addressed during final design.

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[4] PB / Lyon Associates, Inc., (2009), West O'ahu/Farrington Design Build Guideway Drainage Details & Sections Sheet 4 of 5, Contract No. DB-1200, Drawing No. GD104, Rev. B dated 04-03-09.

[5] Lyon Associates, Inc., (2009), Preliminary Drainage Report for Honolulu High Capacity Transit Corridor Project West O'ahu/Farrington Design-Build, March 27, 2009.

[6] HDOT/R.M. Towill, 2007, Appendix F

References

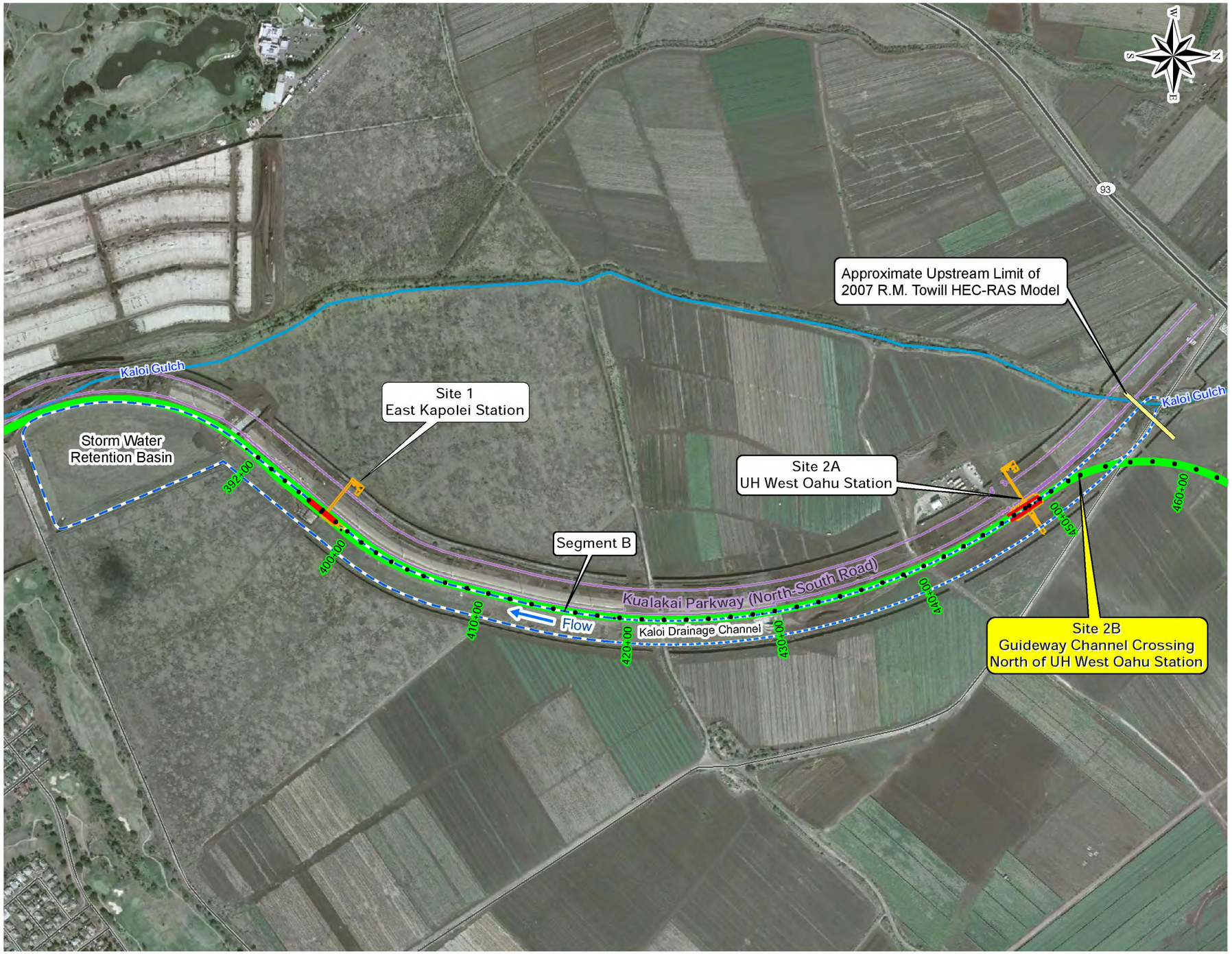
Email from Paul Fujii at PB containing R.M. Towill Corporation's design HEC-RAS model flies for Kaloi Drainage Channel; email dated January 25, 2010.

HDOT/R.M. Towill Corporation, (July 2007), "Drainage Report for North-South Road Kapolei Parkway to Interstate Route H-1 Ewa, Oahu, Hawaii," prepared for Department of Transportation, Highways Division, Kapolei Hawaii, prepared by R.M. Towill, July 2007, 21 pages plus appendices.

Lyon Associates, Inc., (2009), Preliminary Drainage Report for Honolulu High Capacity Transit Corridor Project West O'ahu/Farrington Design-Build, March 27, 2009.

PB / Lyon Associates, Inc., (2009), West O'ahu/Farrington Design Build Guideway Drainage Details & Sections Sheet 4 of 5, Contract No. DB-1200, Drawing No. GD104, Rev. B dated 04-03-09.

Figure 2B.1 - Segment B
Site 2B Location Map

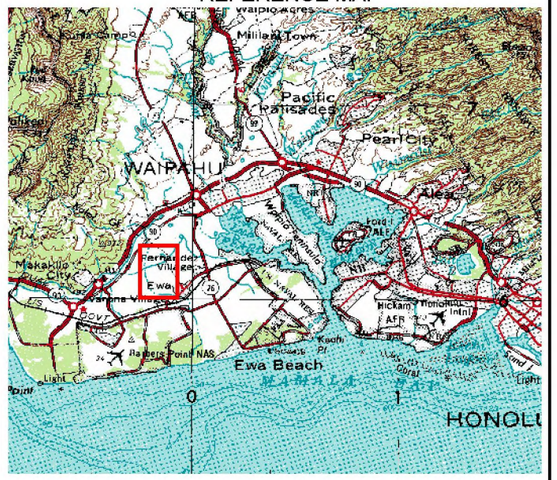


Scale - 1:8,400 1 inch = 700 feet

0 350 700 1,400 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50597	April 2010

REFERENCE MAP

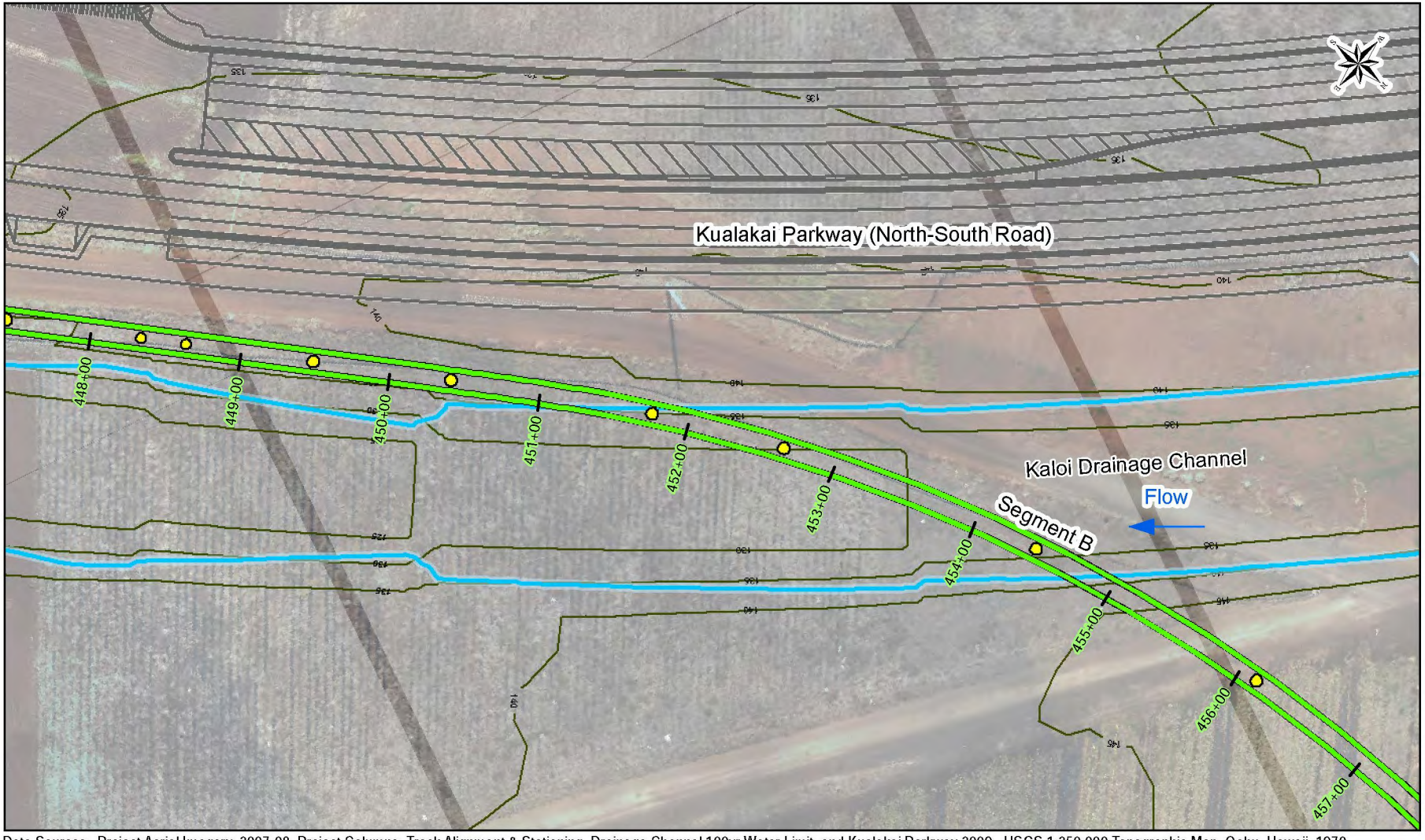


LEGEND

- Proposed Guideway Support Columns
- Proposed Platforms
- Proposed Track Alignments (with stationing)
- Kaloi Drainage Channel
- Roads
- Streams

Data Sources: Google Earth Imagery, Dec 2008. ESRI Roads and Streams, 2008. Project Columns, Platforms, Track Alignment, Channel, and Kualakai Parkway, 2009. USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

Figure 2B.1 Segment B - Site 2B Location Map

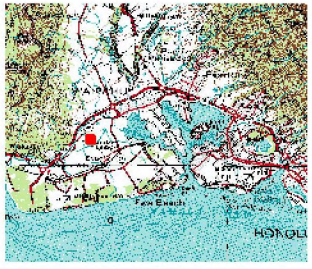


Data Sources: Project Aerial Imagery, 2007-08. Project Columns, Track Alignment & Stationing, Drainage Channel 100yr Water Limit, and Kualakai Parkway 2009. USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

Legend

- Guideway Support Columns
- 5ft Contours
- Track Alignment
- Kaloi Drainage Channel 100yr Water Limits
- West Bank Track Alignment Stationing

Oahu Reference Map



Honolulu High Capacity Transit Corridor

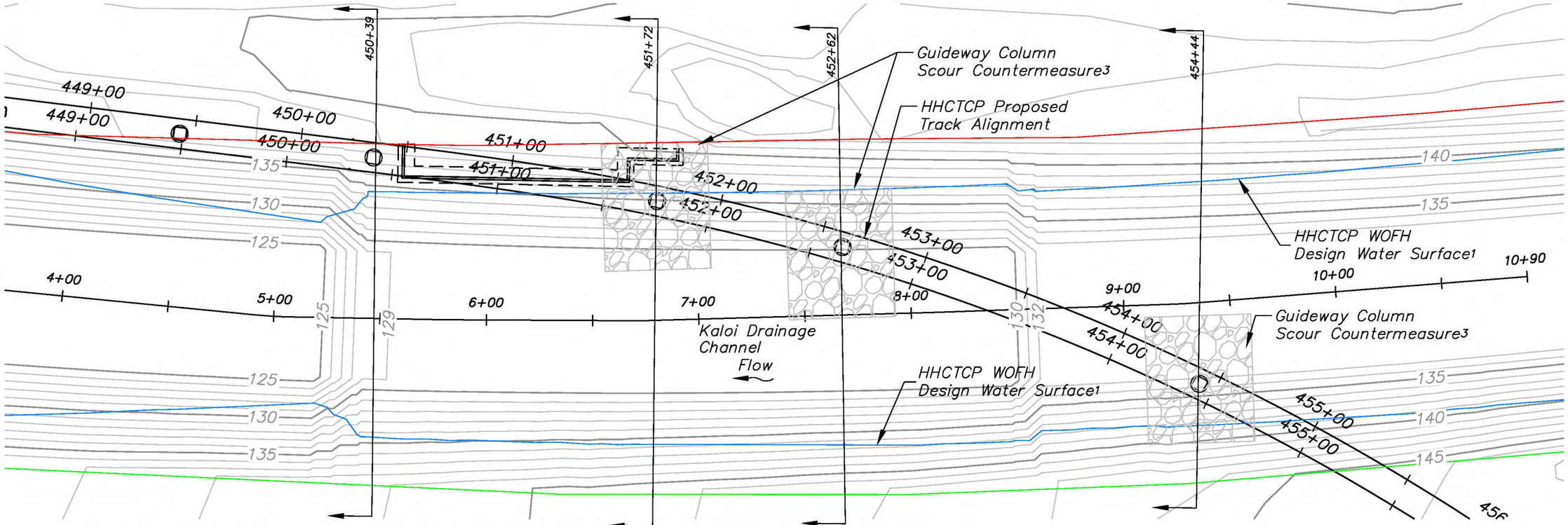
Site2B - Kaloi Drainage Channel Guideway Crossing Assessment

Scale -1:1,200

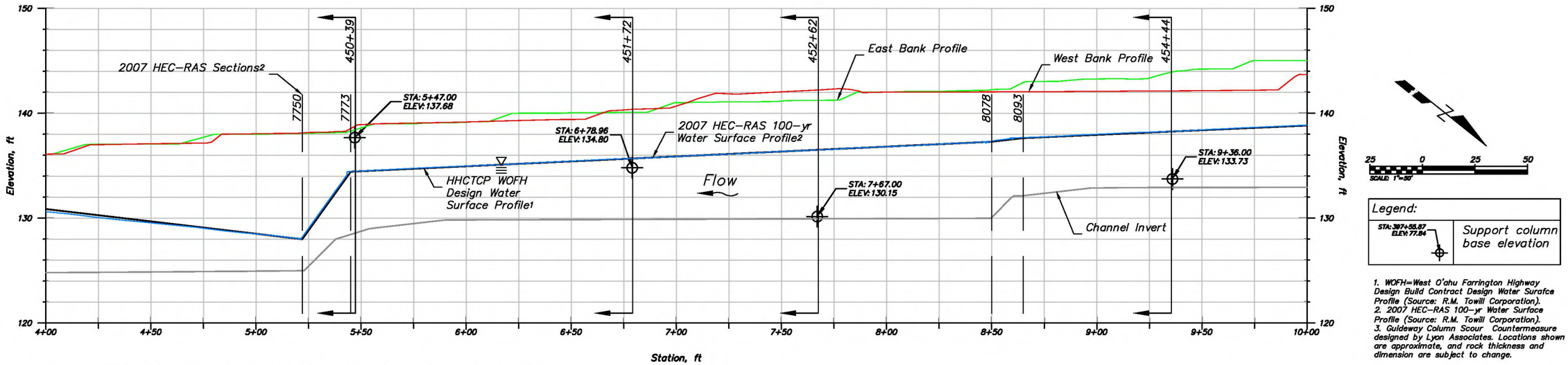
50 25 0 50 100 Feet

coord. syst.: HI State Plane 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50597	Apr-2010

Figure 2B.2



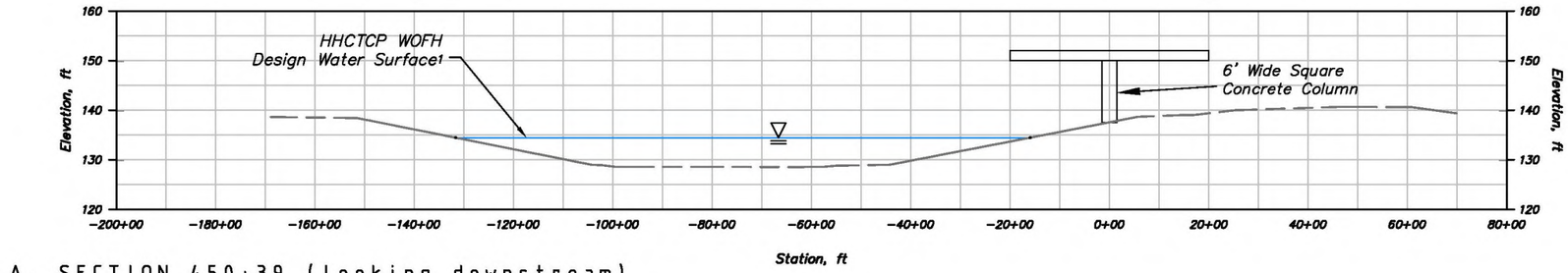
A. Plan-Kaloi Drainage Channel Site 2B
H:1"=50' V:1"=50'



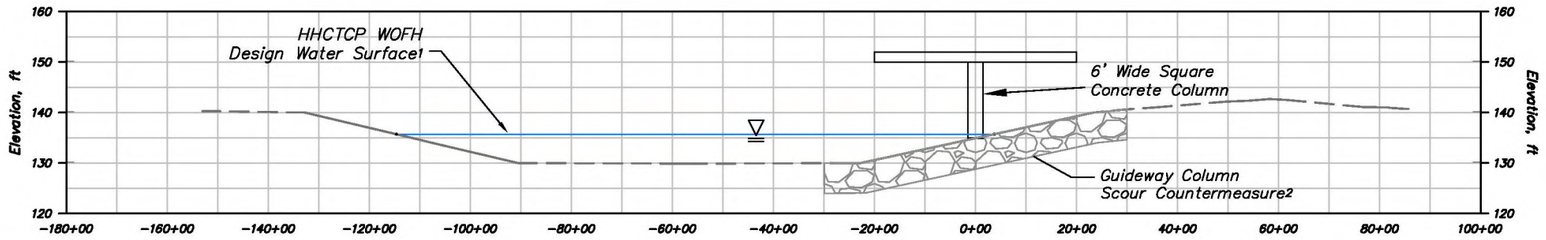
B. Profile-Kaloi Drainage Channel Site 2B
H:1"=50' V:1"=250'

1. WOFH=West O'ahu Farrington Highway Design Build Contract Design Water Surafce Profile (Source: R.M. Towill Corporation).
2. 2007 HEC-RAS 100-yr Water Surface Profile (Source: R.M. Towill Corporation).
3. Guideway Column Scour Countermeasure designed by Lyon Associates. Locations shown are approximate, and rock thickness and dimension are subject to change.

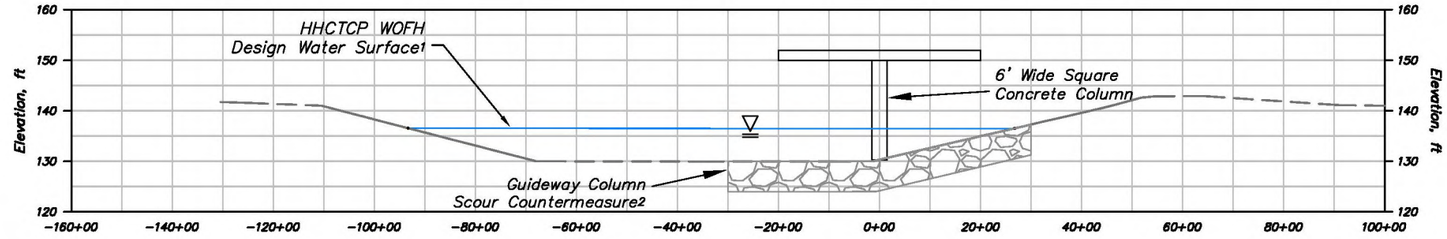
Figure 2B.3 Kaloi Drainage Channel Site 2B-Upper Drainage Canal Crossing Plan and Profile



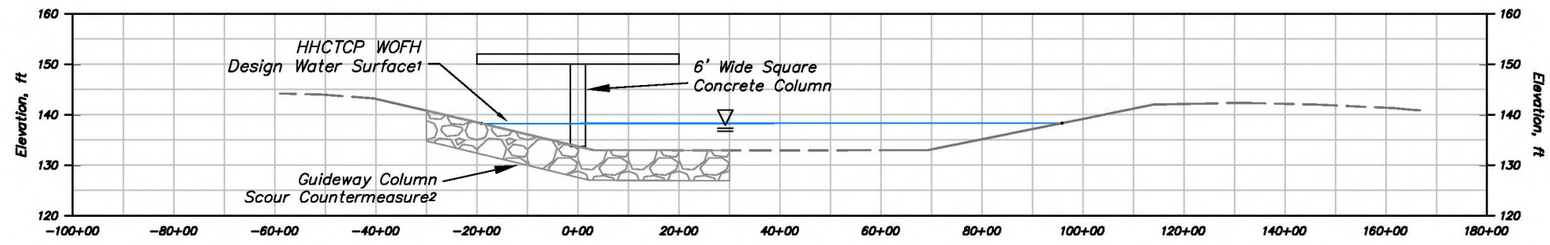
A. SECTION 450+39 (looking downstream)
H:1"=30' V:1"=30'



B. SECTION 451+72 (looking downstream)
H:1"=30' V:1"=30'



C. SECTION 452+62 (looking downstream)
H:1"=30' V:1"=30'



D. SECTION 454+44 (looking downstream)
H:1"=30' V:1"=30'

1. WOFH=West O'ahu Farrington Highway Design Build Contract Design Water Surafce Profile (Source: R.M. Towill Corporation).
2.Guideway Column Scour Countermeasure designed by Lyon Associates. Locations shown are approximate, and rock thickness and dimension are subject to change.

Figure 2B.4 Kaloi Drainage Channel Site 2B-Upper Drainage Canal Crossing Sections

3 - Honouliuli Stream Crossing

Stream Crossing Description

The Honouliuli Stream extends from the headwaters of the Waianae Mountain Range and discharges into the West Loch of Pearl Harbor. The proposed Honolulu High Capacity Transit Corridor Project (HHCTCP) crosses Honouliuli Stream at the Farrington Highway Bridge (see Figure 3.1). The elevated guideway will be supported by a series of 6-ft square columns spaced approximately 160-ft apart. The alignment follows the north side of the Farrington Highway and the columns span the stream without touching-down within the channel. A close up of the stream crossing, column locations, and 1-ft LIDAR contours are illustrated in Figure 3.2.

Representatives from NHC, PB and LAI visited the Honouliuli Stream Crossing location on September 23, 2009. On that date, the stream channel was dry. Photos 3.1 to 3.5 show the stream channel and the Farrington Highway Bridge.

Hydrologic Analysis and FEMA Study

Hydrology in the effective Flood Insurance Study (FIS) was completed using discharge records from the USGS Gaging Station No. 16212500 near Waipahu. A log-Pearson Type III analysis determined that the peak 100-year discharge at Farrington Highway was approximately 7730 cfs.

Flood hazards on Honouliuli Stream were analyzed as part of the effective FIS for Honolulu, HI. Zone AE (100-year flooding, detailed hydraulic analysis) is delineated from Pearl Harbor upstream to the Farrington Highway Bridge, and Zone A (100-year flooding, approximate study) boundaries extend upstream from the bridge (Figure 3.3).

Hydraulic Analysis

The existing FIS, site inspection, new 1-ft LiDAR contours, and high quality aerial photos were used to determine if the proposed columns will have an impact on flooding.

The floodplain upstream from the Farrington Highway is mapped as a FEMA Zone A (see Figure 3.3). New LiDAR data reveals these floodplain boundaries are incorrect because they do not follow consistent contour elevations. Only one column is located within the limits of the FEMA Zone A floodplain; however, it is on high ground that is above anticipated flood levels (Figure 3.4). The computed flood level at the downstream edge of the bridge is 75.2-ft (Figure 3.5) and although no flood level is reported at the upstream side of the bridge, it would not reach 80 feet which is the elevation of the ground surrounding the column. We have added a red line to Figure 3.4 to show approximately where the flood boundary should be along the left (east) side of the channel.

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Regulatory Compliance

There is sufficient evidence to conclude that the proposed columns will be located on high ground that is above 100-year flood levels. Therefore, there should be no Flood Hazard District concerns at this crossing.

Conclusions

The HHCTCP will have no impact on flood levels within Honouliuli Stream. The contour lines created from the new LiDAR reveal that the boundaries of the FEMA Zone A floodplain upstream from the Farrington Highway Crossing include areas of high ground which should not have been included. The one column located within the FEMA Zone A is on high ground that will remain dry during the 100-year flood event.

AR00041146

Photos

Photo 3.1. Honouliuli crossing viewing downstream

Photo 3.2. Under Honouliuli crossing viewing downstream.

AR00041147

Photo 3.3. Under Honouliuli crossing viewing upstream.

Photo 3.4. Honouliuli crossing viewing downstream.

AR00041148

Photo 3.5 Honouliuli crossing. NHC employee (in yellow vest) standing at location of proposed HHCTCP Column.

AR00041149

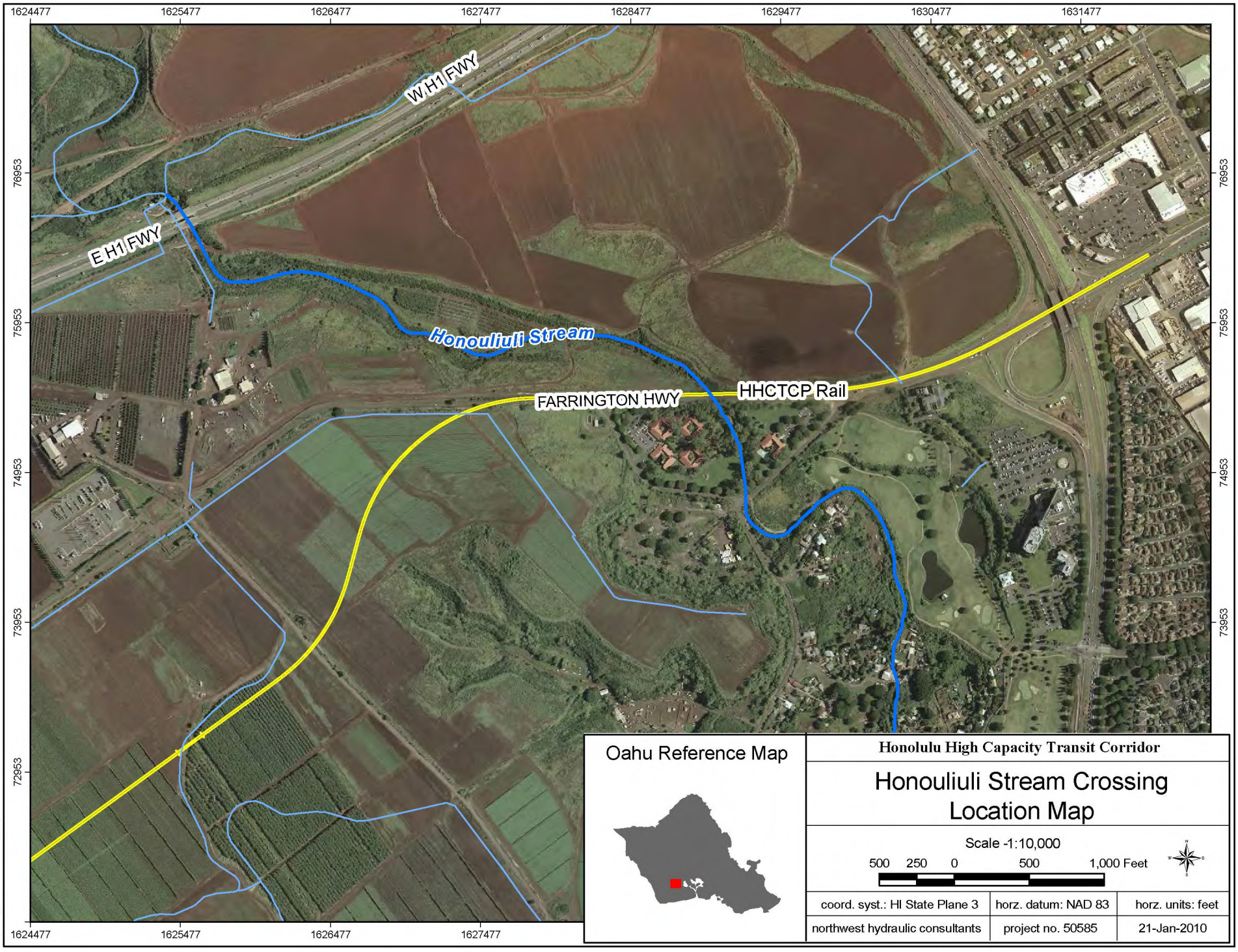
1624477
1625477
1626477
1627477
1628477
1629477
1630477
1631477
76953
75953
74953
73953
72953
W H1 FWY
E H1 FWY
Honouliuli Stream
FARRINGTON HWY
HHCTCP Rail
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Honouliuli Stream Crossing Location Map
Scale -1:10,000
500 250 0 500 1,000 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
21-Jan-2010

Figure 3.1

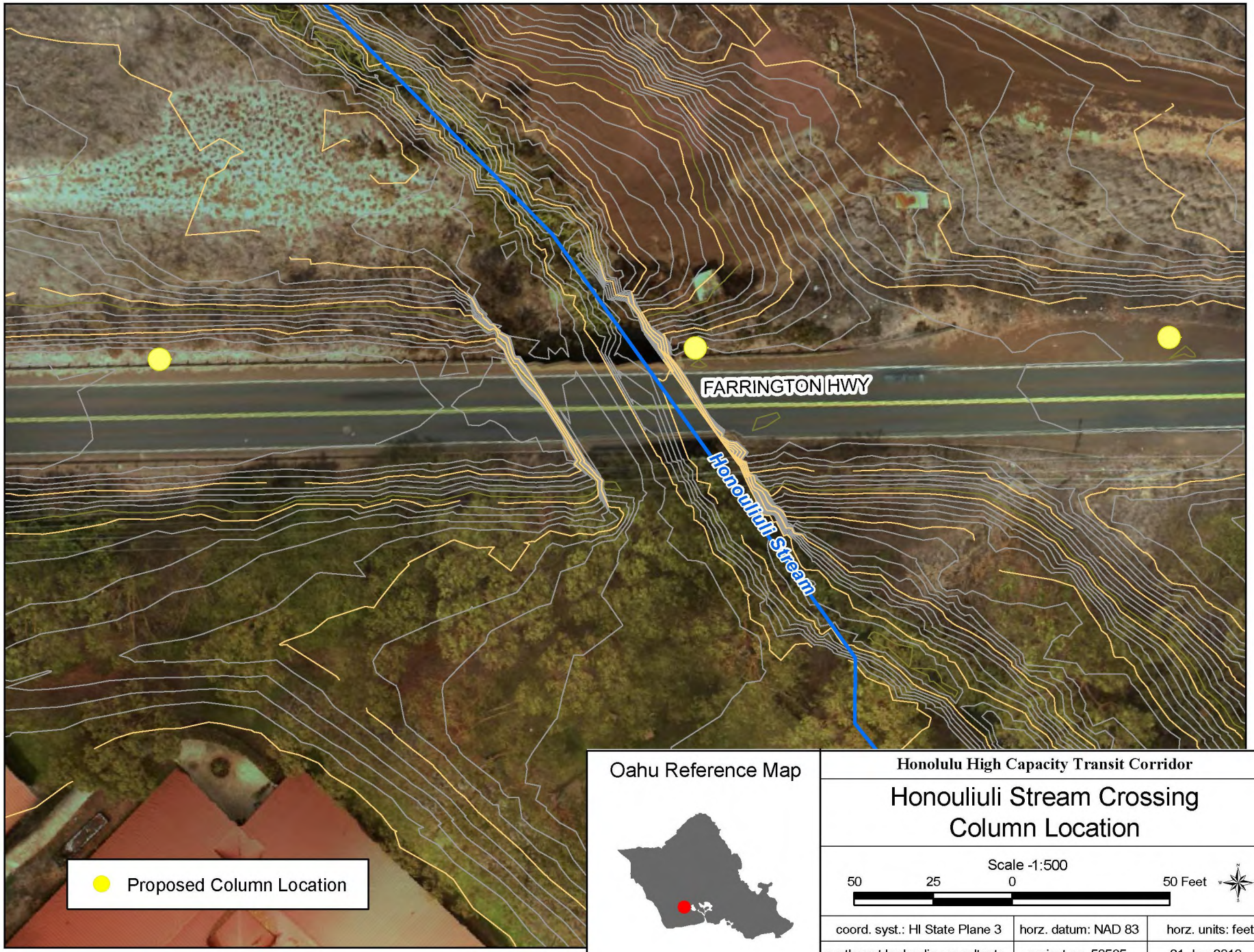
FARRINGTON HWY
Honouliuli Stream
Proposed Column Location
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Honouliuli Stream Crossing
Column Location
Scale -1:500
50
25
0
50 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
21-Jan-2010

Figure 3.2

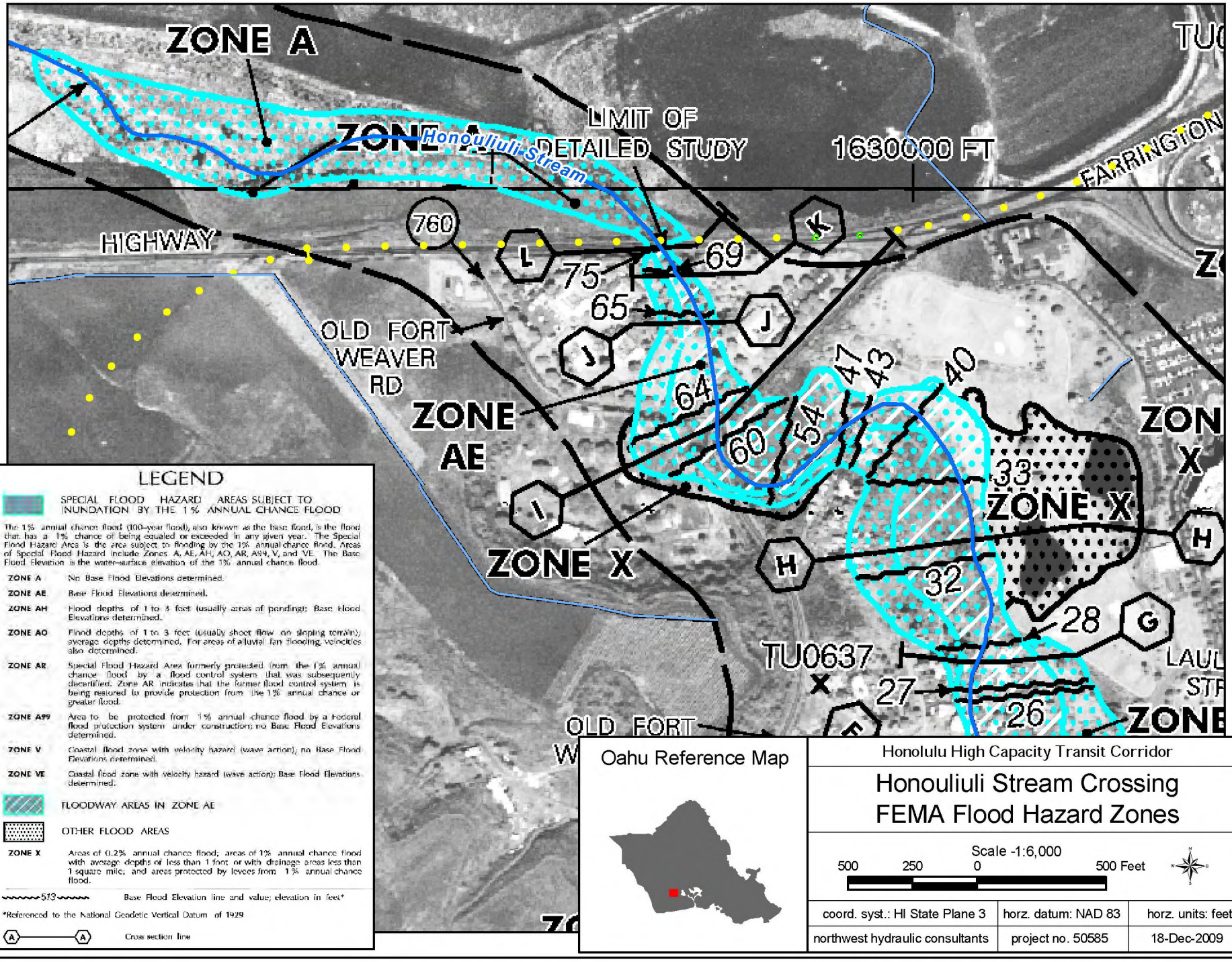
ZONE A
ZONE AE
Honouliuli Stream
LIMIT OF DETAILED STUDY
1630000 FT
FARRINGTON
HIGHWAY
760
OLD FORT WEAVER RD
ZONE AE
ZONE X
ZONE X
TU0637
OLD FORT
69
75
65
64
60
54
47
43
40
33
32
28
27
26
LEGEND
SPECIAL FLOOD HAZARD AREAS SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD
The 1% annual chance flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V, and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.
ZONE A No Base Flood Elevations determined.
ZONE AE Base Flood Elevations determined.
ZONE AH Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.
ZONE AO Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
ZONE AR Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
ZONE A99 Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.
ZONE V Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.
ZONE VE Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.
FLOODWAY AREAS IN ZONE AE
OTHER FLOOD AREAS
ZONE X Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.
513 Base Flood Elevation line and value; elevation in feet*
*Referenced to the National Geodetic Vertical Datum of 1929
A A Cross section line
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Honouliuli Stream Crossing
FEMA Flood Hazard Zones
Scale -1:6,000
500 250 0 500 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
18-Dec-2009

Figure 3.3

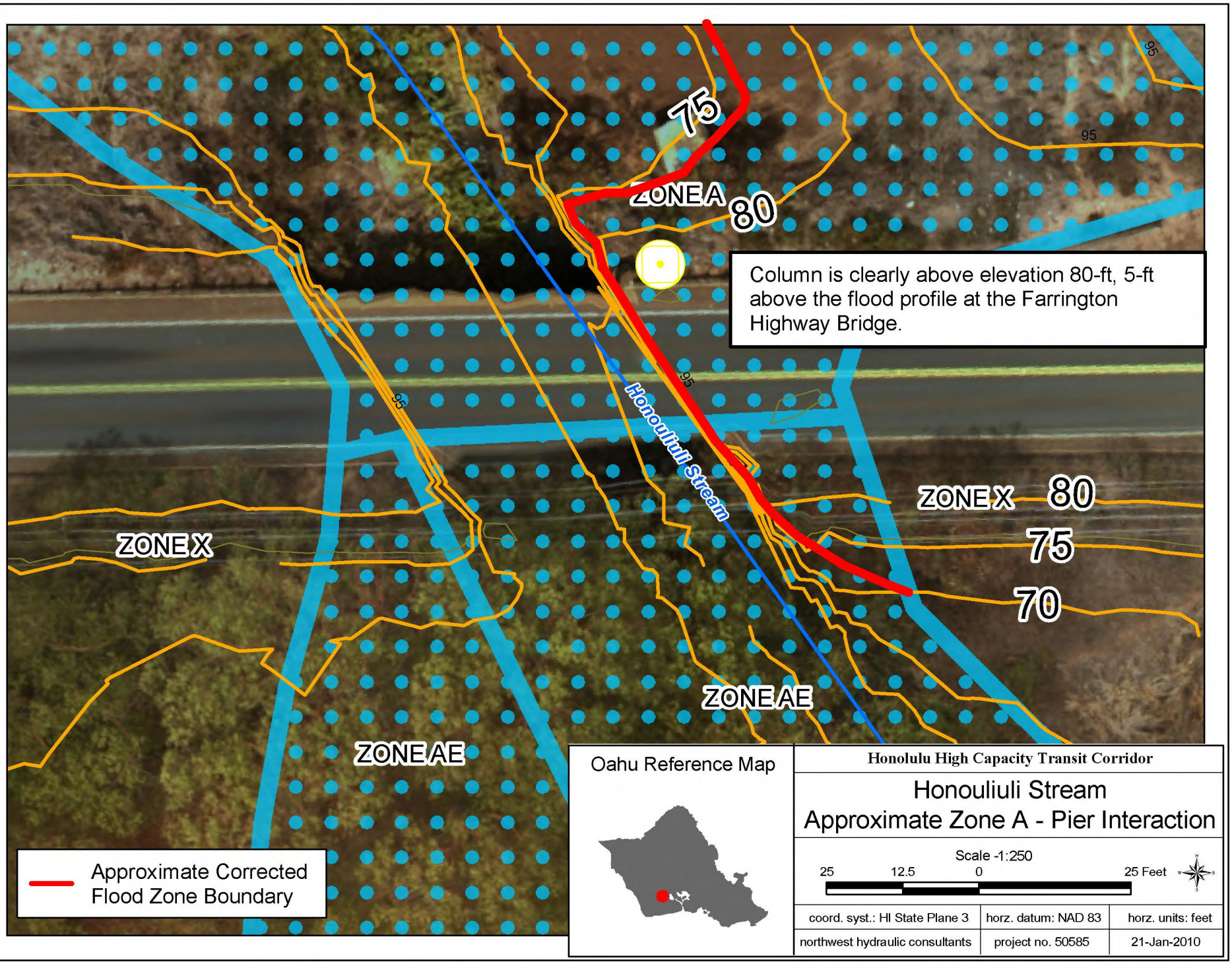
75
ZONE A
80
95
95
95
Column is clearly above elevation 80-ft, 5-ft above the flood profile at the Farrington Highway Bridge.
Honouliuli Stream
ZONE X
80
75
70
ZONE X
ZONE AE
ZONE AE
Approximate Corrected Flood Zone Boundary
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Honouliuli Stream
Approximate Zone A - Pier Interaction
Scale -1:250
25
12.5
0
25 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
21-Jan-2010

Figure 3.4

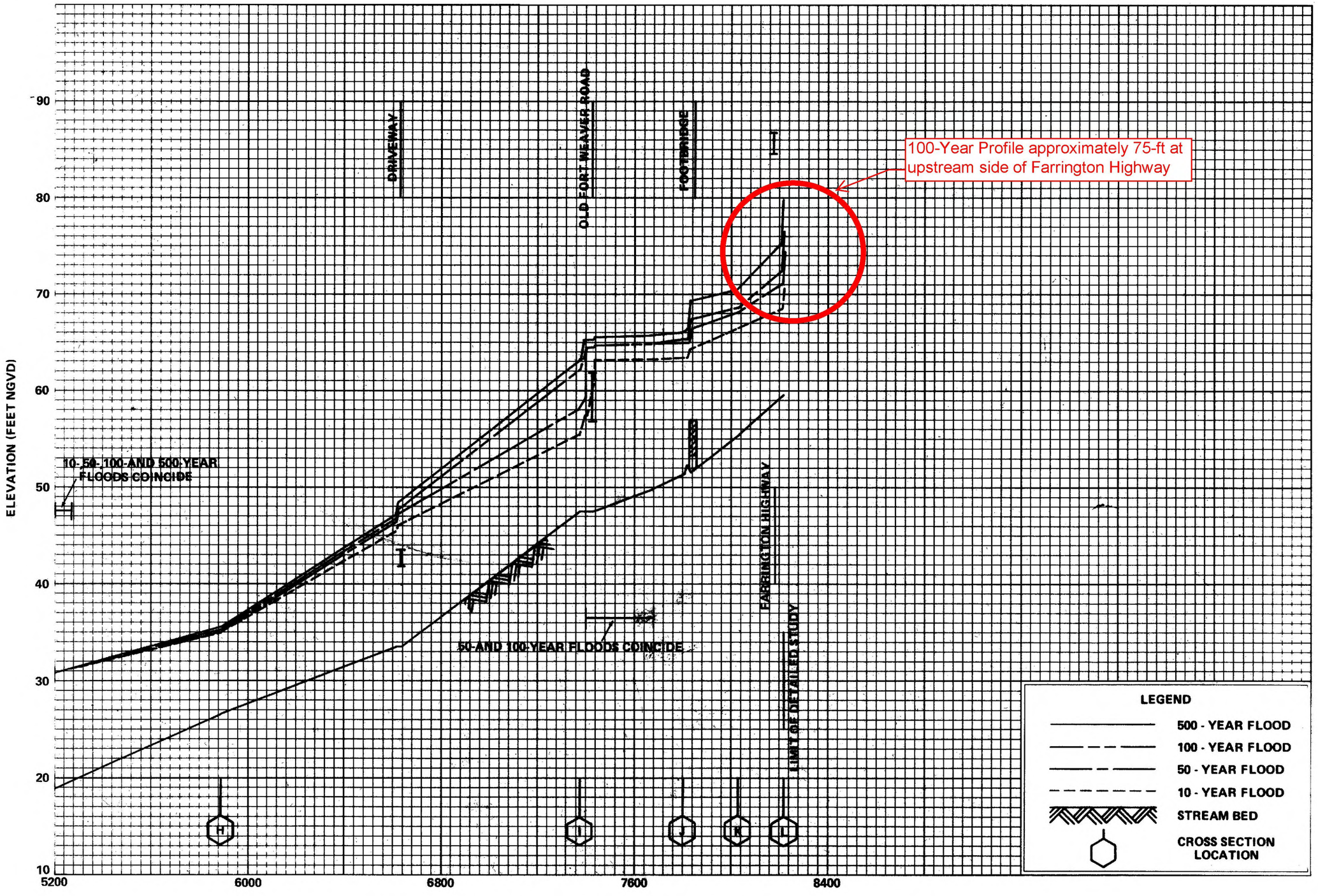

Figure 3.5. Honouliuli Effective FEMA Flood Profile

4 - Hoaeae Stream Crossing

Stream Crossing Description

The Hoaeae Stream Crossing is located near the Waipahu Shopping Center, where Hoaeae Stream flows under Farrington Highway on its way to the West Loch of Pearl Harbor (Figure 4.1). The lower portions of Hoaeae Stream are contained within a concrete lined trapezoidal "flood control channel" (see Photo 4.1). The proposed Honolulu High Capacity Transit Corridor Project (HHCTCP) will be located within the median of the Farrington Highway and will be elevated on columns that are setback from the stream crossing. The highway crossing consists of a pair of 12'4" x 7'9" sectional plate pipe arch culverts (Photo 4.2-4.3). An aerial view the crossing, column locations, and 1-ft LiDAR contours are illustrated in Figure 4.2.

Representatives from NHC, PB and LAI visited the Hoaeae Stream Crossing location on September 23, 2009. On that date, the stream channel was dry. Photos 4.1-4.5 depict the channel and proposed HHCTCP column locations near the crossing.

Hydrologic Analysis and FEMA Study

No hydrologic analysis was completed for this study. Hoaeae Stream does not have an effective Flood Insurance Study (FIS) or FEMA Flood Insurance Rate Map (FIRM).

Hydraulic Analysis

No hydraulic investigation was completed at this site because there is little chance that the crossing will have any measurable impact on flooding. Proposed columns are located along the median of Farrington Highway (see Figure 4.2, Photo 4.4), and are setback from the stream channel. Because a FEMA study has not been completed for Hoaeae stream we do not know if water will overtop the channel banks and cross the highway during a 100-year flood. This reach of the stream has been highly modified and enlarged to increase capacity for drainage and flood relief. Therefore, even during a major flood water may not leave the channel, or if it does, the amount will be relatively small. If water does leave the channel, the contours shown in Figure 4.2 indicate that it will cross the Farrington Highway and median as shallow sheet flow (See Photos 4.4-4.5). Therefore, only the base of the 6-ft by 6-ft track columns may touch the water and this will have no significant impact on flood levels.

Regulatory Compliance and Conclusions

To our knowledge a flood study has not been completed for this reach of the Hoaeae Stream and therefore, we do not know if the columns are located within the limits of the flood fringe. Regardless, it is our opinion that the likely impact the columns would have on flooding is so inconsequential that

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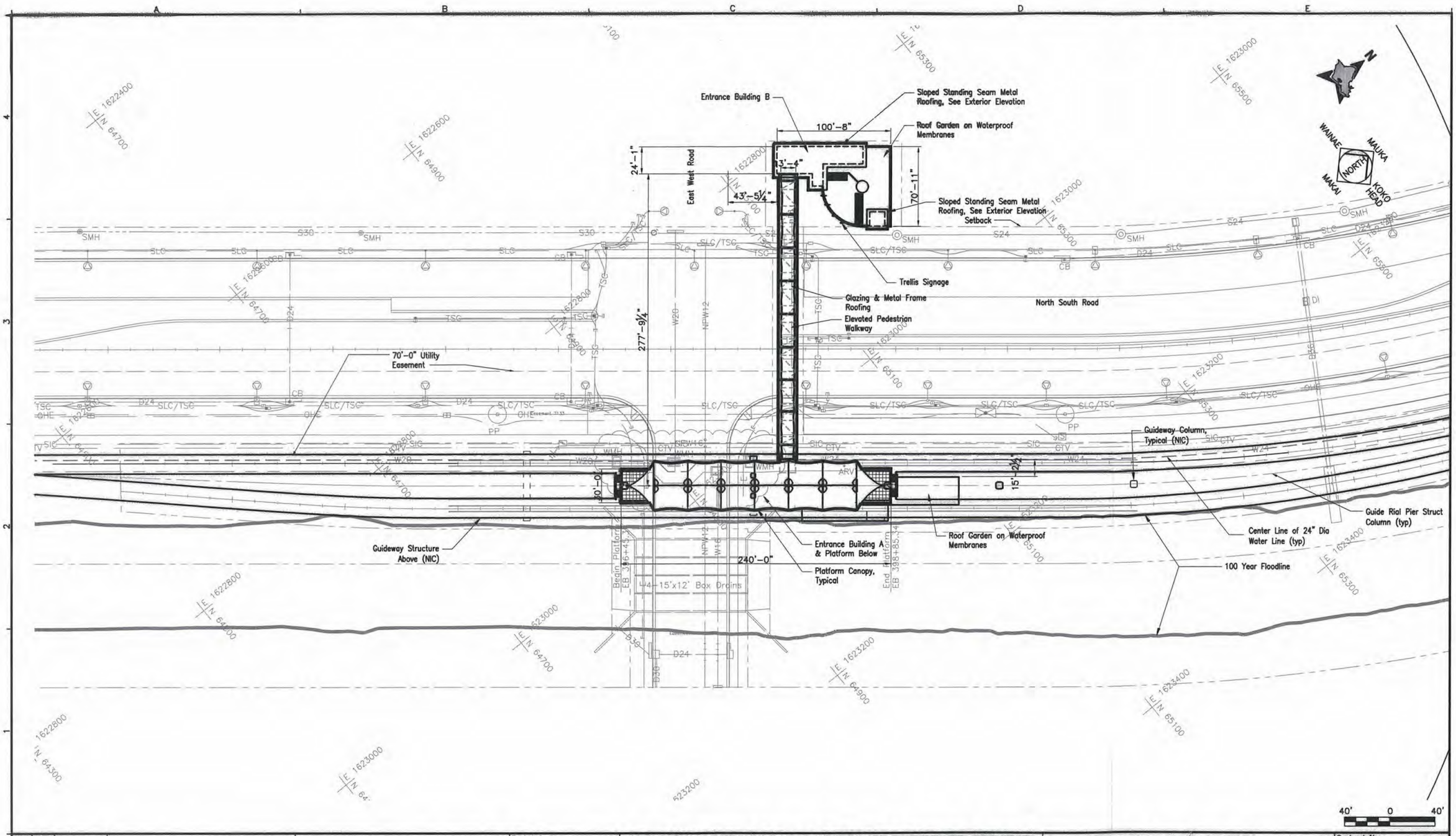
detailed hydrologic and hydraulic investigations are not warranted. The columns themselves are not at risk because they will be supported on deep foundations so scour is not a concern. Also, anticipated flood depths and velocities in the flood fringe would not likely damage 6-ft by 6-ft square reinforced concrete columns.

AR00041156



Rev	By	Date	Description

PRELIMINARY ENGINEERING SUBJECT TO REVISION

Designed: J Stone
Drawn: H Xue
Checked: EZ Honda
Approved: K Parmar
Date: 09-25-09

HONOLULU HIGH-CAPACITY TRANSIT CORRIDOR PROJECT

CITY & COUNTY OF HONOLULU - DEPARTMENT OF TRANSPORTATION SERVICES - RAPID TRANSIT DIVISION

Prime Consultant: PB PARSONS BRINCKERHOFF
1003 Bishop Street, Suite 2250 – Honolulu, HI 96813

Subconsultant: GROUP 70 INTERNATIONAL
Group 70 International, Inc.
925 Bethel Street, Fifth Floor
Honolulu, Hawaii 96813-4307

For reduced prints, original page size in inches: 0 1 2 3 4

EAST KAPOLEI STATION

ARCHITECTURAL SITE PLAN

Contract No.: SV-140
CADD File: SB1-H02-AR001
Drawing No.: AR001 Rev.
Scale: 1"=40'
Page No.: 43 of 56

AR00041157

Photos

Photo 4.1. Viewing Downstream at Hoaeae Stream channel and pipe arch culverts passing under Farrington Highway.

Photo 4.2. Hoaeae Stream culverts passing under Farrington Highway.

AR00041158

Photo 4.3. Hoaeae Stream channel, viewing upstream from Farrington Highway.

Photo 4.4. Looking east along Farrington Highway. NHC employee in orange vest at location of proposed HHCTCP Column.

AR00041159

Photo 4.5. Viewing west along Farrington Highway from the upstream side of the Farrington Highway Crossing.

AR00041160

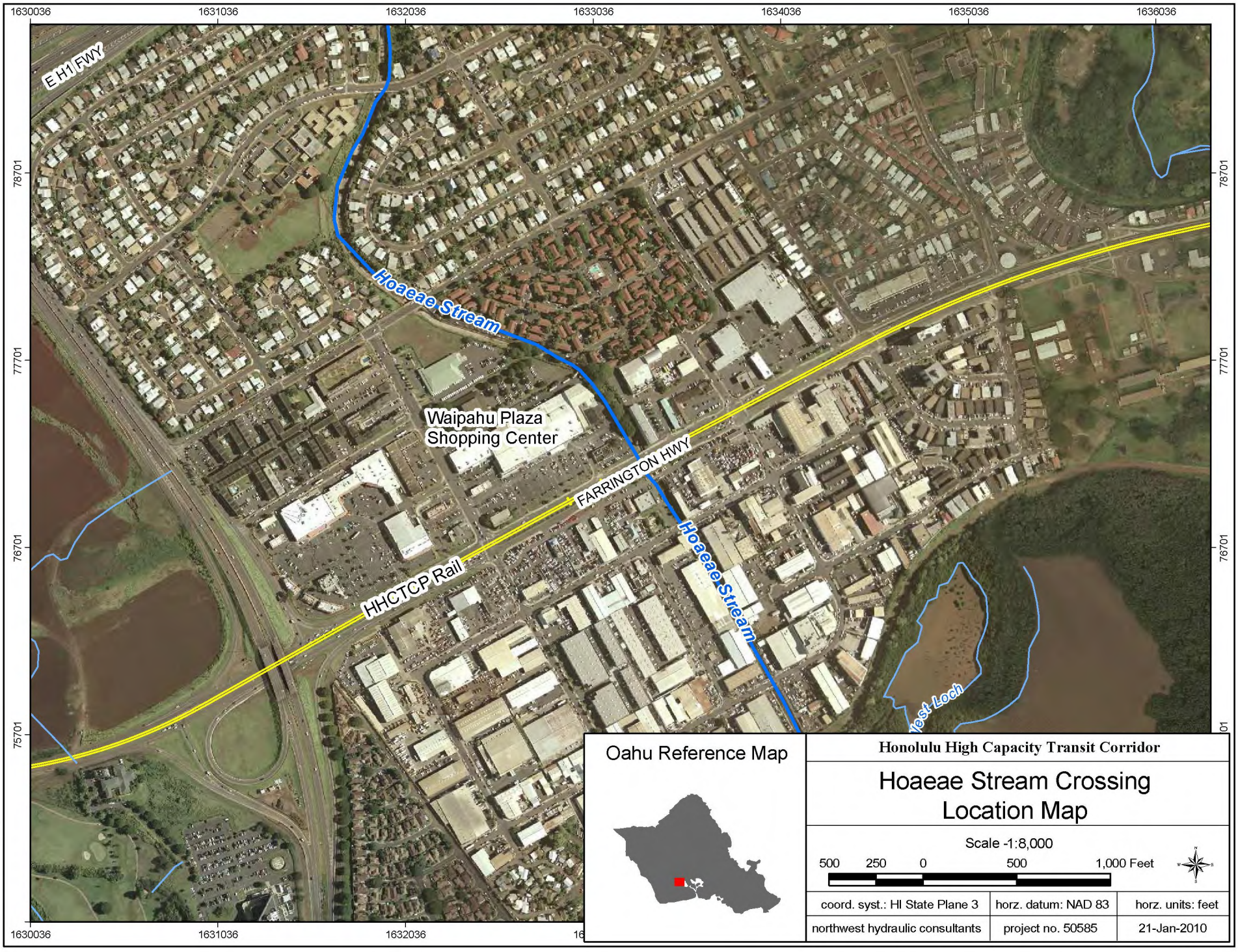


Figure 4.1

AR00041161

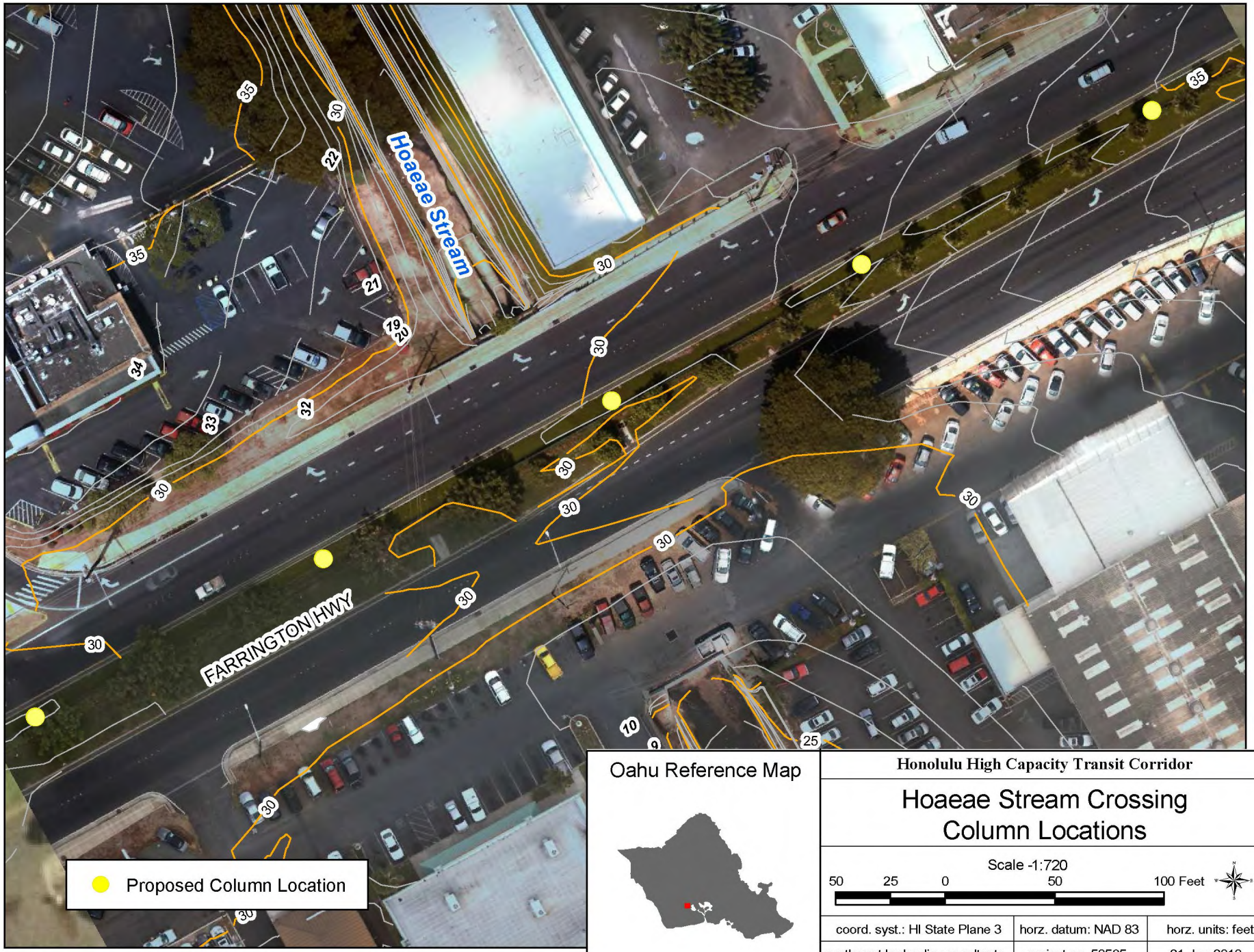

Hoaeae Stream
FARRINGTON HWY
Proposed Column Location
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Hoaeae Stream Crossing
Column Locations
Scale -1:720
50 25 0 50 100 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
21-Jan-2010

Figure 4.2

5 - Panakauahi Gulch Crossing

Stream Crossing Description

The Panakauahi Gulch Crossing is located approximately 1000-ft upstream of the H1 and H2 Freeway Junction, where Panakauahi Gulch flows beneath the H2 freeway and empties into Waiawa Stream (see Figure 5.1). The Panakauahi Gulch is an ephemeral stream that is steep, wide, and very deep (Figure 5.2). An access ramp for the proposed Honolulu High Capacity Transit Corridor Project (HHCTCP) elevated rail parallels the H2 Freeway, and crosses Panakauahi Gulch on a series of 8-ft circular columns located in and around the stream channel (see Figure 5.2).

Representatives from NHC, PB and LAI visited the Panakauahi Crossing location on September 23, 2009. On that date, the stream channel was dry.

Hydrologic Analysis and FEMA Study

Hydrology completed for the effective Flood Insurance Study (FIS) for Honolulu determined that the 100 year discharge on Panakauahi Gulch is 14,200-cfs at the confluence with Waiawa Stream.

Zone AE (100-year flooding, detailed hydraulic analysis) is delineated from the confluence with Waiawa Stream, through the H2 Freeway, and continues upstream of the crossing area (Figure 5.3). A FEMA Floodway is also delineated, starting at the Waiawa Floodway, and continuing through and upstream of the crossing site.

Hydraulic Analysis

No hydraulic investigation was completed at this site because there is little chance that the crossing will have any measurable impact on flooding. The effective flood profiles are steep through the project reach, and columns will have insignificant impact on water surface elevations.

Regulatory Compliance

Three of the proposed HHCTCP columns are located within the effective Flood Fringe, but remain outside of the Floodway District (see Figure 5.3). The columns are unlikely to have any significant effect on the steep water surface profile, and any increase in water surface elevation would be local to the column and would not increase the flood hazard risk of surrounding areas. Scour / erosion protection in the form of a rock riprap apron at the base of each column may be needed. This will be addressed during detailed design.

AR00041163

Conclusions

Although two HHCTCP columns are located within the Flood Fringe, they will have no significant impact on flood levels. Due to the relatively steep water surface profile and confined canyon any impact will be local to the column and will have no adverse impact on adjacent facilities or properties.

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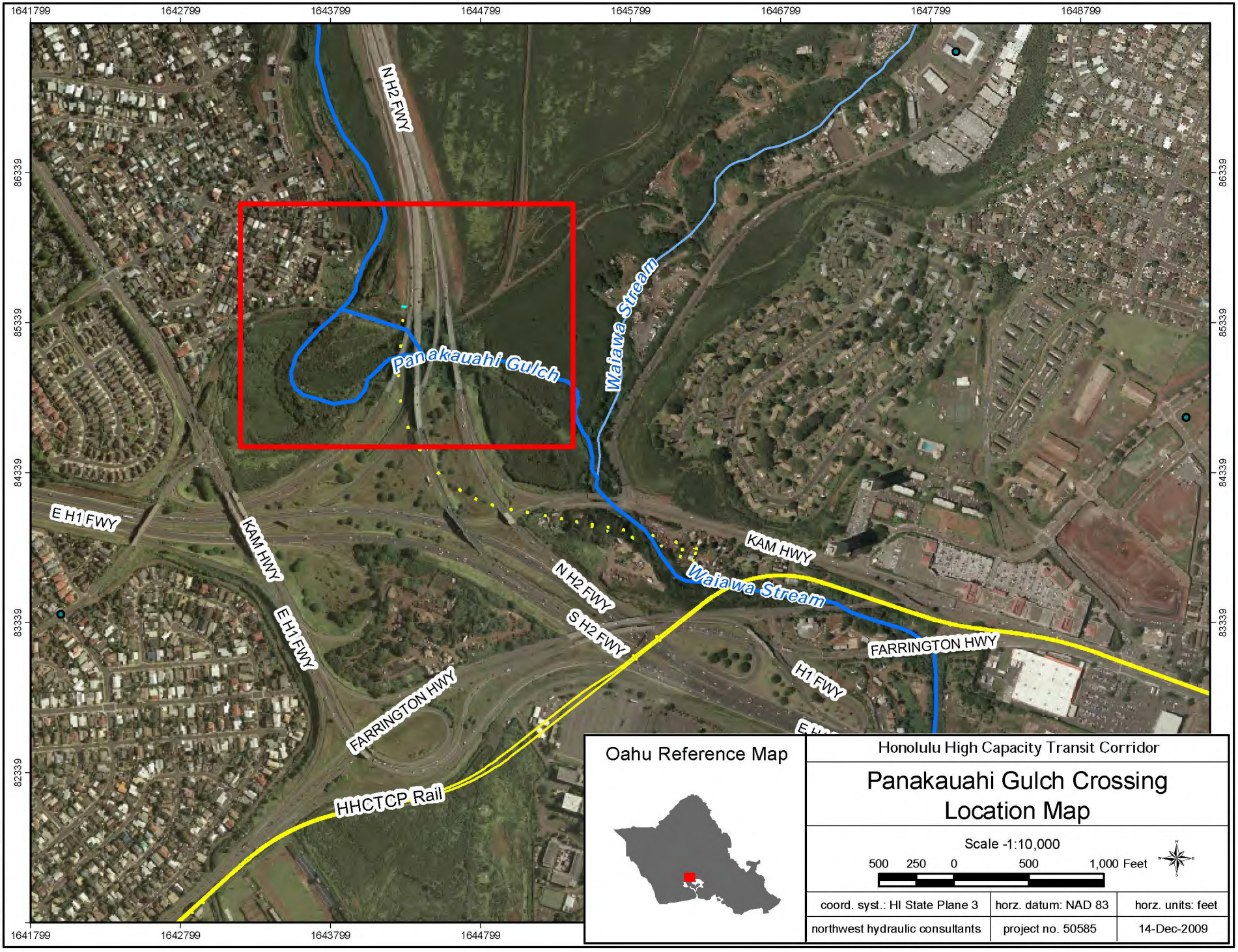


Figure 5.1

AR00041165

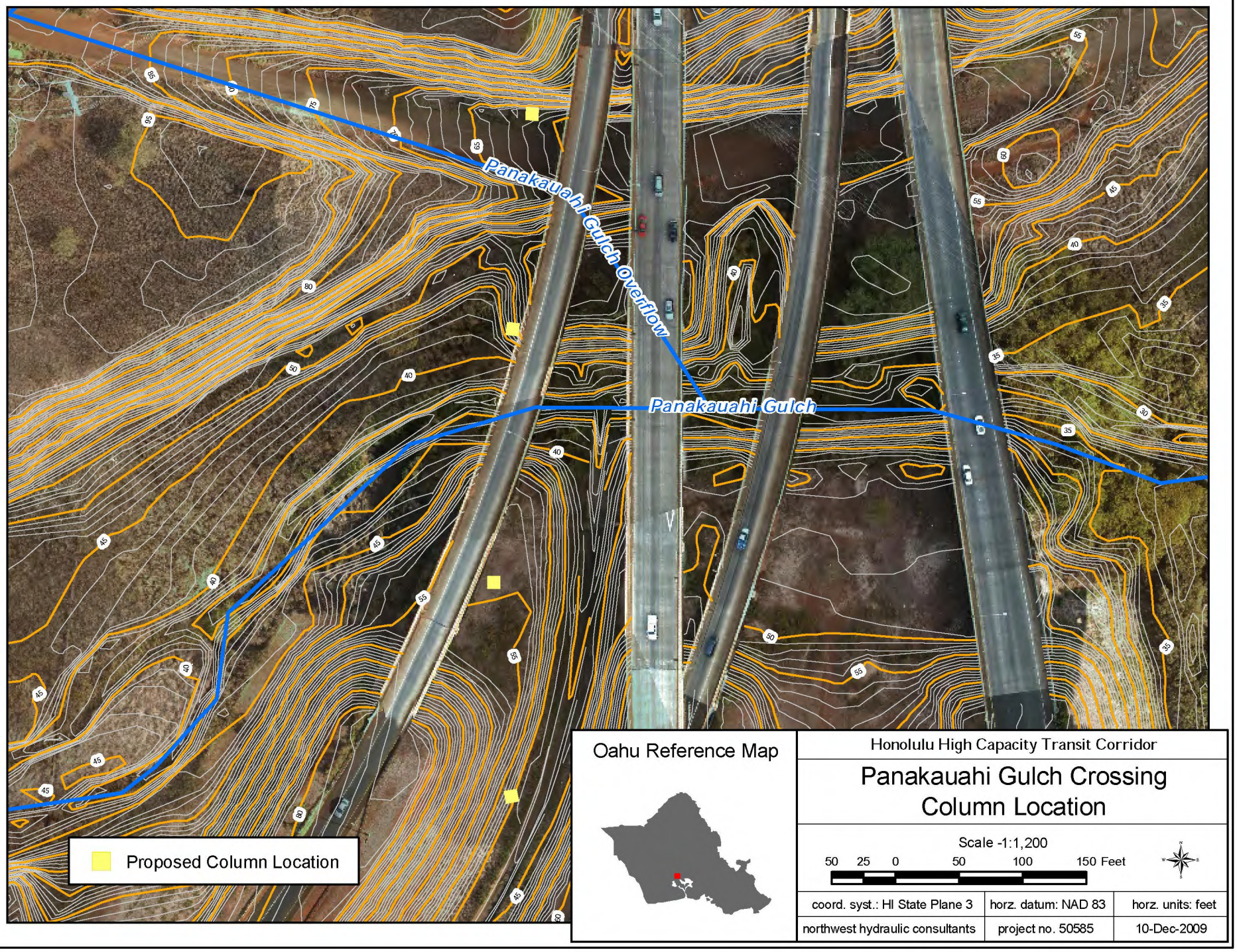

Panakauahi Gulch Overflow
Panakauahi Gulch
Proposed Column Location
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Panakauahi Gulch Crossing
Column Location
Scale -1:1,200
50 25 0 50 100 150 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
10-Dec-2009

Figure 5.2

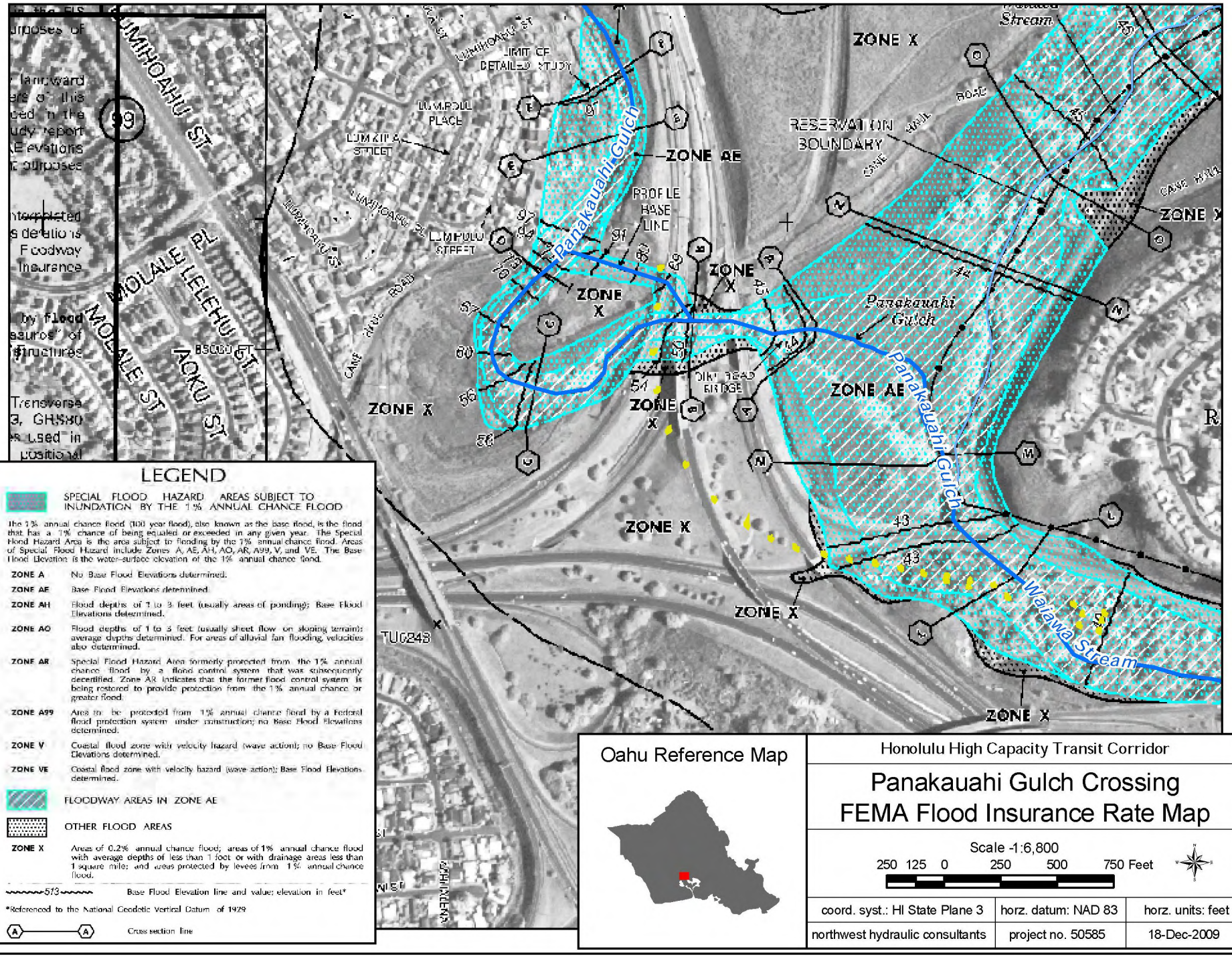


Figure 5.3

AR00041167

6 - Pearl City Stream Crossing

Stream Crossing Description

The Pearl City Stream is located in the Waimalu Watershed, near the Pearl City Shopping Center (see Figure 6.1). The stream is confined within a concrete culvert that passes under the Pearl City Shopping Center, and daylights briefly before passing under the Kamehameha Highway. The proposed Honolulu High Capacity Transit Corridor Project (HHCTCP) elevated rail parallels Kamehameha Highway and crosses the stream in the highway median between the east and west lanes on a series of 6-ft by 6-ft square columns setback from the stream channel location (see Figure 6.2).

Representatives from NHC, PB and LAI visited the Pearl City Stream Crossing location on September 23, 2009. On that date, the stream channel was dry. Photos 6.1-6.4 depict the stream channel and proposed HHCTCP column locations near the stream crossing.

Hydrologic Analysis and FEMA Study

No hydrologic analysis was completed for this investigation. Pearl City Stream does not have an effective Flood Insurance Study (FIS) or FEMA Flood Insurance Rate Map (FIRM).

Hydraulic Analysis

No hydraulic investigation was completed at this site because there is little chance that the crossing will have any measurable impact on flooding. As noted above, the track is supported on columns which are not located in the stream channel (Figure 6.2). Because a FEMA study has not been completed for Pearl City Stream we do not know if water will exit the channel and cross the highway during a 100-year flood. This reach of the stream has been highly modified and enlarged to increase capacity for drainage and flood relief. Therefore, even during a major flood water may not leave the channel, or if it does, the amount will be relatively small. If water does leave the channel, the contours shown in Figure 6.2 indicate that it will spread out and cross the highway as shallow sheet flow. The depth of water flowing around the columns would be relatively small and therefore the impact insignificant.

Regulatory Compliance

To our knowledge a flood study has not been completed for this reach of the Pearl City Stream and therefore, we do not know if the columns are located within the limits of the flood fringe. Regardless, it is our opinion that the likely impact the columns would have on flooding is so inconsequential that detailed hydrologic and hydraulic investigations are not warranted. The columns themselves are not at risk because they will be supported on deep foundations so scour is not a concern. Also, anticipated flood depths and velocities in the flood fringe would not likely damage 6-ft by 6-ft square reinforced concrete columns.

AR00041168

Conclusions

The columns are setback from the stream channel and may or may not be located within the flood fringe. The channel has been modified and enlarged to increase capacity, therefore, it is unlikely that significant flow will overtop the channel banks during the 100-year flood. However, if it does reach the columns, the water will be shallow and the impact to flooding will be insignificant. There will be no adverse impact on adjacent facilities or properties.

AR00041169

Photos

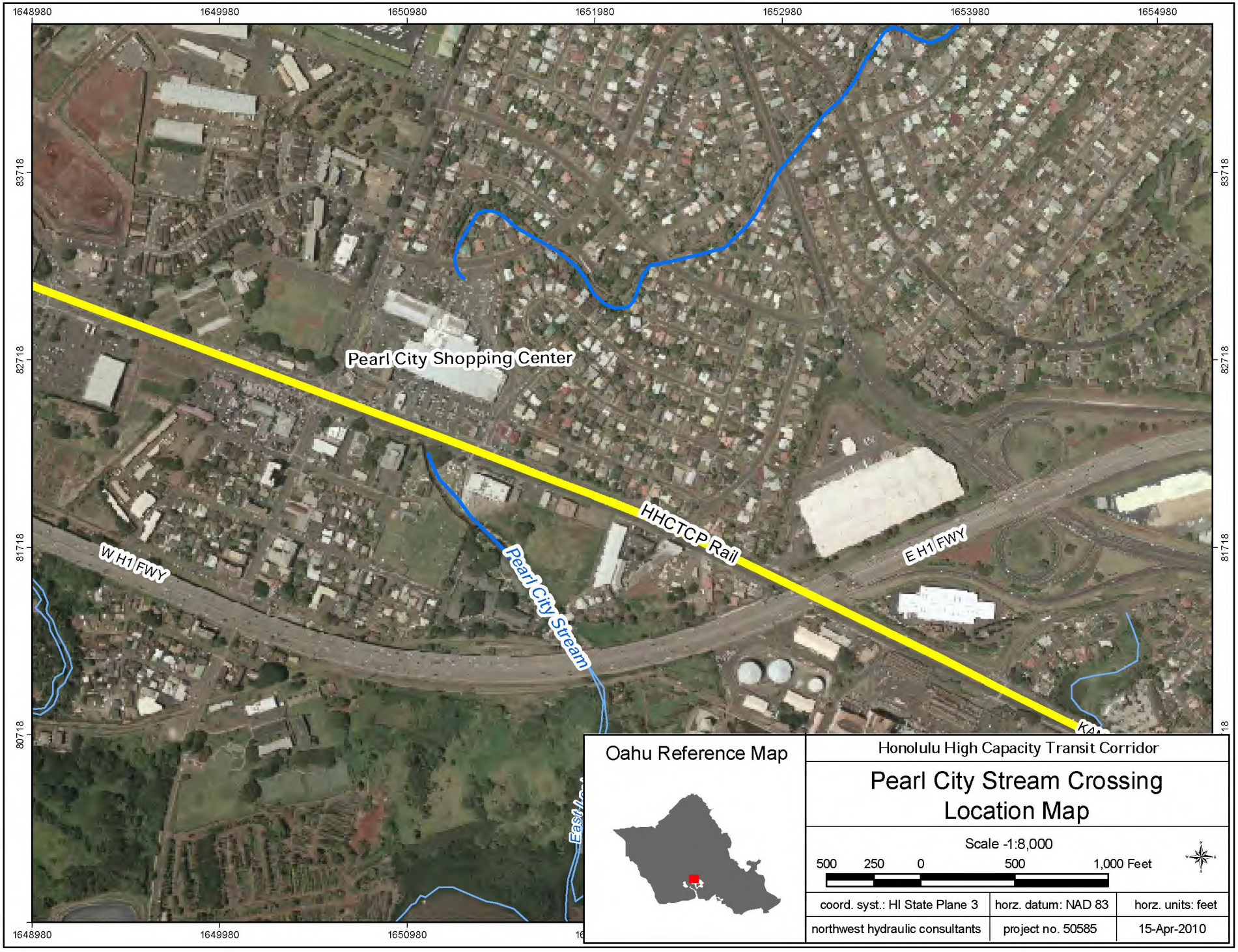
Photo 6.1. Pearl City Stream Crossing viewing downstream from right bank. Yellow barrels in photo center are near the proposed HHCTCP column locations.

Photo 6.2. Pearl City Stream Crossing viewing downstream from left bank. Yellow barrels in photo center are near the proposed HHCTCP column locations.

AR00041170

Photo 6.3. Standing on the Kamehameha Highway viewing upstream to the outlet of the culvert that carries the stream under the adjacent shopping center.

Photo 6.4. Viewing west along Kamehameha Highway towards Pearl City Stream Crossing. Yellow barrels in photo center are near the proposed HHCTCP column locations.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83718
82718
81718
80718
Pearl City Shopping Center
HHCTCP Rail
W H1 FWY
E H1 FWY
Pearl City Stream
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Pearl City Stream Crossing
Location Map
Scale -1:8,000
500 250 0 500 1,000 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
15-Apr-2010

Figure 6.1

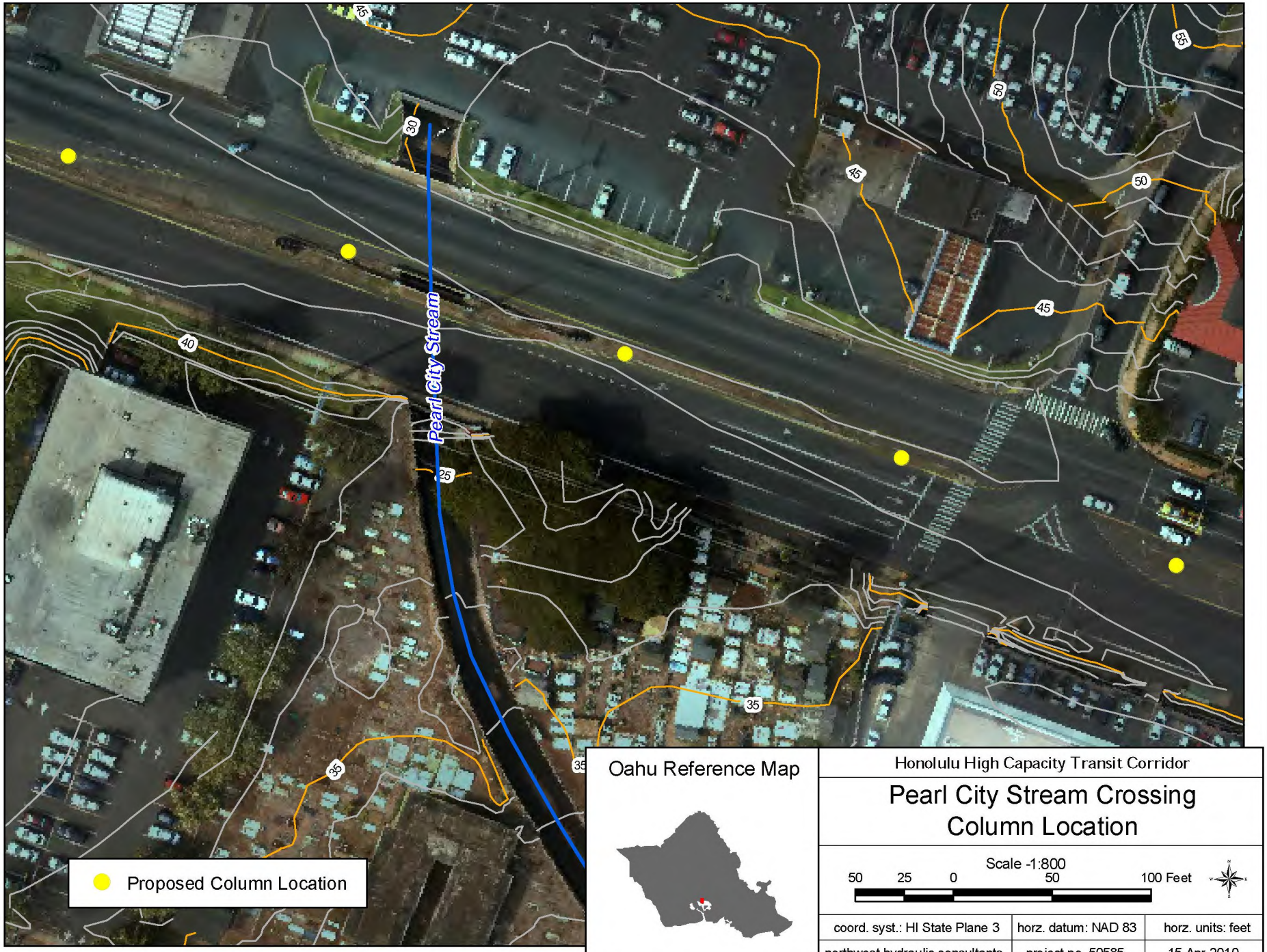
Pearl City Stream
Proposed Column Location
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Pearl City Stream Crossing
Column Location
Scale -1:800
50 25 0 50 100 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
15-Apr-2010

Figure 6.2

7 - Waiau Springs Crossing

Stream Crossing Description

The Waiau Springs Crossing is located in the Waimalu Watershed approximately 500-ft west of Neal S. Blaisdell Park (Figure 7.1). The proposed Honolulu High Capacity Transit Corridor Project (HHCTCP) elevated rail follows the median of the Kamehameha Highway and crosses Waiau Springs using a 150-ft span that is supported by 6-ft by 6-ft columns (Figure 7.2). At this location, the stream is contained within a series of concrete pipe culverts.

Representatives from NHC, PB and LAI visited the Waiau Springs Crossing location on September 23, 2009. Photos 7.1-7.6 depict the stream channel and proposed HHCTCP column locations near the stream crossing.

Hydrologic Analysis and FEMA Study

No hydrologic analysis was completed for this investigation. Waiau Stream does not have an effective Flood Insurance Study (FIS) or FEMA Flood Insurance Rate Map (FIRM).

Hydraulic Analysis

No hydraulic investigation was completed at this site because there is little chance that the crossing will have any measurable impact on flooding. As noted above, the crossing is supported on columns which are not located in the stream channel (Figure 7.2). Because a FEMA study has not been completed for Waiau stream we do not know if water will overtop the channel banks and cross the highway during a 100-year flood. If water does leave the channel, the contours shown in Figure 7.2 indicate that the water will flow east along the highway. There is nothing to confine the flow and thus it will move as shallow sheet flow. If water does reach the base of the HHCTCP columns, the depth would be relatively small and therefore the impact insignificant.

Regulatory Compliance

To our knowledge a flood study has not been completed for this reach of the Waiau Stream and therefore, we do not know if the columns are located within the limits of the flood fringe. Regardless, it is our opinion that the likely impact the columns would have on flooding is so inconsequential that detailed hydrologic and hydraulic investigations are not warranted. The columns themselves are not at risk because they will be supported on deep foundations so scour is not a concern. Also, anticipated flood depths and velocities in the flood fringe would not likely damage 6-ft by 6-ft square reinforced concrete columns.

AR00041174

Conclusions

The columns are setback from the stream channel and may or may not be located within the flood fringe. If water does reach the columns, the depth of flow will be shallow and therefore the impact to flooding will be insignificant and there will be no adverse impact on adjacent facilities or properties.

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Photos

Photo 7.1. Waiau Springs Crossing culvert outlet viewing east.

Photo 7.2. Standing on top of Waiau Springs culvert outlet viewing downstream channel.

AR00041176

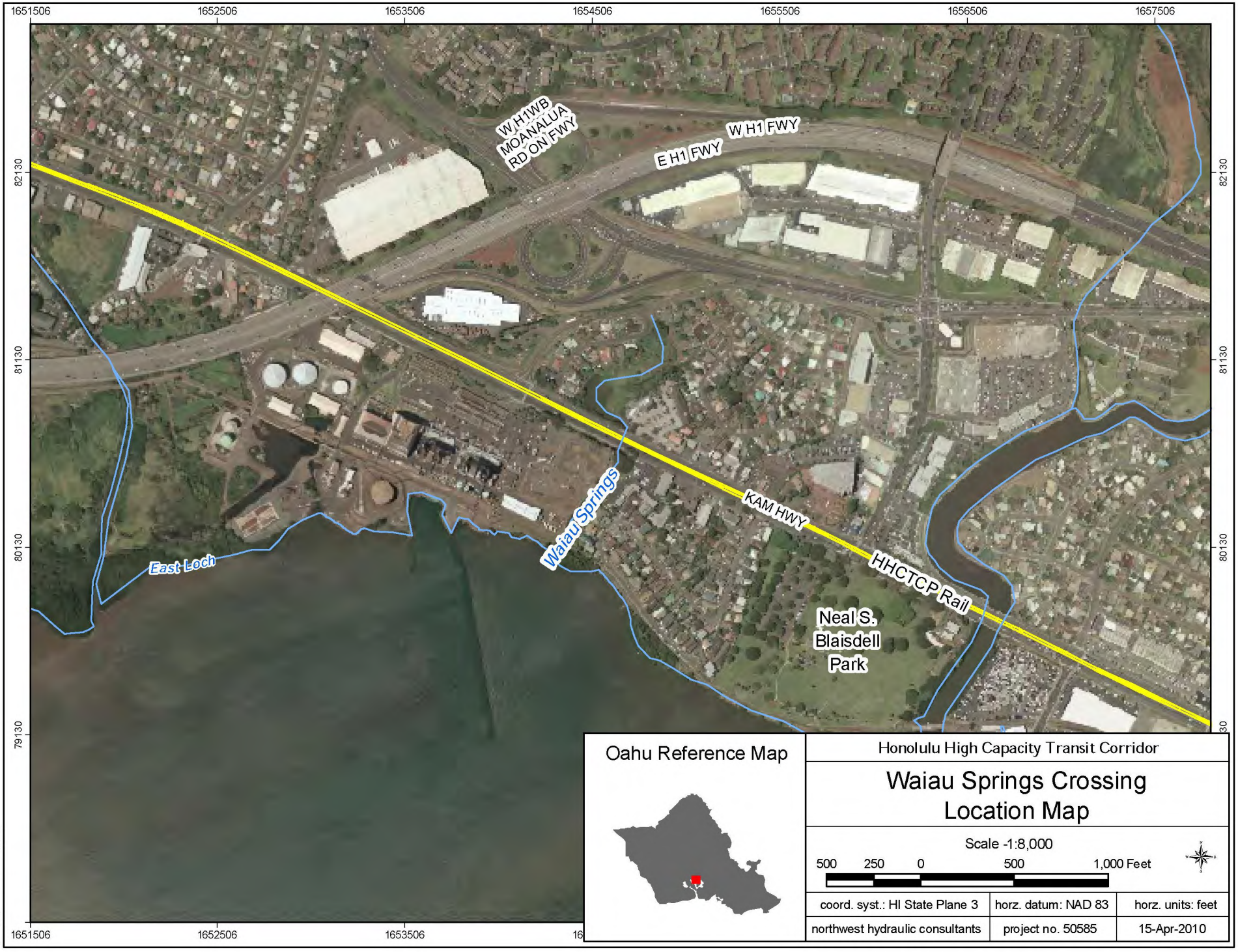
Photo 7.3. Upstream of Waiau Springs culvert inlet viewing downstream.

Photo 7.4. Standing on Waiau Springs Crossing viewing upstream channel/pond.

AR00041177

Photo 7.5. Viewing west along Kamehameha from just east of the Waiau Springs Crossing. NHC employees (in yellow and orange vests) standing at location of proposed HHCTCP columns.

Photo 7.6. Viewing east along Kamehameha Highway from just west of the Waiau Springs Crossing.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W H1WB
MOANALUA
RD ON FWY
W H1 FWY
E H1 FWY
KAM HWY
HHCTCP Rail
Waiau Springs
East Loch
Neal S.
Blaisdell
Park
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Waiau Springs Crossing
Location Map
Scale -1:8,000
500 250 0 500 1,000 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
15-Apr-2010

Figure 7.1

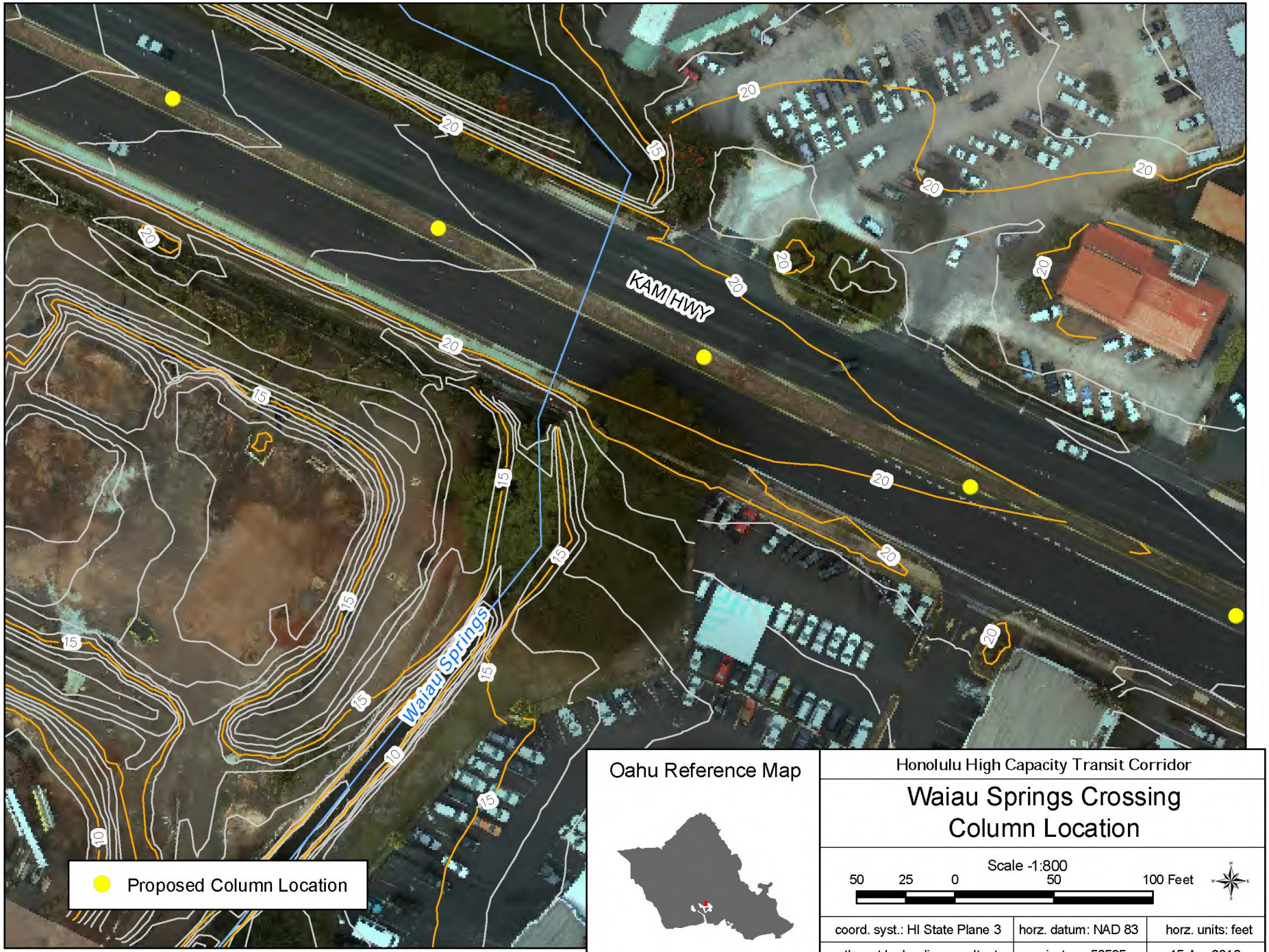

KAM HWY
Waiau Springs
Proposed Column Location
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Waiau Springs Crossing
Column Location
Scale -1:800
50 25 0 50 100 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
15-Apr-2010

Figure 7.2

8 - Waimalu Stream Crossing

Stream Crossing Description

Waimalu Stream is located southeast of the Waikele Stream Watershed, and empties into the East Loch of Pearl Harbor (see Figure 8.1). The proposed Honolulu High Capacity Transit Corridor Project (HHCTCP) elevated rail parallels the Kamehameha Highway and crosses the stream between the east and west bound highway bridges. The crossing consists of a single 180-ft span that is supported on columns that are setback from the stream channel (Figure 8.2). The columns are 6-ft by 6-ft square.

Representatives from NHC, PB and LAI visited the Waimalu Stream Crossing location on September 23, 2009. Photos 8.1-8.4 show the project site as it existed on the day of the visit.

Hydrologic Analysis and FEMA Study

No hydrologic analysis was completed for this investigation. Waimalu Stream does not have an effective Flood Insurance Study (FIS) or FEMA Flood Insurance Rate Map (FIRM).

Hydraulic Analysis

No hydraulic investigation was completed at this site because there is little chance that the crossing will have any measurable impact on flooding. As noted above, the guideway is supported on columns which are not located in the stream channel (Figure 8.2). Because a FEMA study has not been completed for Waimalu Stream we do not know if water will overtop the channel banks and cross the highway during a 100-year flood. This reach of the stream has been highly modified and enlarged to increase capacity for drainage and flood relief. Therefore, even during a major flood water may not leave the channel, or if it does, the amount will be relatively small. If water does leave the channel, the contours shown in Figure 8.2 indicate that it will bypass the bridge to the south east. If water flows around any of the columns, the depth would be relatively small and therefore the impact insignificant.

Regulatory Compliance

To our knowledge a flood study has not been completed for this reach of the Waimalu Stream and therefore, we do not know if the columns are located within the limits of the flood fringe. Regardless, it is our opinion that the likely impact the columns would have on flooding is so inconsequential that detailed hydrologic and hydraulic investigations are not warranted. The columns themselves are not at risk because they will be supported on deep foundations so scour is not a concern. Also, anticipated flood depths and velocities in the flood fringe would not likely damage 6-ft by 6-ft square reinforced concrete columns.

AR00041181

Conclusions

The columns are setback from the stream channel and may or may not be located within the flood fringe. The channel has been modified and enlarged to increase capacity, therefore, it is unlikely that significant flow will overtop the channel banks during the 100-year flood. However, if it does reach the columns, the impact to flooding will likely be insignificant and there will likely be no adverse impact on adjacent facilities or properties.

AR00041182

Photos

Photo 8.1. Upstream of the Kamehameha crossing viewing east.

Photo 8.2. Upstream Kamehameha crossing viewing east at bridge columns.

AR00041183

Photo 8.3. On Kamehameha Highway viewing east along the upstream side of the bridge.

Photo 8.4. On Kamehameha Highway viewing west along the upstream side of the bridge.

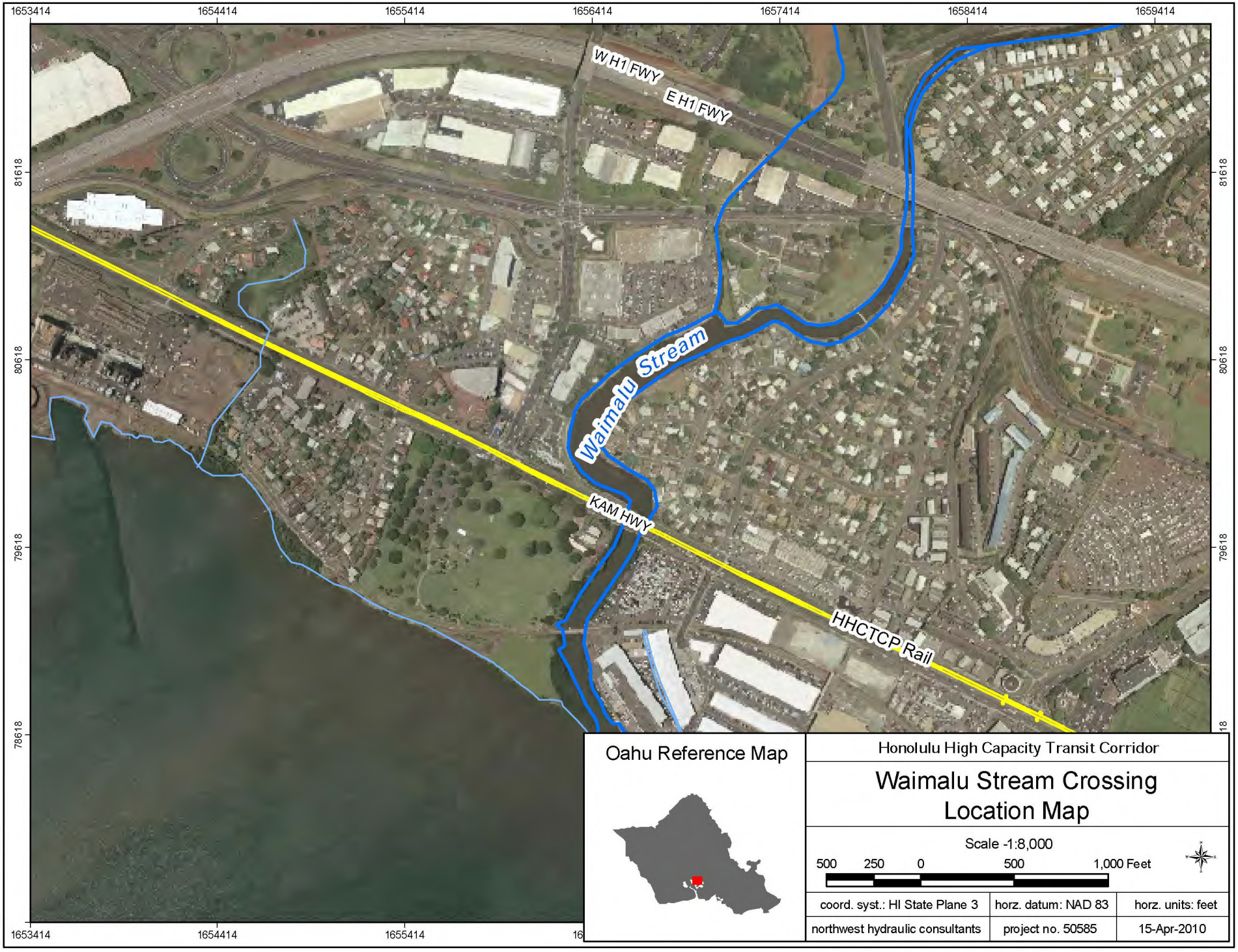


Figure 8.1

AR00041185

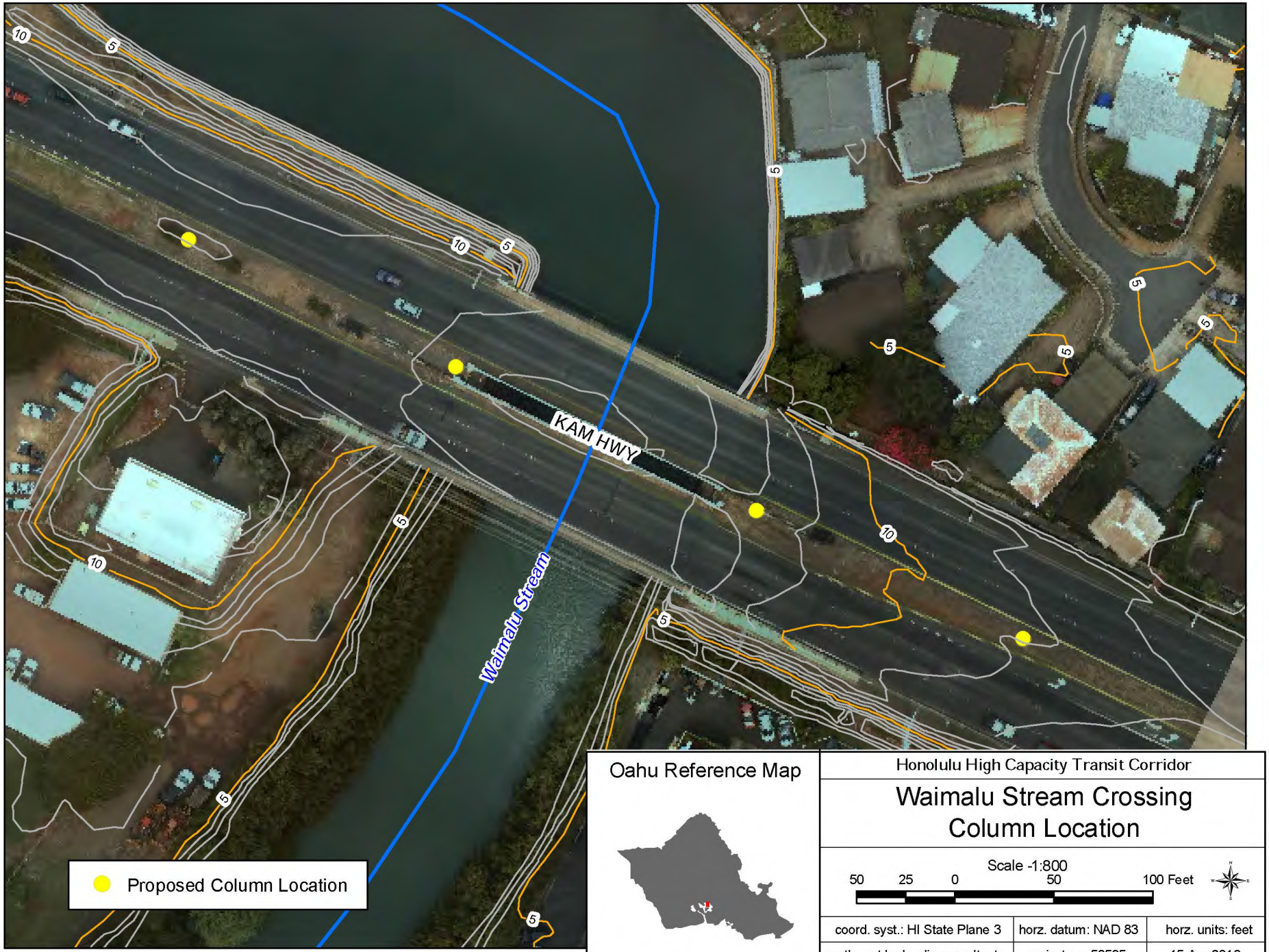

KAM HWY
Waimalu Stream
Proposed Column Location
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Waimalu Stream Crossing
Column Location
Scale -1:800
50 25 0 50 100 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
15-Apr-2010

Figure 8.2

9 - Kalauao Springs Crossing

Stream Crossing Description

The Kalauao Springs crossing is located south west of the Pearl Ridge Shopping Center, approximately 7 miles northwest of downtown Honolulu (See Figure 9.1). The lower portions of the Springs are contained within a concrete lined trapezoidal channel that flows into the East Loch of Pearl Harbor. The proposed Honolulu High Capacity Transit Corridor Project (HHCTCP) elevated rail parallels Kamehameha Highway and crosses the stream between the east and west bound highway bridges approximately 1000-ft upstream of the Pearl Harbor (Figure 9.2). The crossing consists of a single span over the channel supported on 6 by 6 foot columns.

Representatives from NHC, PB and LAI visited the Kalauao Springs Crossing location on September 23, 2009. Photos 9.1-9.5 depict the stream channel conditions on that date, as well as proposed HHCTCP column locations.

Hydrologic Analysis and FEMA Study

No hydrologic analysis was completed for this study. Kalauao Springs does not have an effective Flood Insurance Study (FIS) or FEMA Flood Insurance Rate Map (FIRM).

Hydraulic Analysis

No hydraulic investigation was completed at this site because there is little chance that the crossing will have any measurable impact on flooding. As noted above, the guideway is supported on columns which are not located in the stream channel (Figure 9.2). Because a FEMA study has not been completed for Kalauao Springs we do not know if water will overtop the channel banks and cross the highway during a 100-year flood. This reach of the stream has been highly modified and enlarged to increase capacity for drainage and flood relief. Therefore, even during a major flood water may not leave the channel, or if it does, the amount will be relatively small.

Regulatory Compliance

To our knowledge a flood study has not been completed for this reach of the Kalauao Springs and therefore, we do not know if the columns are located within the limits of the flood fringe. Regardless, it is our opinion that the likely impact the columns would have on flooding is so inconsequential that detailed hydrologic and hydraulic investigations are not warranted. The columns themselves are not at risk because they will be supported on deep foundations so scour is not a concern. Also, anticipated flood depths and velocities in the flood fringe would not likely damage 6-ft by 6-ft reinforced concrete columns.

AR00041187

Conclusions

The columns are setback from the stream channel and may or may not be located within the flood fringe. The channel has been modified and enlarged to increase capacity, therefore, it is unlikely that significant flow will overtop the channel banks during the 100-year flood. However, if it does reach the columns, the impact to flooding will be insignificant and there will be no adverse impact on adjacent facilities or properties.

AR00041188

Photos

Photo 9.1. Upstream of Kalauao Springs Crossing viewing downstream.

Photo 9.2. Upstream of Kalauao Springs Crossing viewing downstream.

Photo 9.3. On Kamehameha Highway viewing upstream along Kalauao Springs.

Photo 9.4. On Kamehameha Highway viewing west from upstream face of Kalauao Springs Crossing. NHC employee (in orange vest) standing at proposed HHCTCP column location.

AR00041190

Photo 9.5. On Kamehameha Highway, upstream of Kalauao Springs Crossing, viewing south east. NHC employee (in yellow vest) standing at proposed HHCTCP column location.

AR00041191

1656600
1657600
1658600
1659600
1660600
1661600
1662600
79959
78959
77959
76959
E H1 FWY
W H1 FWY
Pearl Ridge
Shopping Center
KAM HWY
Kalauao Springs
East Loch
HHCTCP Rail
Kalauao Stream
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Kalauao Springs Crossing
Location Map
Scale -1:8,000
500 250 0 500 1,000 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
15-Apr-2010

Figure 9.1

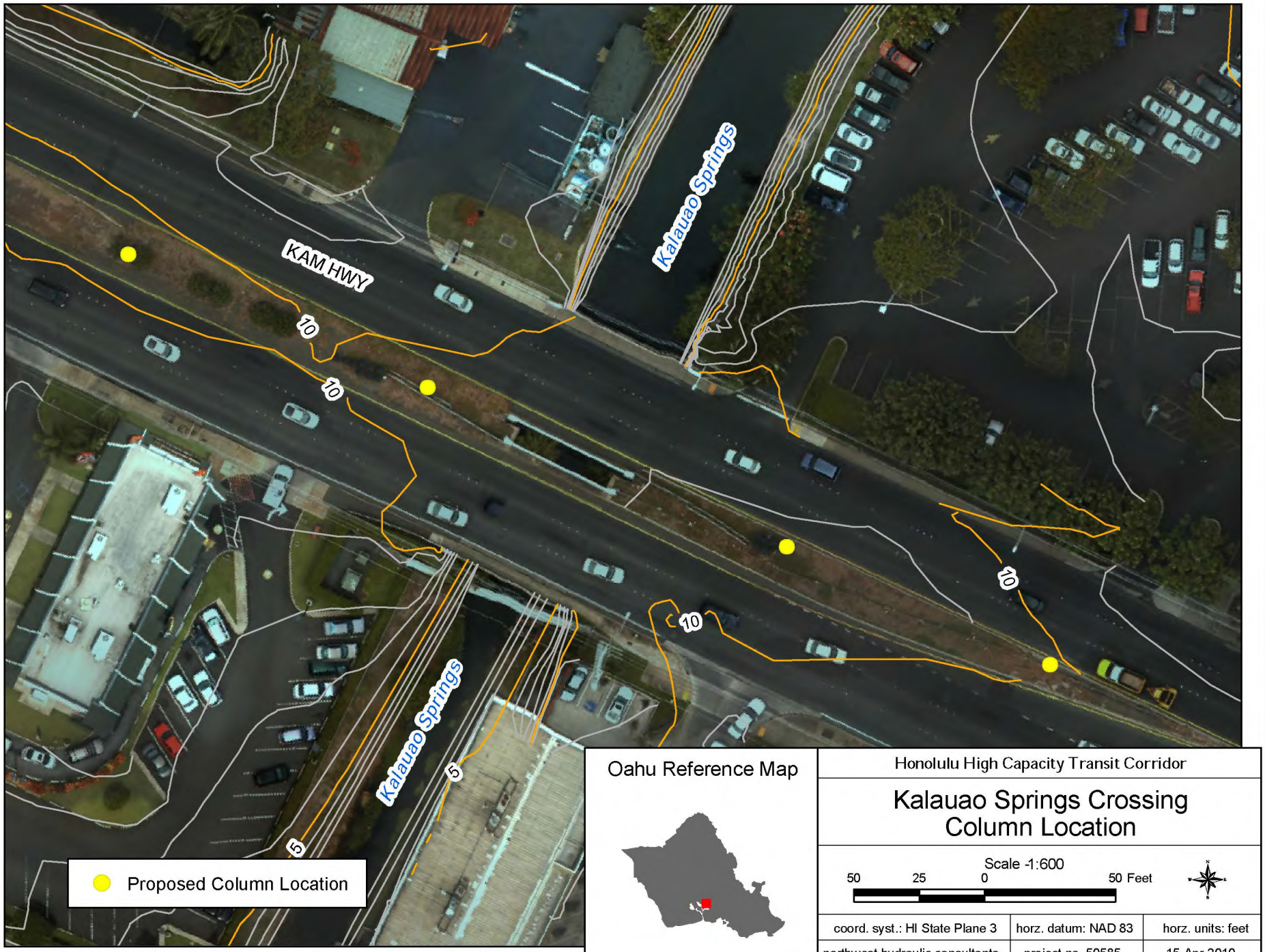

KAM HWY
Kalauao Springs
Kalauao Springs
10
10
10
10
5
5
Proposed Column Location
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Kalauao Springs Crossing
Column Location
Scale -1:600
50 25 0 50 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
15-Apr-2010

Figure 9.2

10 - Kalauao Stream Crossing

Stream Crossing Description

The Kalauao Stream crossing is located south east of the Pearl Ridge Shopping Center, approximately 7 miles northwest of downtown Honolulu (See Figure 10.1). The proposed Honolulu High Capacity Transit Corridor Project (HHCTCP) elevated track parallels Kamehameha Highway and spans Kalauao Stream on columns running between the east and west bound lanes of the Highway Bridge (Figure 10.2). All columns are 6-ft by 6-ft square.

Representatives from NHC, PB and LAI visited the Kalauao Stream Crossing location on September 23, 2009. Photos 10.1-10.4 depict the stream channel and proposed HHCTCP column locations near the stream crossing.

Hydrologic Analysis and FEMA Study

A detailed FEMA Flood Insurance Study (FIS) has been completed for this reach of Kalauao Stream. Hydrology in the effective FIS was completed using discharge records from the USGS Gaging Station No. 16224500 at Moanalua Road, at Aiea. A log-Pearson Type III analysis determined that the peak 100 year discharge of 3400 cfs and 3540 cfs at the Kamehameha Highway, and Ocean respectively. The FEMA FIS map at the crossing is shown in Figure 10.3. It shows that the 100-year floodplain and floodway are quite narrow at the crossing, but do surround one track column.

Hydraulic Analysis

The existing FIS, site inspection, new LiDAR contours, and aerial photos were used to examine flood hazard extents, proposed column locations, and potential flood impacts.

Although columns are located outside of physical channel banks, one column is located within the effective FEMA Floodplain and Floodway (See Figure 10.3 and Photo 10.4). New LiDAR data shows that the ground elevation at the column location is slightly above 13-ft (see Figure 10.2). The Base Flood Elevations at the crossing varies from about 12-feet at the upstream side to 6-feet at the downstream side with an elevation of 10 feet between the two highway bridges at the location of the proposed column (Figure 10.3). Therefore, the column will not come into contact with 100-year flood waters and therefore will cause "No-Rise".

Regulatory Compliance and Conclusions

The HHCTCP will have no impact on flood levels within Kalauao Stream. One column is located within the FEMA regulatory Floodway, however, it is on existing high ground that will remain dry during the 100-year flood. Therefore, the project complies with the City's "No-Rise" requirement.

AR00041194

Photos

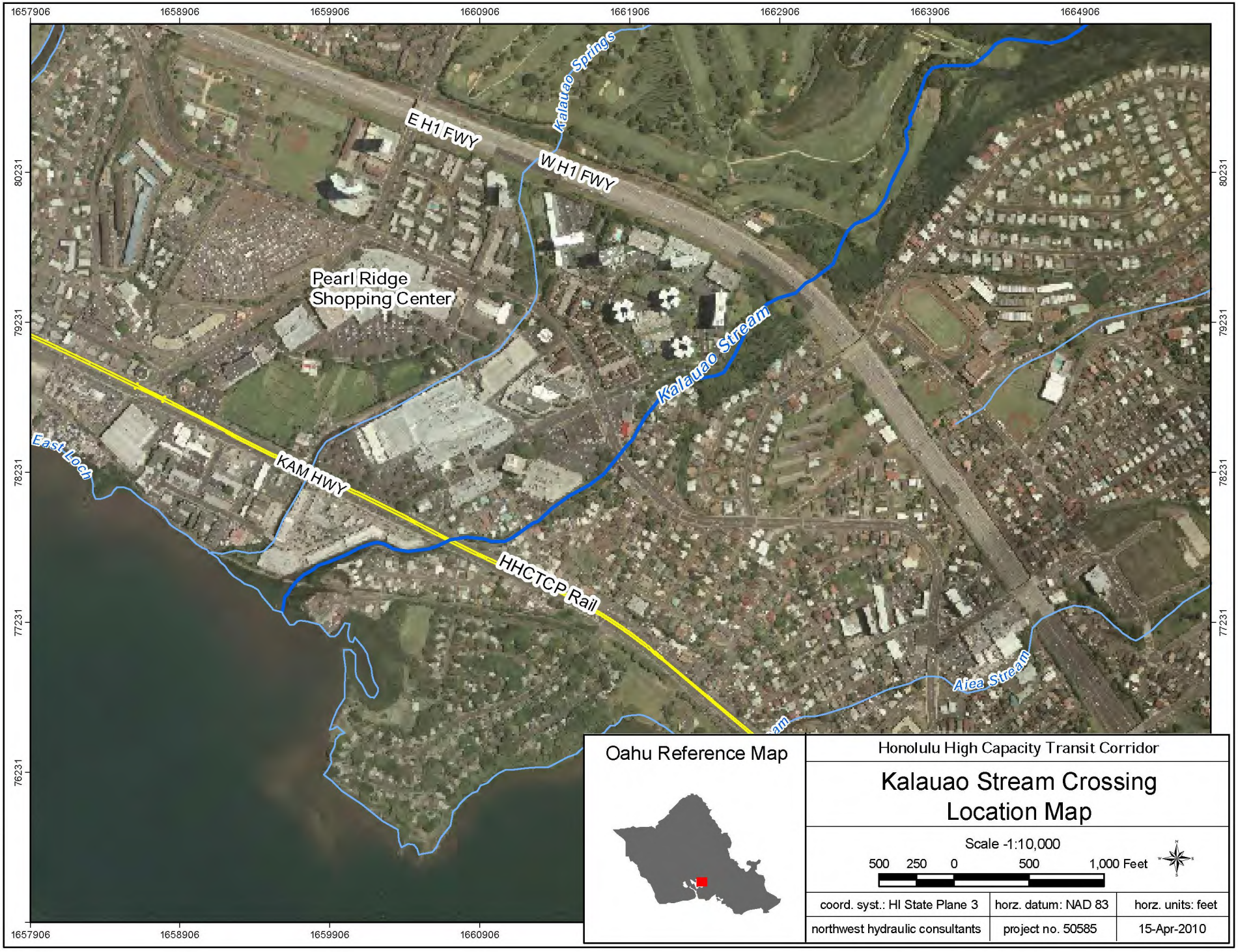
Photo 10.1. Upstream of Kalauao Crossing (Kamehameha Highway) viewing downstream

Photo 10.2. On Kamehameha Highway viewing upstream.

AR00041195

Photo 10.3. Viewing downstream under the Kamehameha Highway crossing.

Photo 10.4. Viewing downstream at the Kamehameha Highway crossing. NHC employee (in yellow vest) is standing at location of proposed HHCTCP column to be located in current FEMA Floodway.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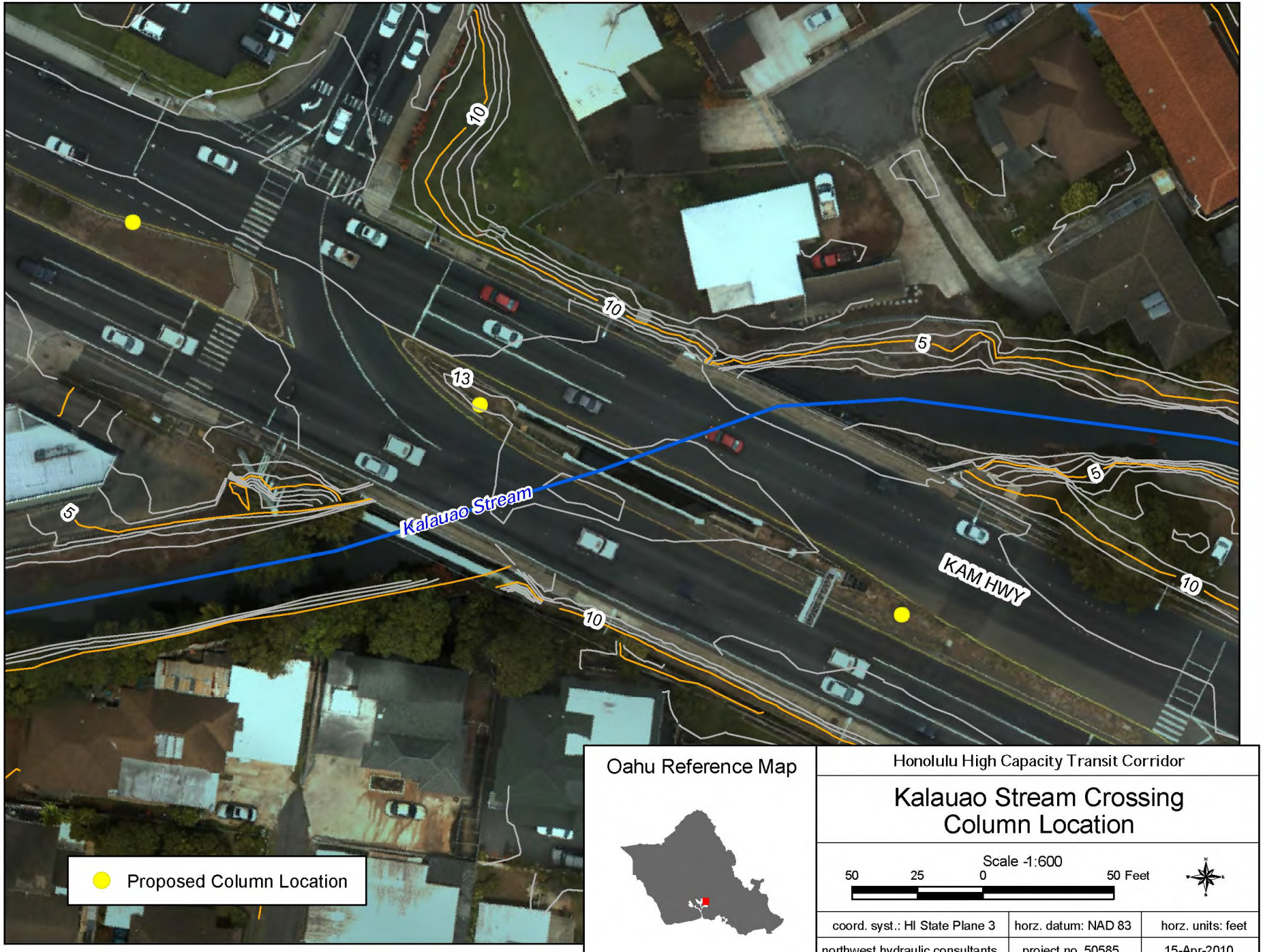
77231
76231
E H1 FWY
W H1 FWY
Kalauao Springs
Pearl Ridge
Shopping Center
Kalauao Stream
East Loch
KAM HWY
HHCTCP Rail
Aiea Stream
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Kalauao Stream Crossing
Location Map
Scale -1:10,000
500 250 0 500 1,000 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
15-Apr-2010

Figure 10.1



Honolulu High Capacity Transit Corridor

Kalauao Stream Crossing Column Location

Scale -1:600

50 25 0 50 Feet

coord. syst.: HI State Plane 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50585	15-Apr-2010

Figure 10.2

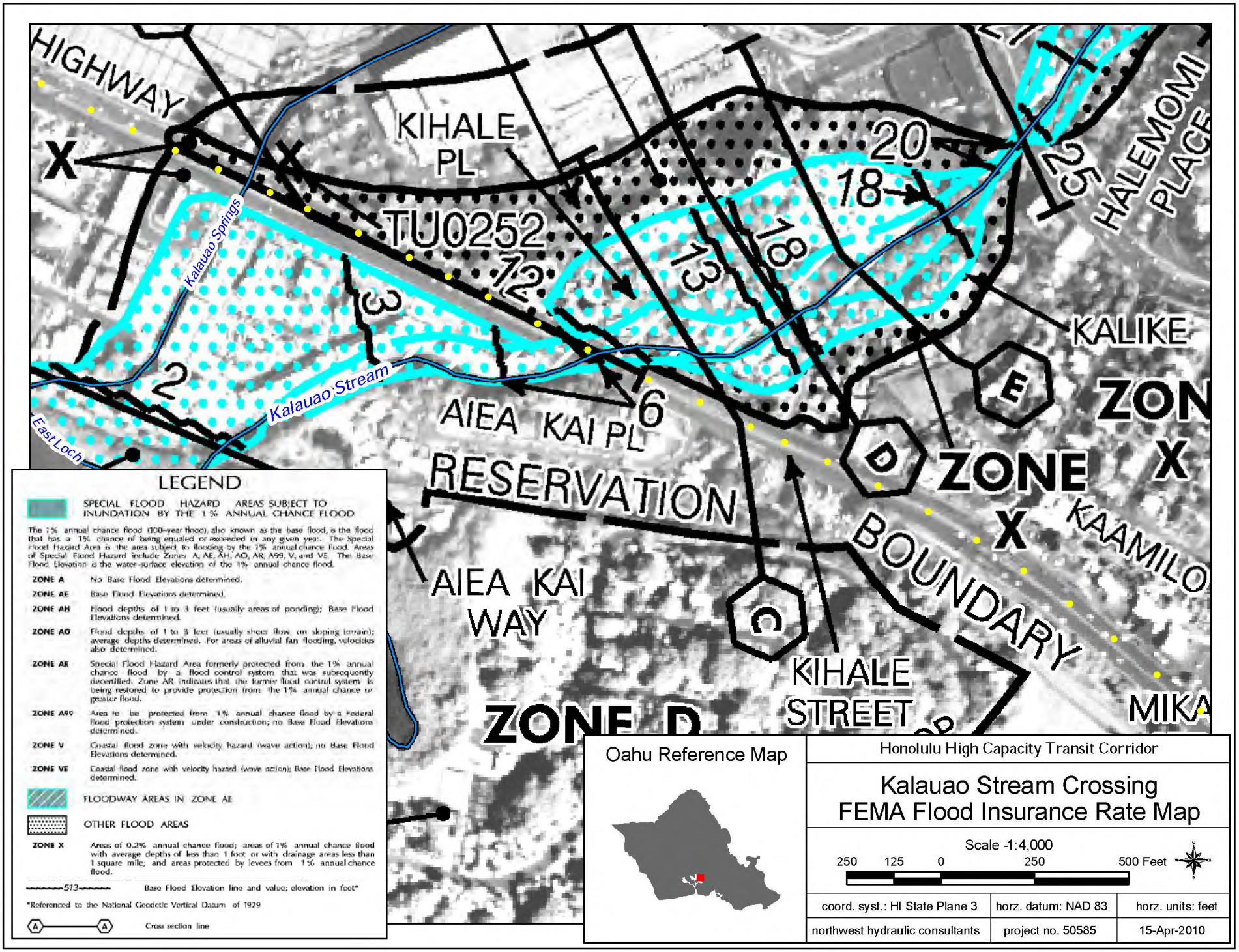

HIGHWAY
KIHALE PL
TU0252
Kalauao Springs
Kalauao Stream
East Loch
AIEA KAI PL
RESERVATION
AIEA KAI WAY
KIHALE STREET
ZONE D
ZONE X
KALIKE
HALEMOMI PLACE
KAAMILO
BOUNDARY
MIKA
20
18
25
13
18
12
3
2
6
LEGEND
SPECIAL FLOOD HAZARD AREAS SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD
The 1% annual chance flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V, and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.
ZONE A No Base Flood Elevations determined.
ZONE AE Base Flood Elevations determined.
ZONE AH Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.
ZONE AO Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
ZONE AR Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
ZONE A99 Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.
ZONE V Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.
ZONE VE Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.
FLOODWAY AREAS IN ZONE AE
OTHER FLOOD AREAS
ZONE X Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.
513 Base Flood Elevation line and value; elevation in feet*
*Referenced to the National Geodetic Vertical Datum of 1929
A A Cross section line
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Kalauao Stream Crossing
FEMA Flood Insurance Rate Map
Scale -1:4,000
250 125 0 250 500 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
15-Apr-2010

Figure 10.3

11 - Aiea Stream Crossing

Stream Crossing Description

The Aiea Stream Crossing is located north of Aloha Stadium, near the outlet of the stream into Aiea Bay in the East Loch of Pearl Harbor (see Figure 11.1). The proposed Honolulu High Capacity Transit Corridor Project (HHCTCP) elevated rail parallels Kamehameha Highway, and crosses Aiea Stream on a series of 6-ft by 6-ft columns along the Highway median (Figure 11.2). Representatives from NHC, PB and LAI visited the crossing location on September 23, 2009. Photos 11.1-11.4 depict the stream channel and proposed HHCTCP column locations.

Hydrologic Analysis and FEMA Study

Flood hazards on Aiea Stream were analyzed as part of the effective Flood Insurance Study (FIS) for Honolulu, HI. Hydrology in the effective FIS was completed using USGS regression equations, and found the peak 100-year discharge at Aiea Heights to be 2140 cfs. A FEMA Zone A (100 year flooding, approximate study) is delineated from Pearl Harbor to the upstream side of the Kamehameha Highway Bridge, with a Zone X (500 year flooding) extending upstream of the crossing in the vicinity of Ieie Place (Figure 11.3). This delineation, though approximate, indicates that the 100-year flood boundary at the Kamehameha Highway Bridge is confined to the width of the highway bridge. This suggests that flood waters will not overtop the highway and therefore will not interact with the columns.

Hydraulic Analysis

A detailed hydraulic analysis has not been completed for this crossing, however, it is our opinion that one is not needed. Based upon the existing FIS flood boundaries, it appears all HHCTCP structures are located outside of the FEMA flood hazard zone (see Figure 11.3), and will therefore have no impact on 100 year flood levels. Even if flood waters were to overtop the highway and reach the columns, flow depths would be so shallow that the columns would have no measurable impact on flood levels.

Regulatory Compliance and Conclusions

All HHCTCP structures appear to be located outside of Flood Hazard Districts, therefore, there should be no Flood Hazard District concerns at this crossing.

AR00041200

Photos

Photo 11.1. Viewing upstream to Kamehameha Highway crossing and Aiea Stream from pedestrian bridge.

Photo 11.2. On Kamehameha Crossing viewing downstream towards pedestrian bridge and Pearl Harbor.

AR00041201

Photo 11.3. Viewing southeast along Kamehameha Highway at the Aiea Stream Crossing. NHC employee (in orange vest, near truck) standing at proposed column location.

Photo 11.4. Viewing northwest along Kamehameha Highway at the Aiea Stream Crossing. NHC employee (in orange and yellow vest) standing at proposed HHCTCP column location.

Figure 11.1

AR00041203

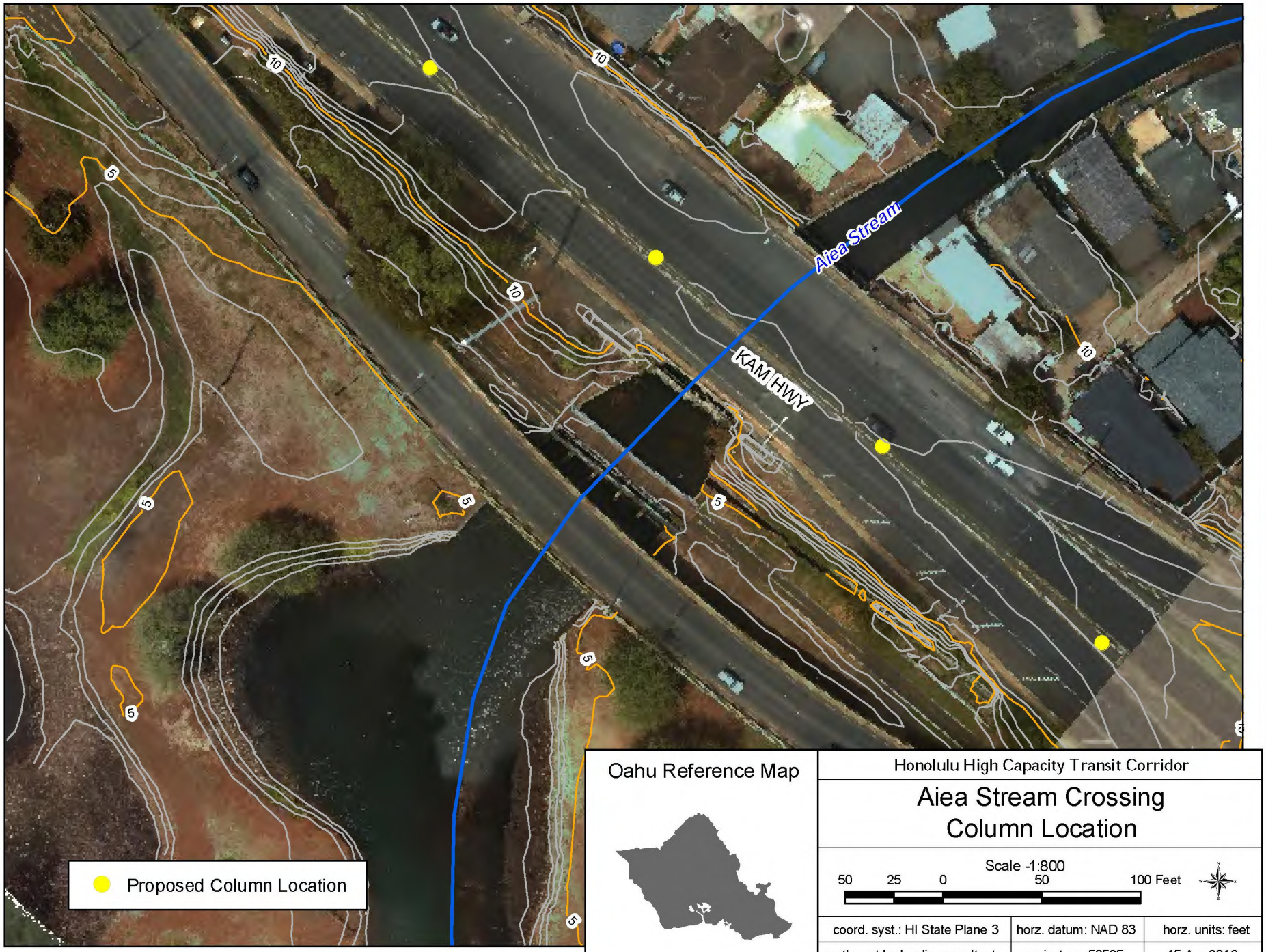

Aiea Stream
KAM HWY
10
5
Proposed Column Location
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Aiea Stream Crossing
Column Location
Scale -1:800
50 25 0 50 100 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
15-Apr-2010

Figure 11.2

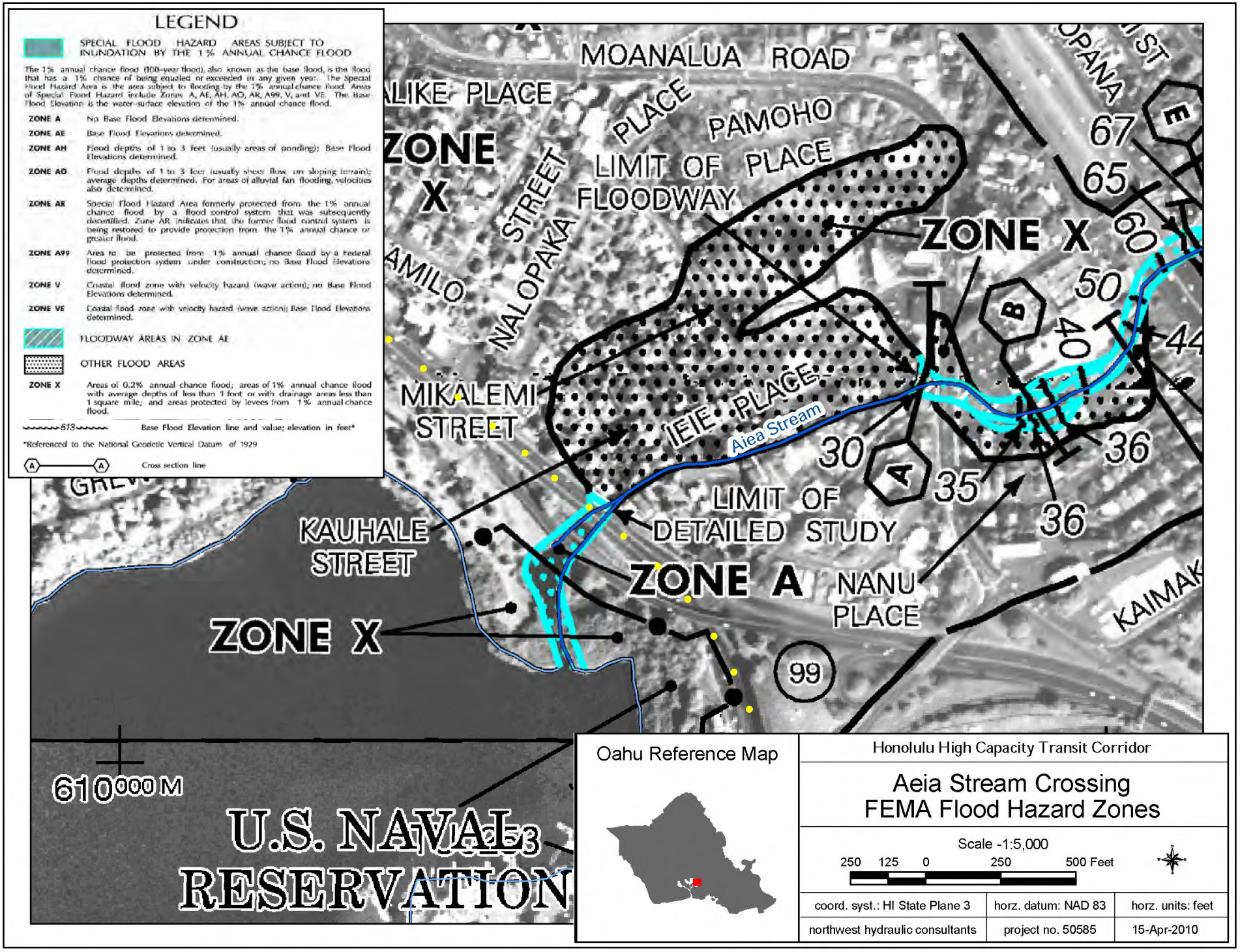

LEGEND
SPECIAL FLOOD HAZARD AREAS SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD
The 1% annual chance flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V, and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.
ZONE A No Base Flood Elevations determined.
ZONE AE Base Flood Elevations determined.
ZONE AH Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.
ZONE AO Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
ZONE AR Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
ZONE A99 Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.
ZONE V Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.
ZONE VE Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.
FLOODWAY AREAS IN ZONE AE
OTHER FLOOD AREAS
ZONE X Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.
513 Base Flood Elevation line and value; elevation in feet*
*Referenced to the National Geodetic Vertical Datum of 1929
A A Cross section line
MOANALUA ROAD
PAMOHO PLACE
LIMIT OF FLOODWAY
ZONE X
NALOPAKA STREET
MIKALEMI STREET
IEIE PLACE
Aiea Stream
LIMIT OF DETAILED STUDY
ZONE A
NANU PLACE
KAUHALE STREET
ZONE X
KAIMAK
99
610000 M
U.S. NAVAL RESERVATION
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Aeia Stream Crossing
FEMA Flood Hazard Zones
Scale -1:5,000
250 125 0 250 500 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
15-Apr-2010

Figure 11.3

12 - Halawa Stream Crossing

Stream Crossing Description

The Halawa Stream crossing is located approximately 6 miles northwest of downtown Honolulu near the US Arizona Memorial at the East Loch of Pearl Harbor (Figure 12.1). The proposed Honolulu High Capacity Transit Corridor Project (HHCTCP) elevated rail parallels the Kamehameha Highway and crosses the stream between the north and south bound highway bridges. The crossing consists of a single 310-ft long bridge that is supported on columns that are setback from the stream channel (Figure 12.2). The columns are 8-ft by 8-ft square.

Representatives from NHC, PB and LAI visited the Halawa Stream Crossing location on September 23, 2009. Photos 12.1 to 12.5 show the project site as it existed on the day of the visit. Note, the southbound Kamehameha bridge was under construction at the time of the visit and therefore southbound traffic was accommodated by a temporary Bailey type bridge which is visible in several of the photographs.

Hydrologic Analysis and FEMA Study

No hydrologic analysis was completed for this investigation. Halawa Stream does not have an effective Flood Insurance Study (FIS) or FEMA Flood Insurance Rate Map (FIRM).

Hydraulic Analysis

No hydraulic investigation was completed at this site because there is little chance that the crossing will have any measurable impact on flooding. As noted above, the bridge is supported on two columns which are not located in the stream channel (Figure 12.2). Because a FEMA study has not been completed for Halawa stream we do not know if water will overtop the channel banks and cross the highway during a 100-year flood. This reach of the stream has been highly modified and enlarged to increase capacity for drainage and flood relief. Therefore, even during a major flood water may not leave the channel, or if it does, the amount will be relatively small. If water does leave the channel, the contours shown in Figure 12.2 indicate that it will bypass the bridge to the south, possibly missing the south column. However, even if water flows around the columns the depth would be relatively small and therefore the impact insignificant.

Regulatory Compliance

To our knowledge a flood study has not been completed for this reach of the Halawa Stream and therefore, we do not know if the columns are located within the limits of the flood fringe. Regardless, it is our opinion that the likely impact the columns would have on flooding is so inconsequential that detailed hydrologic and hydraulic investigations are not warranted. The columns themselves are not at

AR00041206

risk because they will be supported on deep foundations so scour is not a concern. Also, anticipated flood depths and velocities in the flood fringe would not likely damage 6-ft by 6-ft square reinforced concrete columns.

Conclusion

The columns are setback from the stream channel and may or may not be located within the flood fringe. The channel has been modified and enlarged to increase capacity, therefore, it is unlikely that significant flow will overtop the channel banks during the 100-year flood. However, if it does reach the columns, the impact to flooding will be insignificant and there will be no adverse impact on adjacent facilities or properties.

AR00041207

Photos

Photo 12.1. Upstream, on right bank, of Halawa Stream viewing downstream.

Photo 12.2. Upstream of the Halawa Stream crossing viewing downstream.

AR00041208

Photo 12.3. On Kamehameha Highway viewing south from upstream face of Halawa Stream crossing. Proposed column locations are marked with red arrows.

Photo 12.4. On Kamehameha Highway viewing south from upstream face of Halawa Stream crossing.

AR00041209

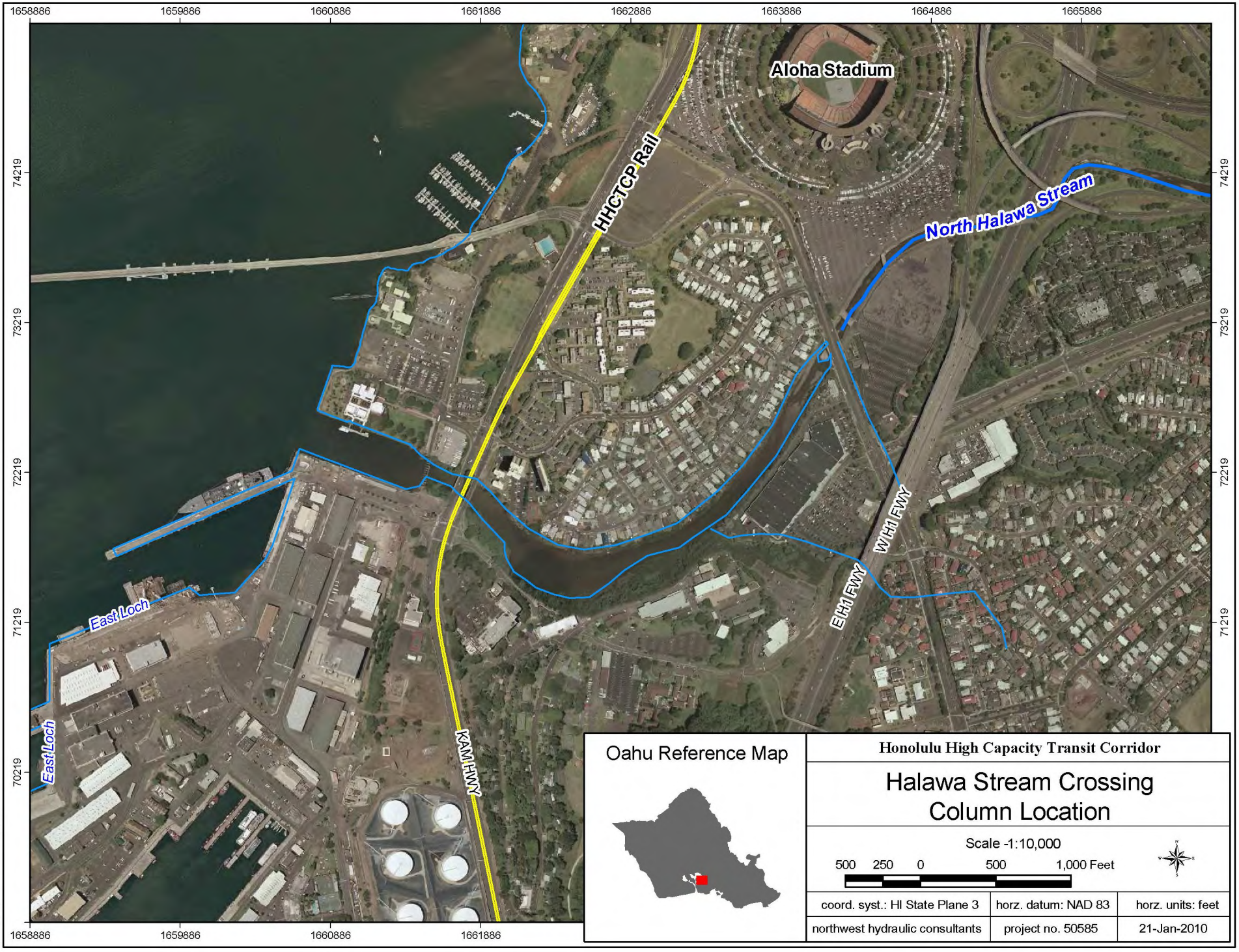


Figure 12.1

AR00041210

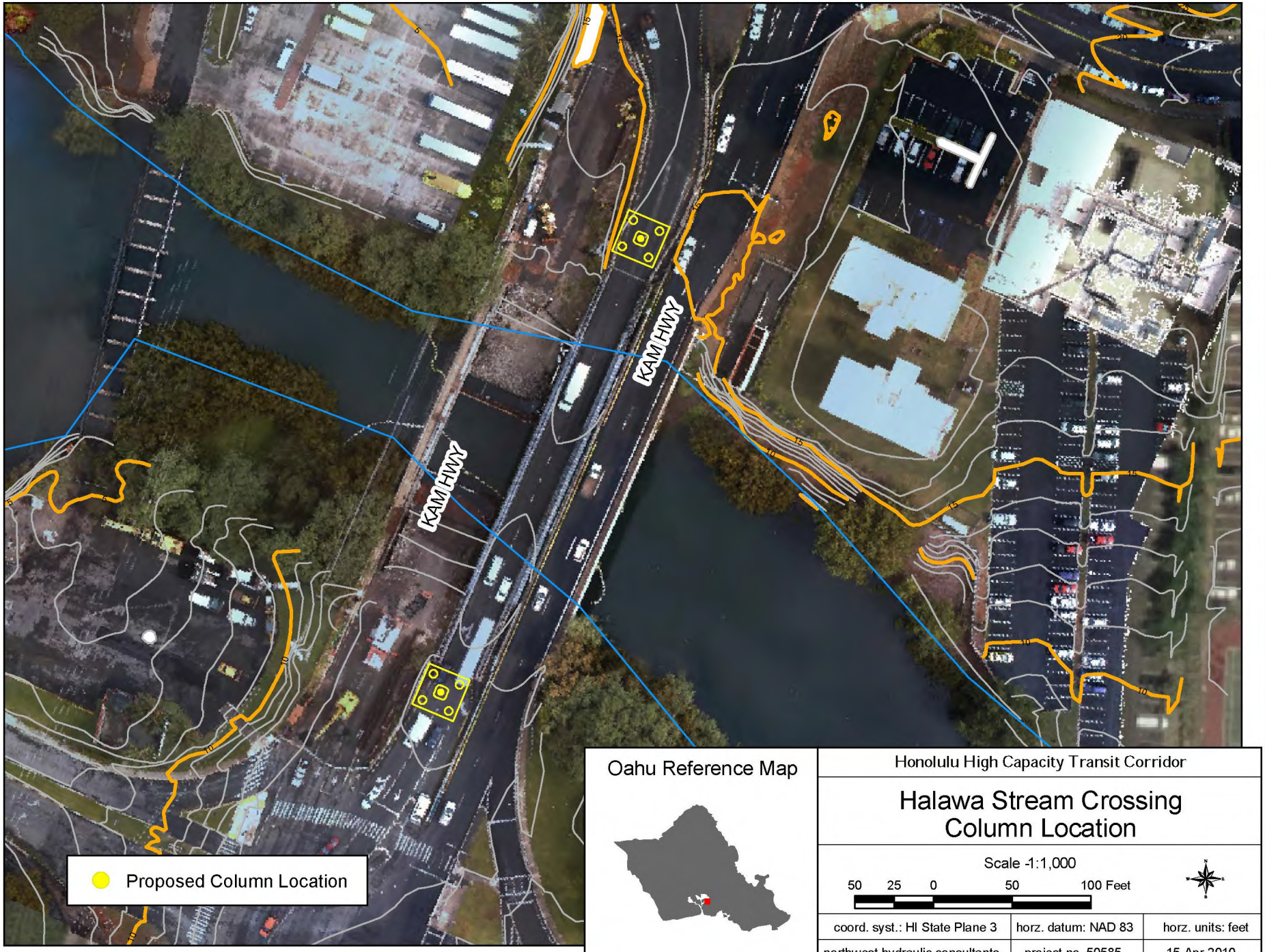

KAM HWY
KAM HWY
Proposed Column Location
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Halawa Stream Crossing
Column Location
Scale -1:1,000
50 25 0 50 100 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
15-Apr-2010

Figure 12.2

13 - Aolele Ditch Crossing

Stream Crossing Description

The Aolele Ditch crossing is located less than half a mile east of the Honolulu International Airport terminal (Figure 13.1). Aolele Ditch is a manmade grass-lined channel that conveys local drainage east along Aolele Street before discharging into the ocean at Keehi Lagoon. The proposed Honolulu High Capacity Transit Corridor Project (HHCTCP) elevated rail (guideway) parallels Aolele Ditch, and then turns and crosses the channel upstream of the Aolele Street bridge before being aligned with Ualena Street. The guideway is supported by 6-ft square concrete columns located outside the channel above the top of the bank. Two spanning concrete supports would be used to support the guideway where it crosses the ditch, such that no columns would be located within the channel.

Representatives from NHC, PB and LAI visited the Aolele Ditch Crossing location on September 23, 2009. Photographs 13.1 and 13.2 show the project site along the main Aolele Ditch as it existed on the day of the visit.

Hydrologic Analysis and FEMA Study

No hydrologic analysis was completed for this investigation. Aolele Ditch does not have an effective Flood Insurance Study (FIS). Aolele Ditch provides drainage to a developed area of approximately 1200 acres based on the State of Hawaii's general watershed delineations; though the exact extent and drainage network of the urbanized basin is unknown.

Hydraulic Analysis

No hydraulic investigation was completed at this site because there is little chance that the crossing will have any measurable impact on flooding. As noted above, the crossing is supported on columns that are not located in the stream channel. Because a FEMA study has not been completed for Aolele Ditch we do not know if water will overtop the channel banks and cross the highway during a 100-year flood. If water does leave the channel, it would spread out on both sides of the channel and flow east. If water does reach the base of the HHCTCP columns, the depth would be relatively small and therefore the impact on water surface elevations would likely be insignificant.

Regulatory Compliance

To our knowledge a flood study has not been completed for Aolele Ditch and, based on the effective FEMA FIRM (Figure 13.2), the columns are not located within the limits of a Flood Hazard District. Since no flood study has been completed we do not know if the columns are located within the Aolele Ditch floodplain. Regardless, it is our opinion that the likely impact the columns would have on flooding is insignificant and detailed hydrologic and hydraulic investigations are not warranted. The columns

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themselves are not at risk because they will be supported on deep foundations so scour is not a concern. Also, since the columns are located outside the channel above the top of the channel banks the flood depths and velocities would not likely damage the 6-ft square reinforced concrete columns.

The "Wetland Waters of the US Study HHCTCP" (PB, July 2009) report identifies both ordinary high water (OHW) and in-channel wetlands for the main Aolele Ditch. Both OHW and in-channel wetlands are limited to the bottom of Aolele Ditch channel, from toe of bank to toe of bank. The columns would clearly be located outside of any of the delineated wetlands or OHW since the columns are located above the channel's top of bank.

Conclusion

The columns are outside the stream channel above the top of the banks and may or may not be located within the floodplain. If water does reach the columns, the depth of flow will likely be shallow and therefore the impact to flooding will be insignificant. We do not recommend further mitigation since the columns have been located outside of the Aolele Ditch channel.

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Photos

Photo 13.1. Viewing downstream along Aolele Ditch at the Aolele Street crossing.

Photo 13.2. Viewing upstream along Aolele Ditch towards the upstream end of the ditch.

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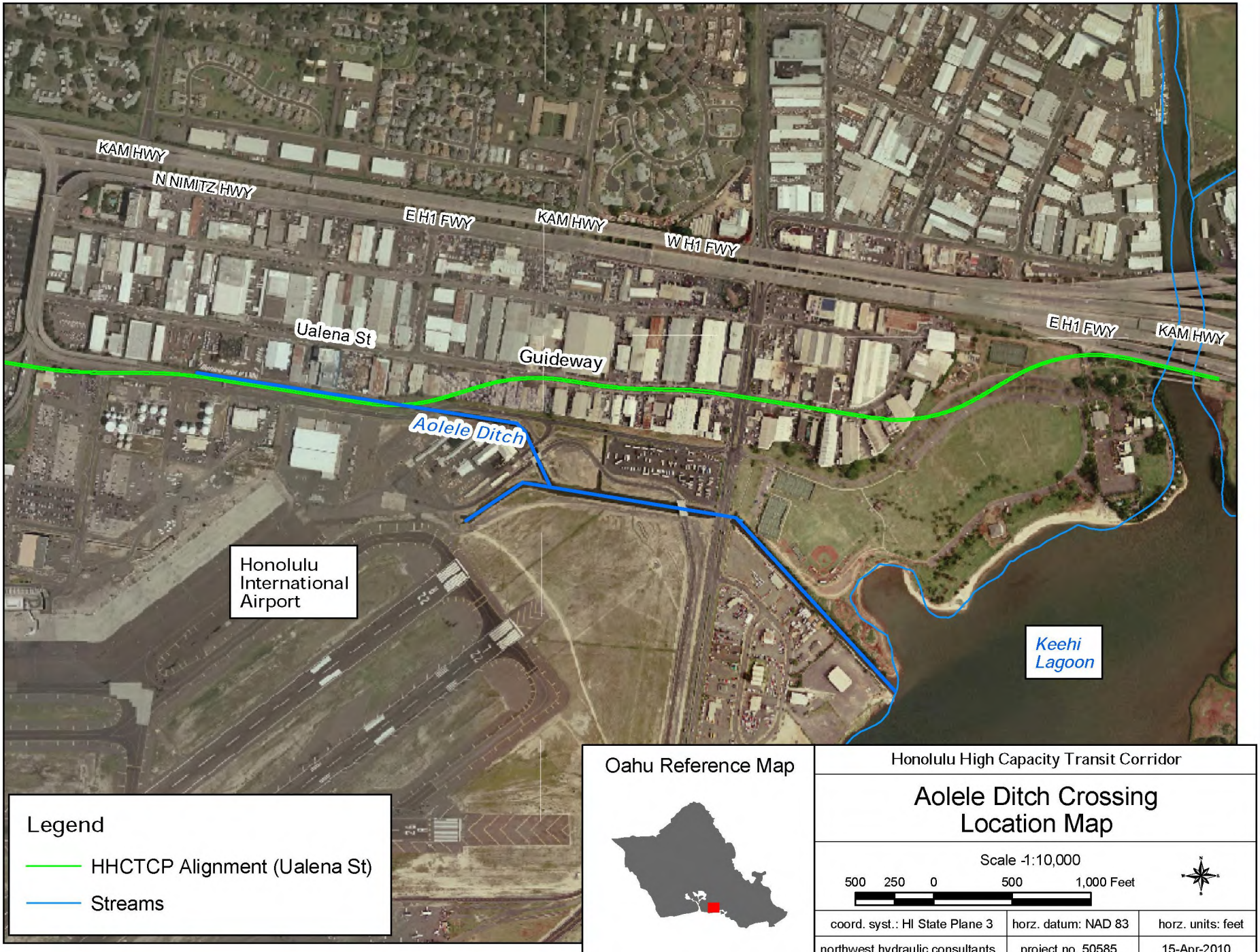
KAM HWY
N NIMITZ HWY
E H1 FWY
KAM HWY
W H1 FWY
Ualena St
Guideway
E H1 FWY
KAM HWY
Aolele Ditch
Honolulu International Airport
Keehi Lagoon
Legend
HHCTCP Alignment (Ualena St)
Streams
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Aolele Ditch Crossing Location Map
Scale -1:10,000
500 250 0 500 1,000 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
15-Apr-2010

Figure 13.1

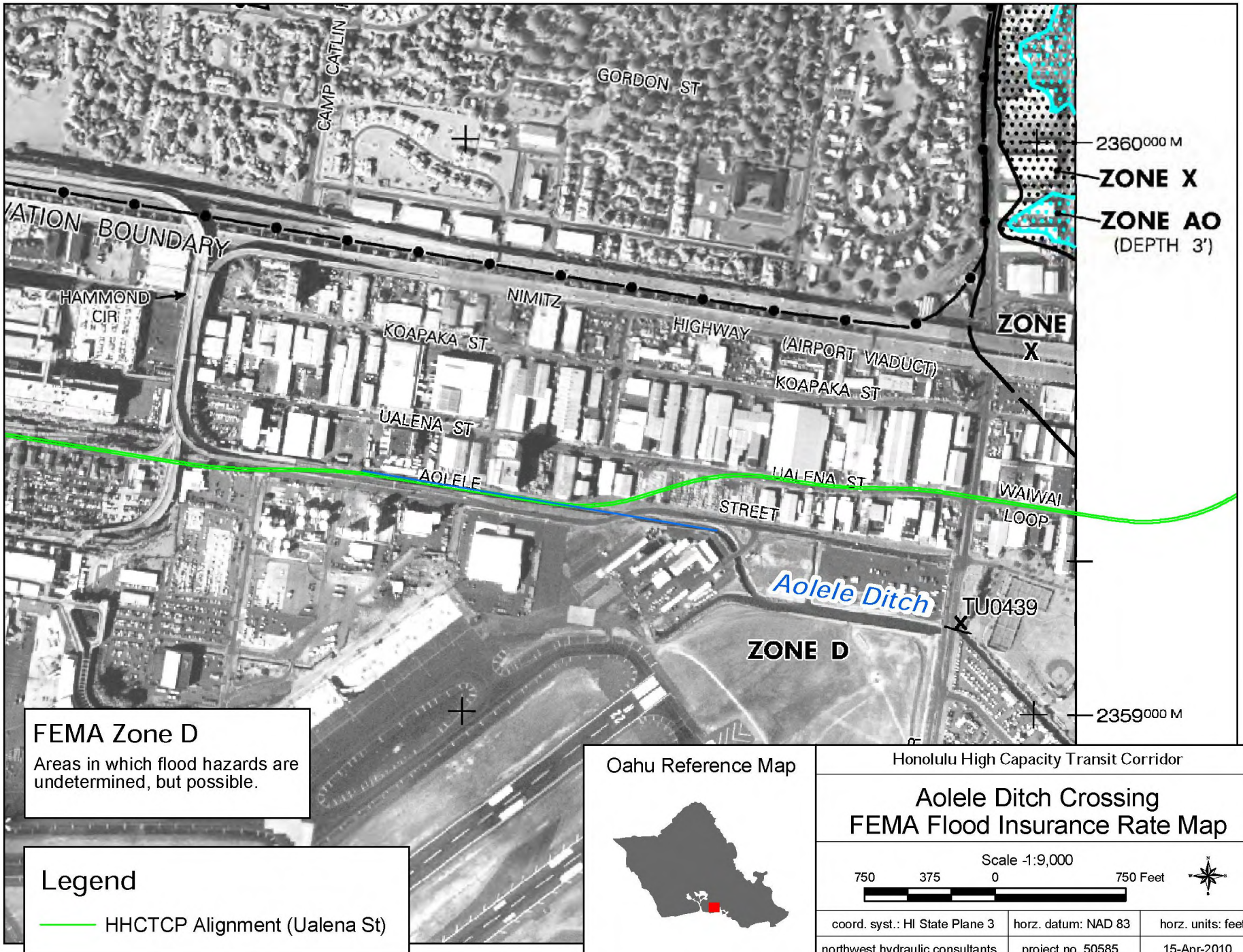
CAMP CATLIN
GORDON ST
VATION BOUNDARY
HAMMOND CIR
NIMITZ HIGHWAY (AIRPORT VIADUCT)
KOAPAKA ST
KOAPAKA ST
UALENA ST
AOLELE STREET
UALENA ST
WAIWAI LOOP
Aolele Ditch
TU0439
ZONE D
ZONE X
ZONE AO
(DEPTH 3')
ZONE X
2360000 M
2359000 M
FEMA Zone D
Areas in which flood hazards are undetermined, but possible.
Legend
HHCTCP Alignment (Ualena St)
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Aolele Ditch Crossing
FEMA Flood Insurance Rate Map
Scale -1:9,000
750 375 0 750 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
15-Apr-2010

Figure 13.2

14 - Moanalua Stream

Stream Crossing Description

The Moanalua Stream crossing (Site 14) is located in Segment J of the HHCTCP corridor, just south of the H-1 Freeway and the Nimitz Highway (Highway 92) bridge complex near the mouth of the Moanalua Stream where it enters Keehi Lagoon. Figure 14.1 provides a location map and shows the proposed guideway alignment and locations of the concrete guideway support columns that cross the floodplain and stream. FEMA has completed a detailed flood hazard investigation and prepared flood maps for this reach of Moanalua (FEMA, September 2004). Approximately 18 guideway support columns are located in the mapped 100-year floodplain at the mouth of Moanalua Stream. Two of these support columns are located in the regulatory floodway. FEMA and the Honolulu City and County Department of Planning and Permitting (DPP) require that all segments of the HHCTCP comply with the "No Rise" policy for any project features located in regulatory floodways. Representatives from NHC, PB, and LAI visited the site and observed that Moanalua Stream is hydraulically complex and heavily influenced by the presence of approximately twelve bridges in the immediate vicinity of the proposed crossing. Figure 14.2 provides a closer view of the guideway alignment and the locations of the support columns located in mapped AE zones and in the regulatory floodway. Photos 14.1 and 14.2 show views of Moanalua Stream looking along the proposed guideway crossing in a westerly and easterly direction, respectively.

Hydrologic Analysis and FEMA Study

According to the 2004 FEMA Flood Insurance Study (FIS), Moanalua Stream has a watershed drainage area of approximately 8 square miles just downstream of the Moanalua Road (currently North King Street) crossing, about 1.55 miles upstream from its mouth. The FEMA FIS reports that the peak 100-year discharge is approximately 13,500 cfs near Moanalua Road. NHC obtained two numerical models for Moanalua Stream: (1) a copy of FEMA's effective model of the stream above Moanalua Road from FEMA's Library; and (2) a HEC-RAS model that is believed to be a modified version of FEMA's effective HEC-2 model (circa 1982) for the lower reach (downstream from the Kikowaena Street crossing) provided by the Honolulu District of the United States Army Corps of Engineers (*personal communication,* Michael Wong email, February 25, 2010). It should be noted that the flows in the two models do not match precisely. The flow at the downstream end of the upper reach model is 13,500 cfs (near North King Street), while the flow at the upper end of the lower reach model is only 11,000 cfs. However, by the time flow reaches the proposed HHCTCP crossing location near Keehi Lagoon (1.55 miles downstream from North King Street) the Corps' lower reach model has gained flow and the total flow at that location is 13,500 cfs. It is obvious that the models for the lower and upper reaches were completed at different times and for different purposes, but it appears that these are the only available models that come close to approximating FEMA's currently published floodplain analyses and water levels (FEMA, 2004).

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FEMA's FIS Study maps show Zone AE areas (100-year flooding, from detailed hydraulic analyses) delineated along Moanalua Stream from its mouth at Keehi Lagoon to upstream of Moanalua Road. A FEMA floodway is also delineated on the current FIS maps for the entire reach that contains the proposed HHCTCP corridor.

Hydraulic Analysis

Model History and Revisions: NHC received the HEC-RAS model for the study reach from the USACE, Honolulu District. The model is believed to be an updated HEC-RAS version of the effective HEC-2 model originally used to prepare the FEMA FIS (which is not available from FEMA and NHC has not been able to obtain). As such, NHC has made the following changes to the USACE HEC-RAS model in order to replicate FEMA's (2004) published effective HEC-2 model results to the extent possible:

- In the HEC-RAS model provided by the Corps, the Corps had placed sediment in all flow areas below the 0 ft tide elevation which made those portions of their cross sections ineffective for conveying flow at all model cross-sections in the Corps' lower reach model. While this sediment may be a more realistic representation of today's existing conditions along the lower Moanalua Stream, NHC removed the fill to better replicate results from the effective model used for the FEMA FIS study.

- The complex bridge configuration near the study site has changed since the time when the FEMA effective HEC-2 model was prepared. The more recently developed HEC-RAS model received from USACE contains an updated bridge configuration that approximates today's conditions. NHC does not have sufficient information to recreate the bridge configuration used in the original FEMA effective model.

- The HEC-RAS model from USACE specified a downstream 1.5 ft tide elevation (water level) boundary in Keehi Lagoon; however, the effective FIS flood profile shows a 4 ft water elevation at the mouth for Moanalua Stream and Keehi Lagoon. NHC revised the USACE model to include a 4 ft downstream boundary to match the water level described in the FIS.

- Encroachment stations representing floodway boundaries were not included in the USACE model. NHC revised the USACE HEC-RAS model to include floodway boundaries estimated by NHC. The floodway boundaries were estimated from the original FEMA workmaps, FIS floodway data table, and DFIRM boundaries obtained from FEMA.

After making these changes, the NHC-revised HEC-RAS model closely replicates the water surface elevations and ground profile shown in the effective FIS. While NHC was unable to exactly duplicate the effective study results (water levels) through the complex bridge corridor, we were able to duplicate water levels shown in the effective study (FIS) upstream from the bridges. The only remaining difference between the NHC-revised HEC-RAS model and the effective model appears to be how the complex grouping of bridges near the mouth of Moanalua Stream are depicted, and it is not clear from the

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available information which bridges were included in the effective study or how they were represented in the original effective model. Because the original HEC-2 model is unavailable, the NHC-revised HEC-RAS model (NHC-Revised Model) provides the most reasonable and consistent model available for evaluating the impacts of the proposed HHCTCP on water levels in the effective Moanalua floodway, but it is NOT considered to be a duplicate effective model because of the lack of information about the original effective model as stated above. Given the extensive effort to try to locate and obtain the original effective HEC-2 model from FEMA and others, and because the NHC-Revised Model closely replicates the water surface elevations and ground profile shown in the effective FIS, NHC believes that we have produced a reasonably close version of a duplicate effective model that can be used to evaluate the effects of constructing two guideway support columns in the regulated floodway near the mouth of Moanalua Stream. Occasionally key information and the original effective models are missing from FEMA's library. FEMA often accepts approximate duplicate models if they have been carefully prepared with the best available information and they replicate published FIS information and BFEs. Use of the NHC-Revised Model to assess the effects of proposed guideway support columns in a regulatory floodway at Moanalua Stream will need to be reviewed by FEMA and DPP. Occasionally during review, FEMA will locate the missing effective model or information that can be used to develop a model that is acceptable to FEMA.

Application of the NHC-Revised Model: There are two square guideway support columns (one 6-foot square and one 7-foot square) located in the mapped FEMA floodway (Figures 14.1 and 14.2). These columns were added to the NHC-Revised Model to evaluate their impacts on water levels in the floodway. Model results are summarized in Table 14.1 below and indicate that the proposed columns will cause approximately a 0.08-ft rise in water levels in the floodway immediately upstream from the propose guideway corridor alignment (see first row in Table 14.1, at Model Cross Section 11.35, Rise in Water Level due to Columns = 0.08 ft). The approximate locations of the FEMA cross sections where Table 14.1 lists changes in water levels are shown in Figure 14.1.

Regulatory Compliance and Conclusions

Portions of the Moanalua crossing are located in mapped FEMA flood zones. Two guideway support columns are located in a regulatory floodway. The locations of all guideway support columns need to be verified and compared with FEMA's floodway boundary limits during final design. FEMA and DPP require that all segments of the HHCTCP comply with the "No Rise" policy for any project features located in regulatory floodways. NHC analyzed the potential water level rise caused by the two guideway support columns using the NHC-Revised Model described above, which is the closest replicate model to the effective study model that is presently available. Model results listed in Table 14.1 indicate a modest rise of 0.08-ft in the floodway water level immediately upstream of the proposed columns. Mitigation for this impact to the floodway or modification of the column layout may be required to achieve FEMA and DPP regulatory compliance for "no rise" in the floodway. Possible mitigation alternatives include (1) modifying the column locations so they are aligned with existing Highway 92 Bridge columns immediately upstream, (2) modifying the shape and possibly the effective width of the columns normal

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to the stream flow, (3) modifying one or more of the nearby bridges and/or bridge support columns (shape and/or effective widths) to be more hydraulically efficient, (4) excavating (slightly lowering) portions of the floodplain or stream banks (that are not armored) beneath the bridge complex that are within the floodway adjacent to the stream, (5) combinations of alternatives 1 through 4, and (6) moving the two columns outside of the regulatory floodway. Use of the NHC-Revised Model for assessing the effects of proposed guideway support columns in a regulatory floodway at Moanalua Stream will need to be approved by FEMA and DPP. Possible mitigation alternatives have not been assessed at this time.

Study Limitations

NHC requested the full FIS study report and all of the backup information including effective models and workmaps from FEMA for this study area. Unfortunately, FEMA's library and archives only contain a portion of the materials that were requested. FEMA is missing most of the backup materials and the effective HEC-2 models for the lower reach of the Moanalua Stream. The Honolulu District Corps of Engineers was able to locate and provide a HEC-RAS model for the lower reach which is believed to be an updated version of FEMA's effective HEC-2 model that was prepared by the Corps of Engineers for a flood control assessment study. NHC revised the HEC-RAS model received from the Corps of Engineers to replicate water levels (BFEs) presented in FEMA's effective study (FIS) to the extent possible. The NHC-Revised Model replicates, with reasonable accuracy, much of the information in the current FIS, with the exception of the flood profile through the cluster of multiple bridges in the vicinity of the proposed project site. The complex bridge configuration near the study site has changed considerably since the time when the FEMA effective HEC-2 model was prepared. The more recently developed HEC-RAS model received from the Corps of Engineers contains an updated bridge configuration that approximates today's conditions. NHC does not have sufficient information to recreate the bridge configuration used in the original FEMA effective model. Although an exact duplicate effective model is not available for this reach, NHC believes the NHC-Revised Model provides the most reasonable and consistent model available for evaluating the impacts of the proposed HHCTCP guideway support columns on water levels in the Moanalua floodway.

References

FEMA Flood Insurance Study (FIS) for City and County of Honolulu, HI. (September, 2004). Flood Insurance Study Number 15003CV001A.

Personal Communication, Michael Wong email, (February 25, 2010), US Army Corps of Engineers, Honolulu District.

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Table 14.1 Summary of Results from the NHC-Refined Model

Cross-Section Name as it is Referred to in FIS	Model Cross-Section	Distance from Keehi Lagoon to Model Cross-section (ft)	Distance from Project Site to Model Cross-section (ft)	Published Regulatory Floodway Water Levels (ft)	Computed Floodway Water Level (ft)		Rise in Water Level due to Columns [ft]
					Without Columns*	With Columns	
n/a, new cross section is located at guideway crossing	11.35	885	5	n/a	4.89	4.97	0.08
A	19.1	1600	720	5.6	5.78	5.83	0.05
B	25	2200	1320	6.4	6.53	6.57	0.04
C	33	3000	2120	9.8	9.82	9.82	0

*Model floodway results do not exactly match regulatory floodway because of the updated model configuration described above.

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Photos

Photo 14.1: Moanalua Stream at proposed guideway crossing adjacent to Highway 92 Bridge. Flow right to left. View west from pedestrian bridge.

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Photo 14.2: Moanalua Stream at proposed guideway crossing adjacent to Highway 92 Bridge. Flow left to right. View east from pedestrian bridge.

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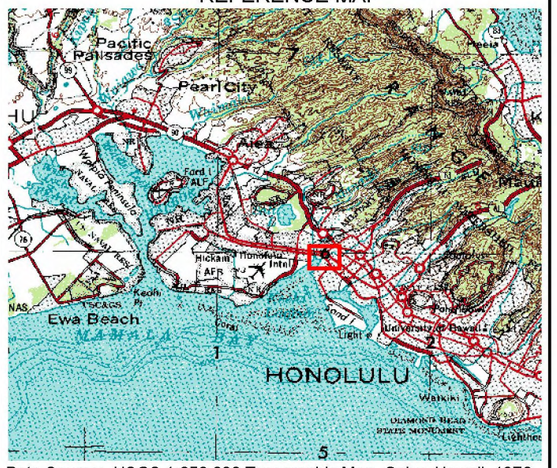
Figure 14.1 Location Map - Sites 14 and 15

Scale - 1:4,800 1 inch = 400 feet

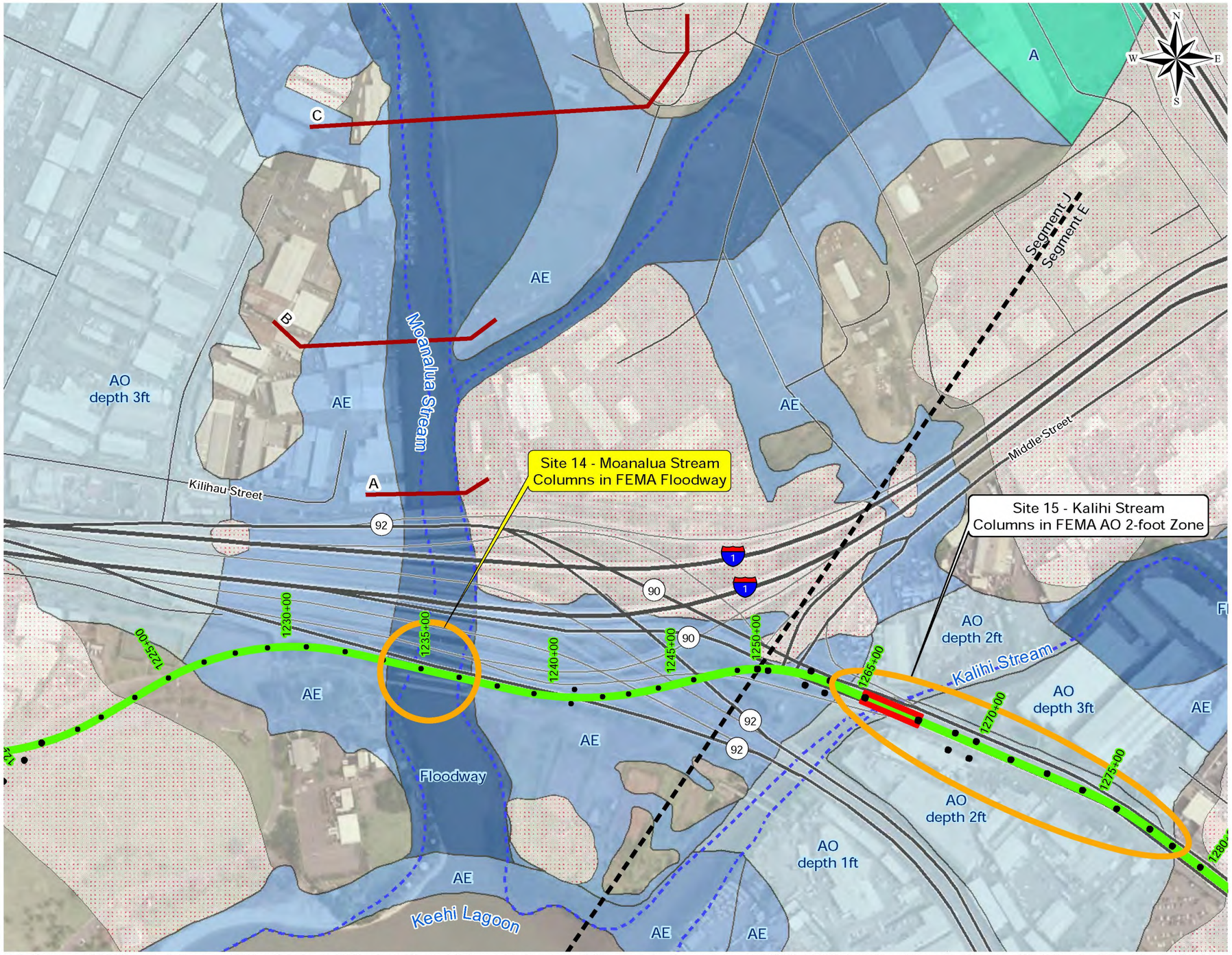
0 200 400 800 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50597	April 2010

REFERENCE MAP



Data Source: USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.



LEGEND

- Guideway Support Columns
- Platforms
- Track Alignment
- Segment Divider
- Highways
- Ramps
- Roads
- Streams

FEMA Flood Hazard Zones

- A
- AE
- Floodway - AE
- AO
- Zone X

- FIS Model Cross Section

Data Sources: USGS Color Aerial Imagery, 2004. ESRI Roads, 2008. Project Support Columns, Platforms, Track Alignments for Segments E and J, 2009. FEMA Flood Hazard Areas and Streams, City & County of Honolulu, HI, 6/2005.

Data Sources: USGS Color Aerial Imagery, 2004. Project Columns, Track Alignment, Stationing, and Contours, 2009. FEMA Floodway, City & County of Honolulu, HI, 6/2005. USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

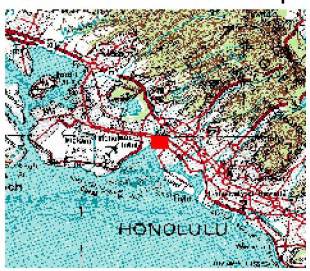
Legend

- Track Alignment
- Track Alignment Stationing
- Guideway Support Columns

FEMA Flood Hazard Zones

- Floodway-AE
- AE
- X

Oahu Reference Map



Honolulu High Capacity Transit Corridor

Site 14 Moanalua Stream Crossing

Scale -1:2,400

100 50 0 100 200 Feet

coord. syst.: HI State Plane 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50597	Apr-2010

Figure 14.2

15 - Kalihi Stream Crossing

Stream Crossing Description

The Kalihi Stream extends from the headwaters of the Koolau Mountains and discharges into the Keehi Lagoon (Figure 15.1). The proposed Honolulu High Capacity Transit Corridor Project (HHCTCP) guideway parallels Dillingham Blvd, crossing Kalihi Stream between the east and west bound lanes of the highway bridge. The guideway will span the crossing on 8-ft by 8-ft columns that are set back from the stream channel. Additionally, an elevated Transit Center Station Platform will be located at the crossing supported 6-ft by 6-ft columns that are set back from the stream channel. Figure 15.2 includes a close up of the stream crossing, column, station, foundation shaft locations, and 1-ft LiDAR contours.

Representatives from NHC, PB and LAI visited the Kalihi Stream Crossing location on September 23, 2009. Photos 15.1-15.5 depict the stream channel and proposed HHCTCP column locations near the stream crossing.

Hydrologic Analysis and FEMA Study

An existing FEMA Flood Insurance Study (FIS) has been completed for this reach of Kalihi Stream. Hydrology in the FIS was completed using discharge records from the USGS Gaging Station No. 16229000. A log-Pearson Type III analysis determined that the peak 100-year discharge is approximately 16,880 cfs at the mouth of the stream which is in the vicinity of the proposed crossing.

Flood hazards on Kalihi Stream were analyzed by FEMA and are illustrated in Figure 15.3 and are discussed below.

Hydraulic Analysis

The Kalihi crossing is located within an effective FEMA Zone AO (Figure 15.3). Twelve of the proposed HHCTCP columns are located completely within the effective Zone AO (Depth 2') and one column is located within the effective Zone AO (Depth 1', see Figure 15.3). Figure 15.4 shows a close up of column locations and flood zone boundaries. Additionally, the Middle Street Transit Center Station will be located within the Zone AO.

No columns or structures are located within a FEMA Floodway.

Regulatory Compliance

City and County development regulations (Sec. 21-9.10-6, 5A) require that "All new construction...within the AO zone shall have the lowest floor elevated above the highest adjacent grade at least as high as the depth number specified on the flood maps; or, together with attendant utility and sanitary facilities, be

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completely floodproof to or above that level..." All proposed columns located within the Flood Fringe District (Zone AO) comply with the City and County requirements regarding floodproofing. All touchdown portions of the Middle Street Transit Center Station located within the Zone AO (depth 2-ft) will need to be floodproofed to an elevation 2-ft above the adjacent grade in order to meet regulatory standards.

Conclusions

HHCTCP structures that are located within the FEMA Zone AO, will need to be adequately floodproofed and raised to comply with City and County regulations. None of the proposed facilities are located within a FEMA Floodway.

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Photos

Photo 15.1. Standing upstream of Kalihi/Dillingham Blvd Crossing on left (southeast) bank. Viewing downstream.

Photo 15.2. Standing on Upstream end of Kalihi/Dillingham Blvd bridge viewing west along the road.

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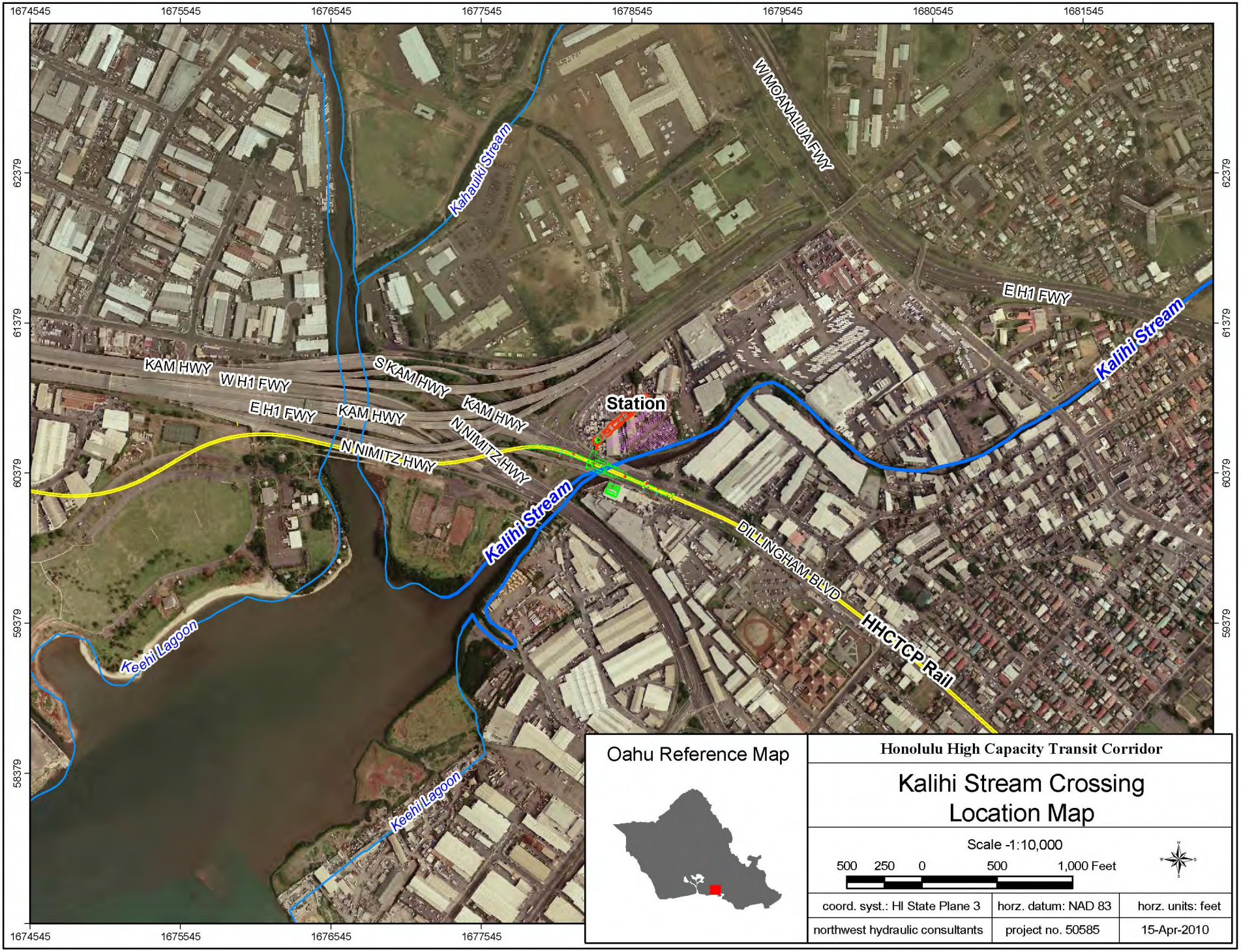
Photo 15.3. Upstream face of Kalihi/Dillingham Blvd bridge viewing southeast along Dillingham Blvd.

Photo 15.4. Standing on upstream end of Kalihi/Dillingham Blvd bridge viewing downstream.

Photo 15.5. Upstream face of Dillingham Blvd bridge viewing upstream (northeast).


1674545
1675545
1676545
1677545
1678545
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1680545
1681545
62379
61379
60379
59379
58379
Kahauiki Stream
W MOANALUA FWY
E H1 FWY
Kalihi Stream
KAM HWY
W H1 FWY
S KAM HWY
KAM HWY
E H1 FWY
KAM HWY
N NIMITZ HWY
N NIMITZ HWY
Station
Kalihi Stream
DILLINGHAM BLVD
HHCTCP Rail
Keehi Lagoon
Keehi Lagoon
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Kalihi Stream Crossing
Location Map
Scale -1:10,000
500 250 0 500 1,000 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
15-Apr-2010

Figure 15.1

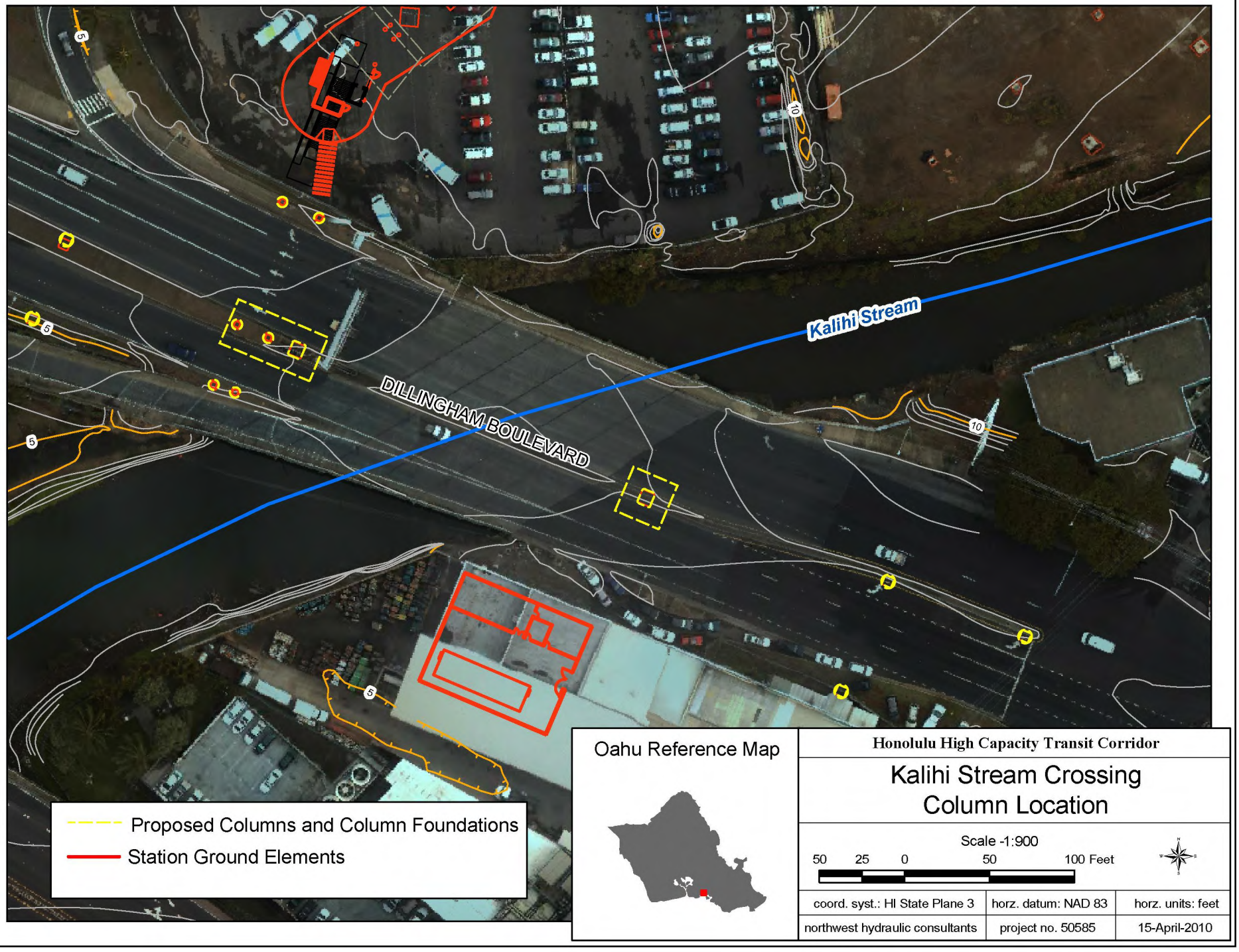
Kalihi Stream
DILLINGHAM BOULEVARD
Proposed Columns and Column Foundations
Station Ground Elements
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Kalihi Stream Crossing
Column Location
Scale -1:900
50 25 0 50 100 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
15-April-2010

Figure 15.2

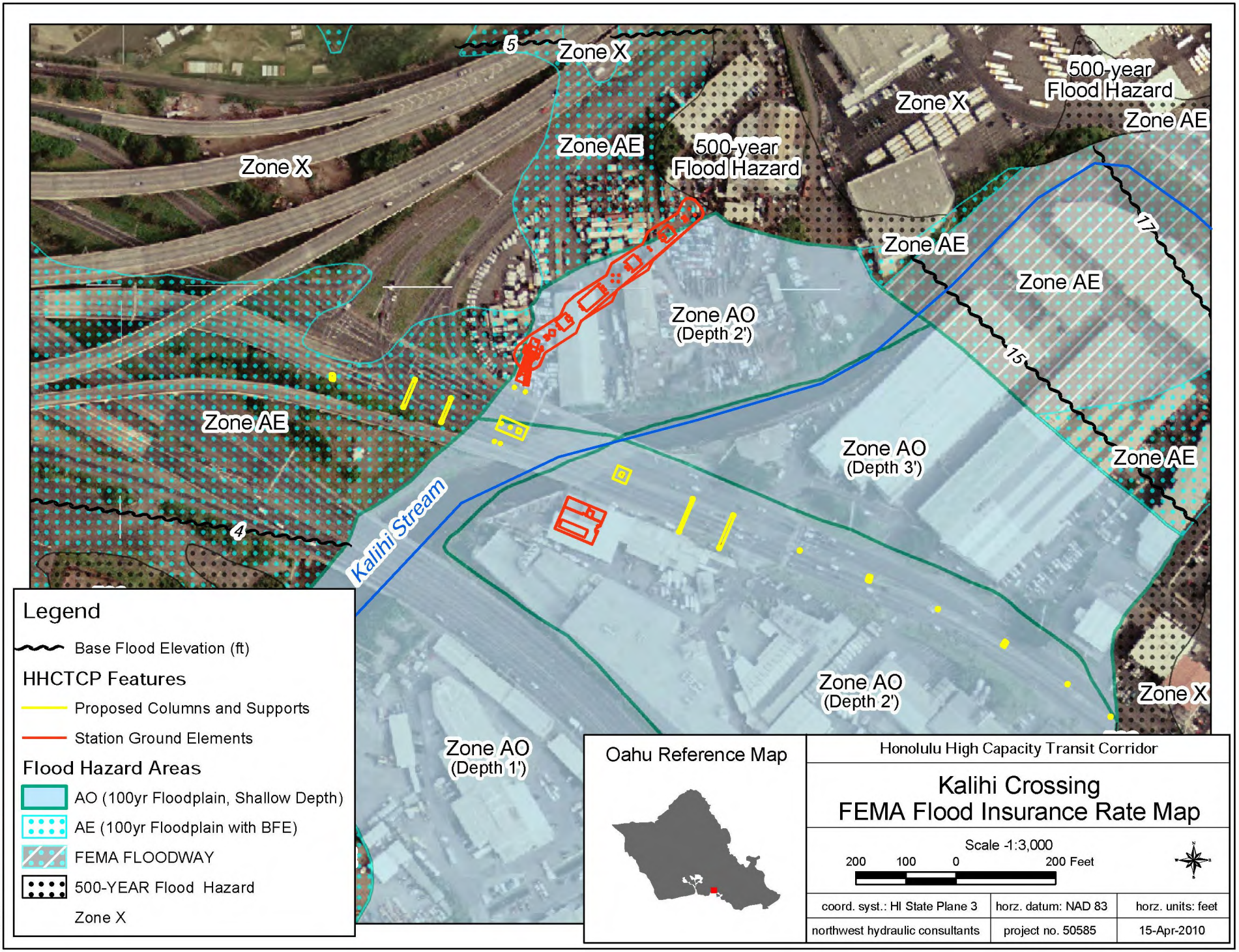

Figure 15.3

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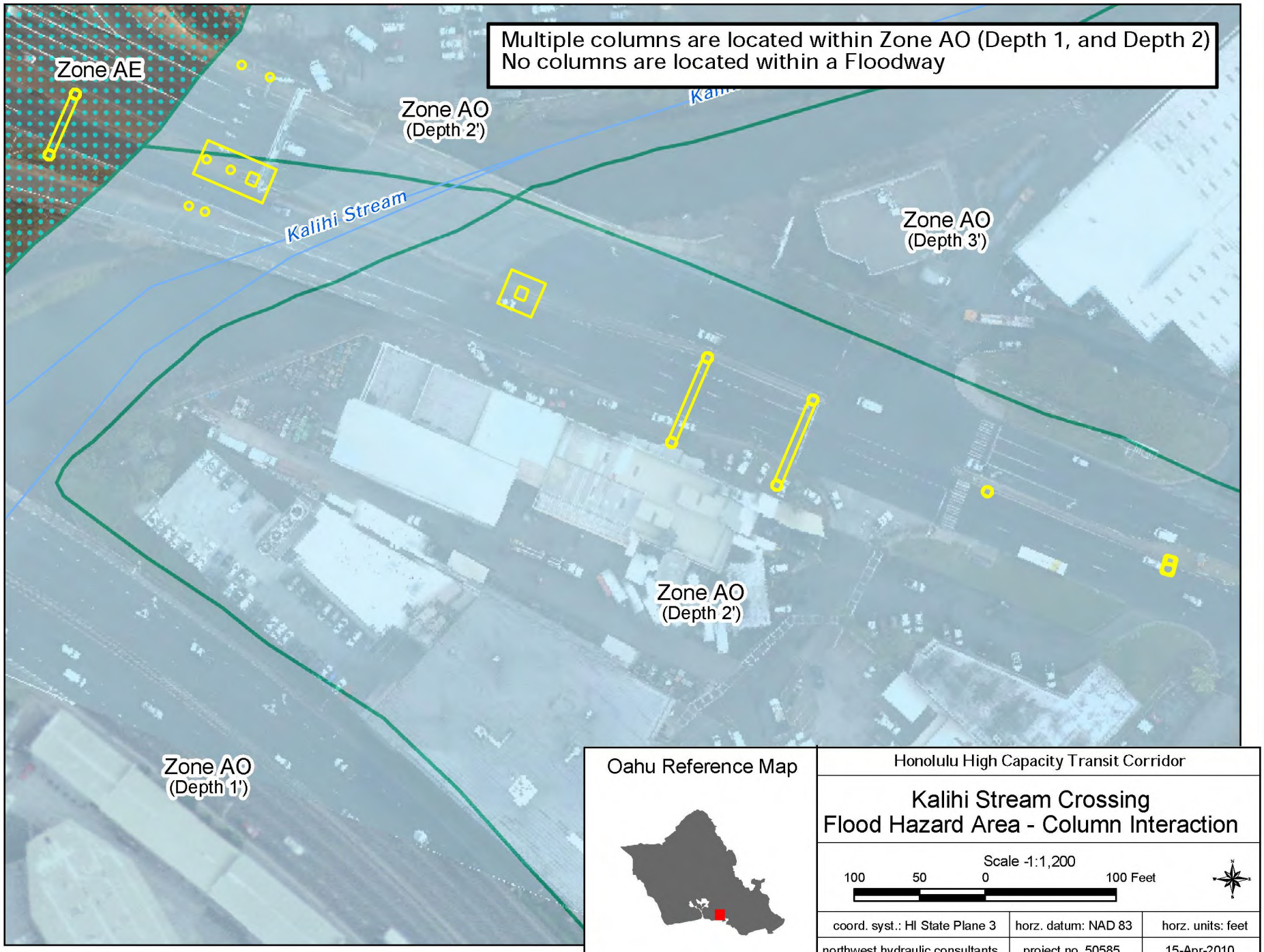
Multiple columns are located within Zone AO (Depth 1, and Depth 2)
No columns are located within a Floodway
Zone AE
Zone AO
(Depth 2')
Kalihi Stream
Zone AO
(Depth 3')
Zone AO
(Depth 2')
Zone AO
(Depth 1')
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Kalihi Stream Crossing
Flood Hazard Area - Column Interaction
Scale -1:1,200
100 50 0 100 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
15-Apr-2010

Figure 15.4

16 - Kapalama Canal Stream Crossing

Stream Crossing Description

The Kapalama Canal Stream Crossing is located adjacent to the Nuuanu Watershed, northwest of downtown Honolulu (see Figure 16.1). The proposed Honolulu High Capacity Transit Corridor Project (HHCTCP) elevated rail parallels Dillingham Boulevard, and crosses Kapalama Canal Stream between the east and west bound lanes of the highway bridge (Figure 16.2). Plans at this location include widening of the existing highway bridge to accommodate the rail line, as well as an HHCTCP rail transit station to be located approximately 100 feet south east of bridge. Rail columns are 6-ft by 6-ft square, and are set back from the stream channel (Figure 16.2).

Representatives from NHC, PB and LAI visited the Kapalama Canal Stream Crossing location on September 23, 2009. Photos 16.1 - 16.8 depict the stream channel and surrounding land features on that date.

Hydrologic Analysis and FEMA Study

No hydrologic analysis was completed for this study. Kapalama Canal Stream does not have an effective Flood Insurance Study (FIS) or FEMA Flood Insurance Rate Map (FIRM).

Hydraulic Analysis

No hydraulic investigation was completed at this site because there is little chance that the crossing will have any measurable impact on flooding. As noted above, the rail line is supported on two columns which are not located in the stream channel (Figure 16.2). Piles driven to support widening of the existing highway bridge will be located in-line with flow around existing piles (See Figure 16.3 – 16.4) and should therefore have insignificant impact on flow under the structure.

Because a FEMA study has not been completed for Kapalama Canal Stream we do not know if water will overtop the channel banks and cross the highway during a 100-year flood. This reach of the stream has been highly modified and enlarged to increase capacity for drainage and flood relief. Therefore, even during a major flood water may not leave the channel, or if it does, the contours shown in Figure 16.2 indicate that it will bypass the bridge to the east and west, dispersing along Dillingham Boulevard. Even if water flows around columns, the depth would be relatively small and therefore the impact of the HHCTCP on flows would be insignificant.

Regulatory Compliance

To our knowledge a flood study has not been completed for this reach of the Kapalama Canal Stream and therefore, we do not know if the columns are located within the limits of the flood fringe.

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Regardless, it is our opinion that the likely impact the columns would have on flooding is so inconsequential that detailed hydrologic and hydraulic investigations are not warranted. The columns themselves are not at risk because they will be supported on deep foundations so scour is not a concern. Also they will be made of solid concrete and can not be damaged.

NHC recommends that the Kapalama Transit Station buildings be floodproofed since the station is located only a short distance from the stream in an area that could be prone to flooding should water exit the channel. The area is very flat and therefore we would expect flood depths to remain shallow. NHC and PB will work together to determine adequate floodproofing features.

Conclusions

The columns are setback from the stream channel and may or may not be located within the flood fringe. If water leaves the channel and reaches the columns, the impact to flooding will be insignificant and there should be no adverse impact on adjacent facilities or properties.

Because the Kapalama Transit Station Building is in a low lying area adjacent to the canal NHC recommends that the building be adequately floodproofed.

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Photos

Photo 16.1. Downstream, on left bank, of Kapalama Canal Stream viewing upstream towards the Dillingham Bridge.

Photo 16.2. Downstream, on left bank, of Kapalama Canal Stream crossing viewing downstream.

AR00041237

Photo 16.3. On Dillingham Boulevard viewing north along Kokea Street. Proposed column would be located in the center of this photo (in the middle of Dillingham Boulevard).

Photo 16.4. Viewing east (along Dillingham Boulevard) from downstream face of Kapalama Canal Stream Crossing. The Kapalama Station will be located about 250 from the crossing along the Dillingham Boulevard.

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Photo 16.5. Upstream of Kapalama Canal Stream viewing west along Dillingham Boulevard.

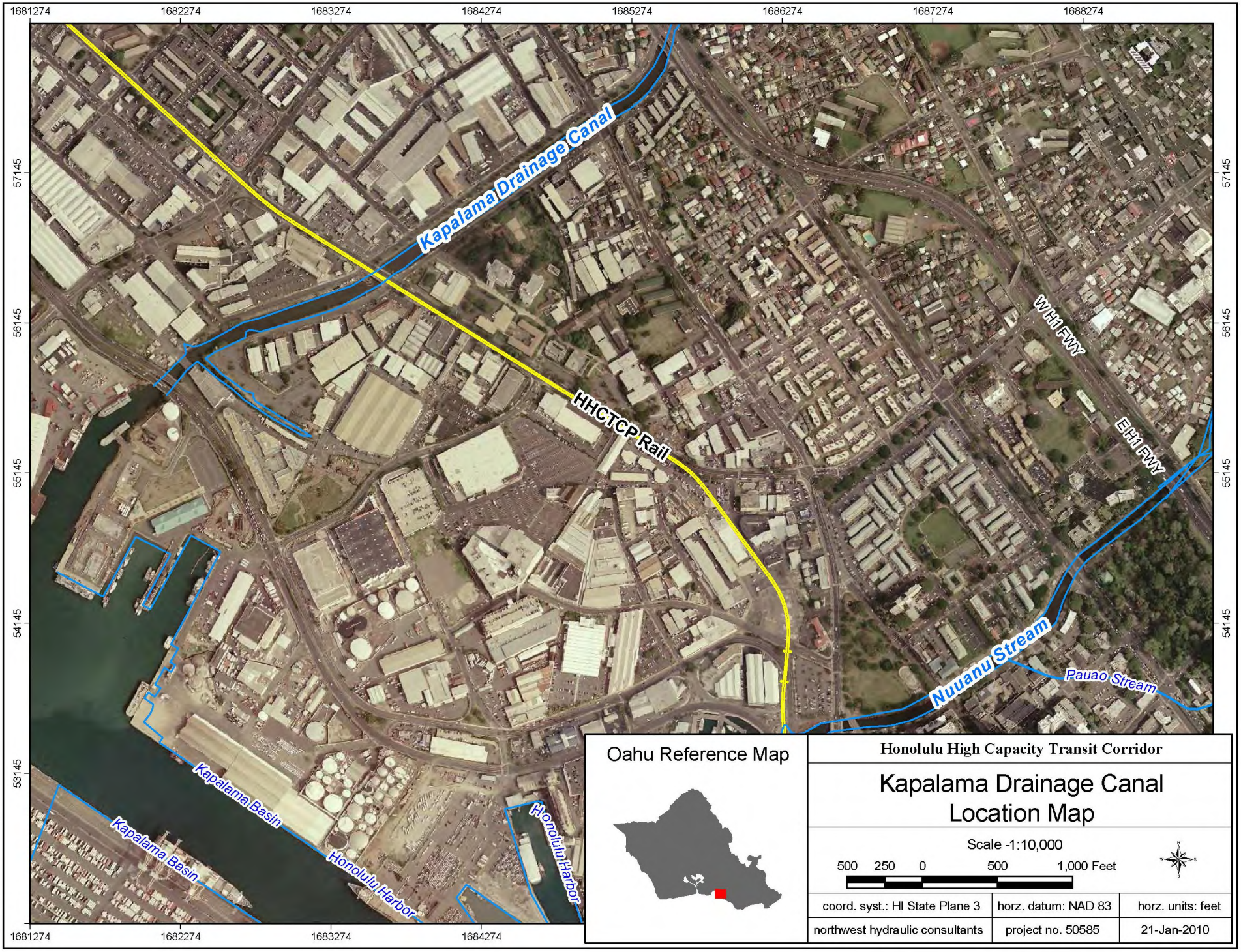
Photo 16.6. On Kapalama Canal Stream crossing viewing upstream.

AR00041239

Photo 16.7. Upstream of Kapalama Canal Stream viewing west along Dillingham Boulevard.

Photo 16.8. Upstream of Kapalama Canal Stream viewing west along Dillingham Boulevard.

AR00041240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1682274
1683274
1684274
1685274
1686274
1687274
1688274
57145
56145
55145
54145
53145
Kapalama Drainage Canal
HHCTCP Rail
WH1 FWY
EH1 FWY
Nuuanu Stream
Pauao Stream
Kapalama Basin
Kapalama Basin
Honolulu Harbor
Honolulu Harbor
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Kapalama Drainage Canal
Location Map
Scale -1:10,000
500 250 0 500 1,000 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
21-Jan-2010

Figure 16.1

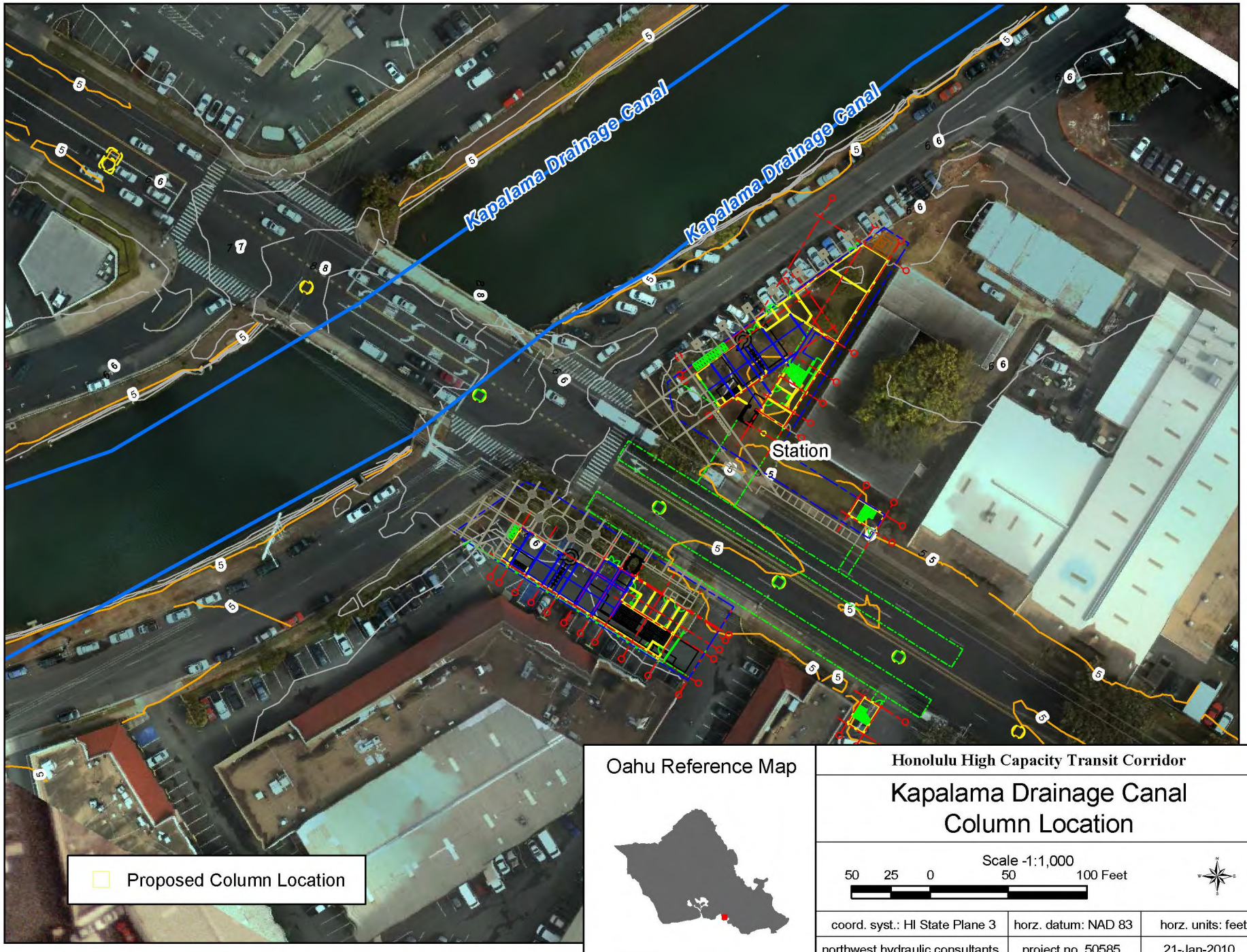
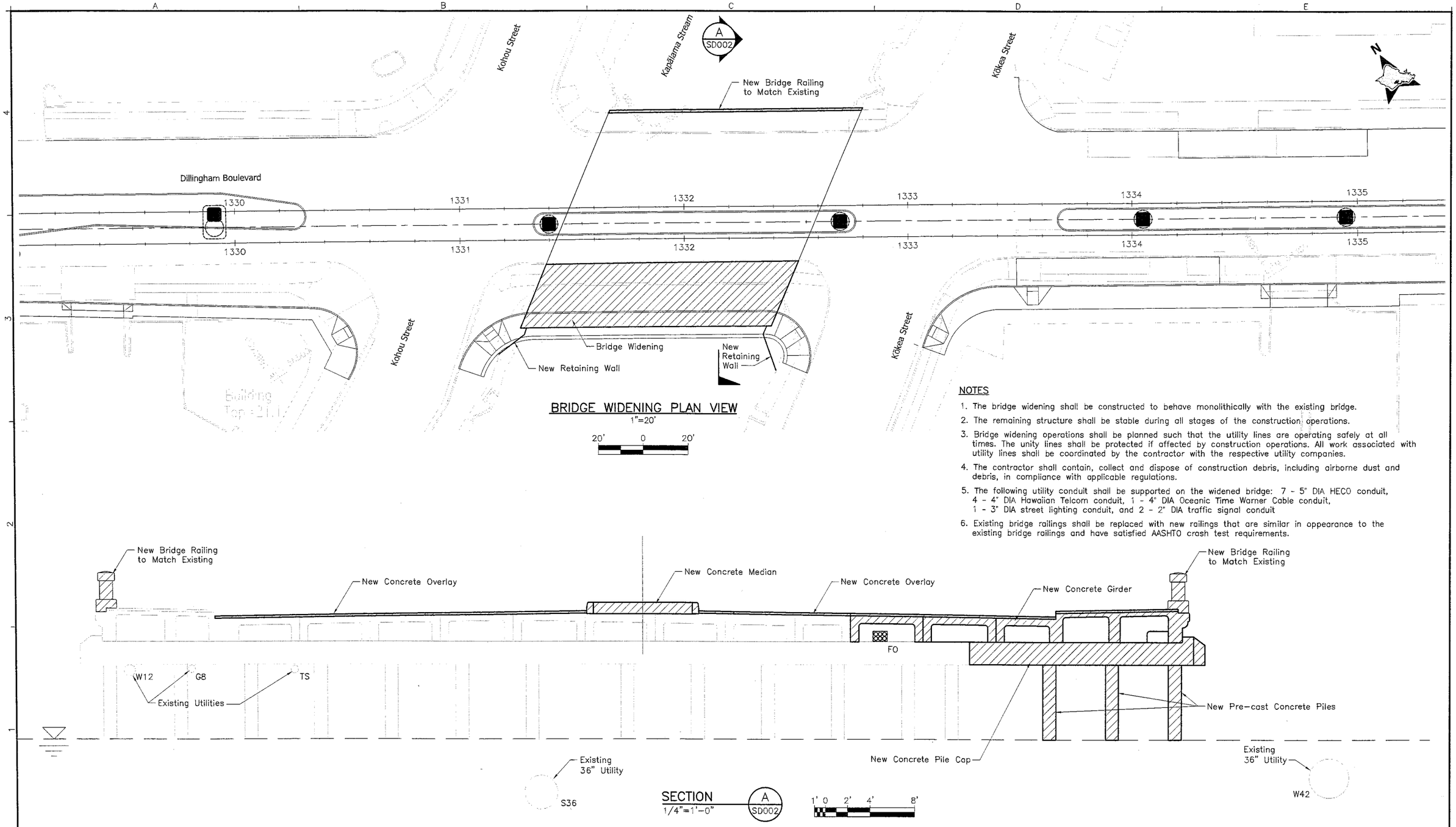

Kapalama Drainage Canal
Kapalama Drainage Canal
Station
Proposed Column Location
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Kapalama Drainage Canal
Column Location
Scale -1:1,000
50 25 0 50 100 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
21-Jan-2010

Figure 16.2



Rev	By	Date	Description

PRELIMINARY ENGINEERING SUBJECT TO REVISION

Designed: T Kimura
Drawn: T Cochran
Checked: D Yavorsky
Approved: A Borst
Date: 12-21-09

HONOLULU HIGH-CAPACITY TRANSIT CORRIDOR PROJECT
CITY & COUNTY OF HONOLULU - DEPARTMENT OF TRANSPORTATION SERVICES - RAPID TRANSIT DIVISION

Prime Consultant: PB PARSONS BRINCKERHOFF
1003 Bishop Street, Suite 2250 - Honolulu, HI 96813

Subconsultant:

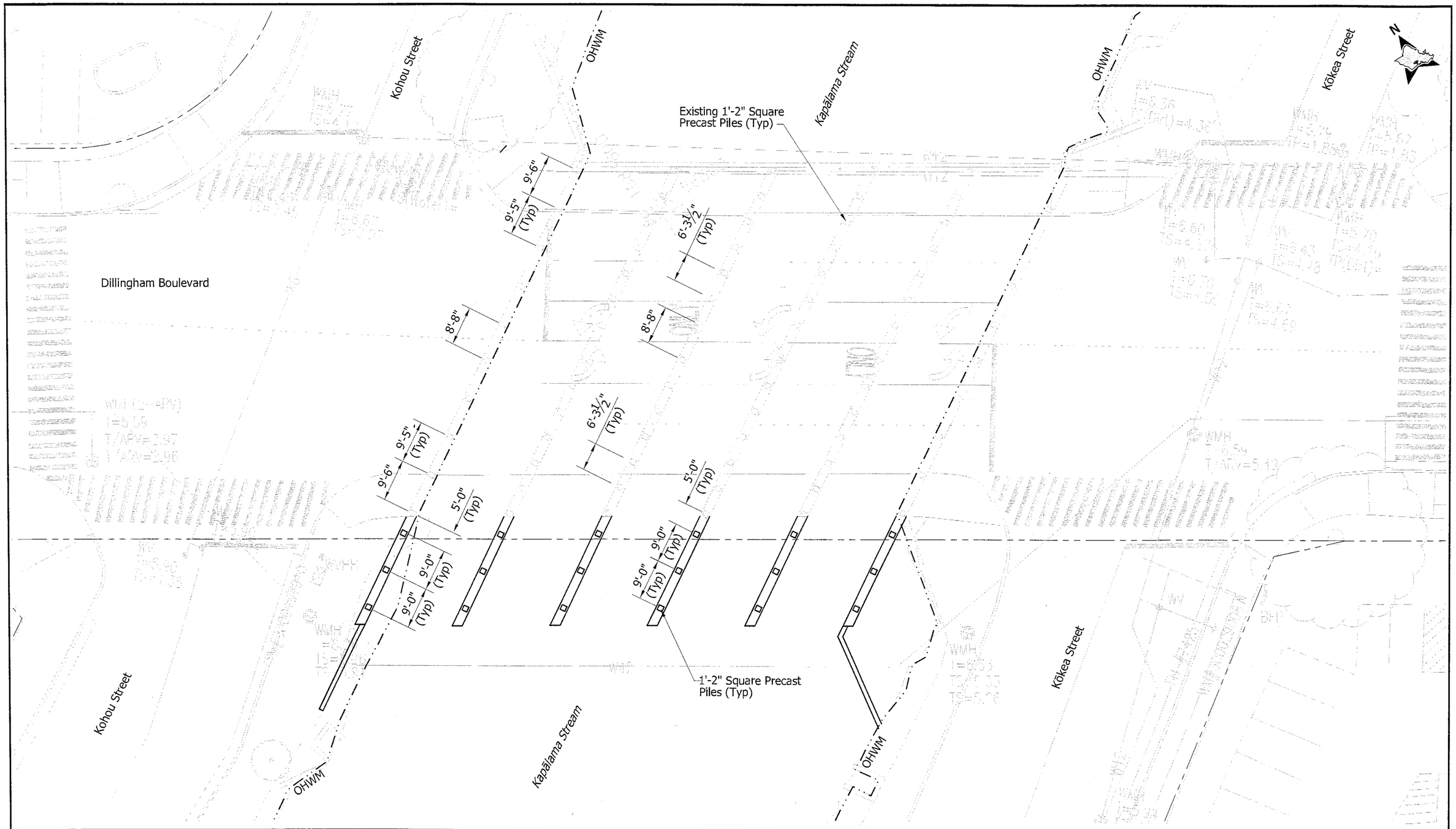
For reduced prints, original page size in inches: 0 1 2 3 4

CITY CENTER GUIDEWAY & UTILITIES

KAPĀLAMA CANAL BRIDGE
BRIDGE WIDENING

Contract No.: SV-530
CADD File: CC-G07-SD002
Drawing No.: SD002 Rev.
Scale: As Noted
Page No. 87 of 202

Figure 16.3. Preliminary Kapalama Canal Bridge Widening Plan - Elevation View



CITY & COUNTY OF HONOLULU DEPARTMENT OF TRANSPORTATION SERVICES RAPID TRANSIT DIVISION	HONOLULU HIGH-CAPACITY TRANSIT CORRIDOR PROJECT	Scale: 10' 0 10' 1"=10'	PRELIMINARY ALIGNMENT PLANS AND DETAILS CWA SECT 401 WATER QUALITY CERTIFICATION KAPĀLAMA CANAL BRIDGE WIDENING LAYOUT	Page No. 1 of 1 Drawing No: - Date: January 2010

Figure 16.4. Preliminary Kapalama Canal Bridge Widening Plan - Plan View

17 - Nuuanu Stream Crossing

Stream Crossing Description

Nuuanu Stream extends from the headwaters in the Koolau Mountains through the Chinatown district of Honolulu before discharging into Honolulu Harbor. The proposed Honolulu High Capacity Transit Corridor Project (HHCTCP) crosses Nuuanu Stream at the Nimitz Highway (see Figure 17.1). The elevated rail will be supported by a series of 6-ft square columns located in the Nimitz Highway median, with two columns located within the channel between the Nimitz Highway bridges (see Figure 17.2). The HHCTCP Chinatown Station will be located immediately south of the Nuuanu Stream crossing.

Representatives from NHC, PB and LAI visited the Nuuanu Stream Crossing location on September 23, 2009. Photos 17.1 to 17.8 show the stream channel and the Nimitz Highway bridges.

Hydrologic Analysis

The effective Flood Insurance Study (FIS) for Nuuanu Stream does not extend downstream of Interstate H-1, which is well upstream of the HHCTCP crossing. The Nuuanu basin boundary at Nimitz Highway was delineated using available State of Hawaii watershed boundary data and National Geospatial-Intelligence Agency LiDAR data provided by the City and County of Honolulu (CCH). The Nuuanu drainage basin above Nimitz Highway is approximately 5,240 acres. Using Plate 6 of the CCH Storm Drainage Standards (2000) the 100-year discharge in Nuuanu Stream at the HHCTCP crossing is estimated to be 14,400 cfs. It should be noted that there are several reservoirs in the upper watershed which may reduce peak flood flows, but this has not been considered in the flow rate stated above.

Hydraulic Analysis

The site inspection, new 1-ft LiDAR contours, high quality aerial photos, and Nimitz Highway bridge plans were used to determine if the proposed columns will have an impact on flooding. Apparently a hydraulic model does not exist for this reach of the Nuuanu Stream and there is no channel cross section or bathymetry data available. The stream consists of a constructed channel lined by concrete walls on both sides.

The two columns that will be placed in the stream channel will have no significant impact on the 100-year flood levels. The north column (Station 1389+80) will be located downstream from the abutment of the upstream highway bridge (see Figure 17.3). The abutment projects into the channel creating a relatively large ineffective flow area just downstream which is where the column will be located (see Photo 17.7). Therefore, this column will likely have no measurable impact on flood levels in the Nuuanu channel. The south HHCTCP column (Station 1391+00) will be located within the active channel (see Figure 17.3). The 100-year flow will likely by contained entirely in the Nuuanu channel through this

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reach, thus the other HHCTCP columns and the Chinatown Station will not impact flood levels in Nuuanu Stream.

To estimate whether this column will impact flood levels a very simplistic HEC-RAS model was constructed that extends from the Honolulu Harbor to upstream through the Nimitz highway bridges. Channel bathymetry was assumed using available rough data shown on the 1950 and 1932 bridge plans and topographic data created for the HHCTCP project. The downstream boundary condition was set to a constant water surface elevation of 1.9-ft in the harbor, which is consistent with the FIS at nearby sites. The simple model indicates that constructing the column at station 1391+00 in the Nuuanu channel would create a negligible rise in the upstream water surface elevation. To further minimize this impact, we suggest that if possible the column be located in line with the existing piers of the upstream Nimitz Highway bridge. Channel bathymetry for Nuuanu Stream would need to be collected and more analysis completed if a more detailed hydraulic evaluation is required at this site.

Regulatory Compliance

To our knowledge a flood study has not been completed for this reach of Nuuanu Stream and, based on the effective FEMA FIRM (Figure 17.4); but the design calls for placing two columns within the stream channel. The City may require the columns to cause "No-Rise" in 100-year flood levels. As discussed above, the north column at station 1389+80 is in the lee of the upstream highway bridge abutment in an area of ineffective conveyance and therefore will not impact flood levels. The south column at station 1391+00 is in the channel, but the simple HEC-RAS analysis suggests that its impact will be negligible.

Conclusions

Although a detailed hydraulic investigation has not been completed for this site, it appears the proposed project will have no significant impact on flood levels.

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Photos

Photo 17.1. Viewing the upstream side of the upper Nimitz Highway bridge.

Photo 17.2. Viewing north along the upper Nimitz Highway bridge.

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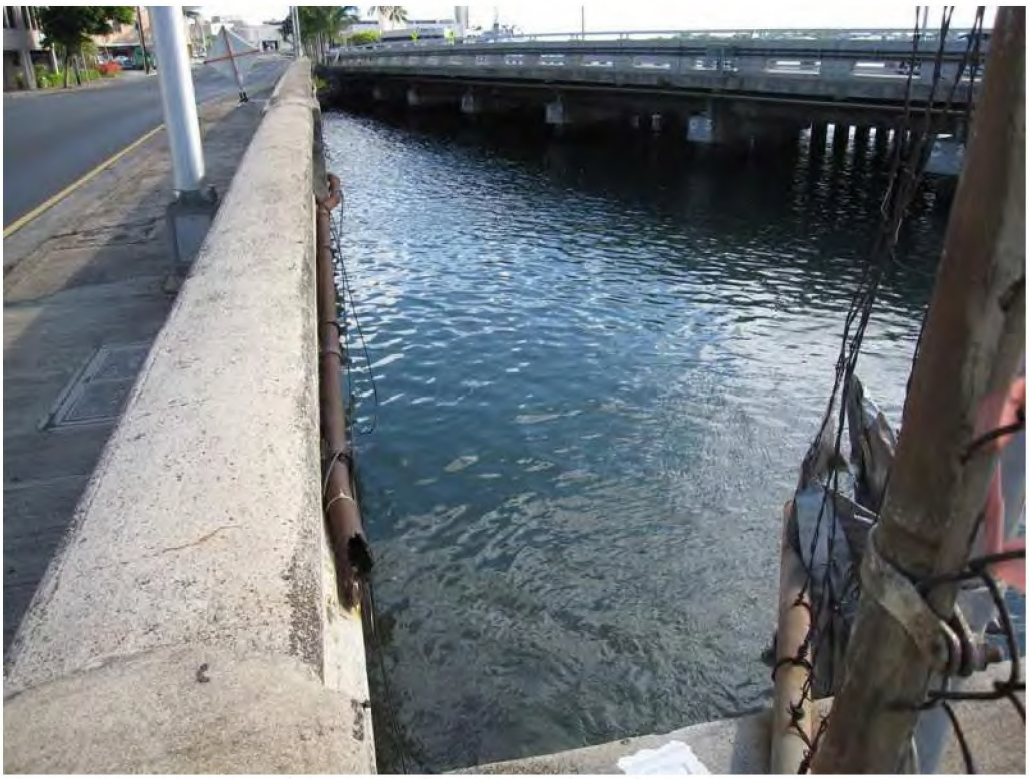


Photo 17.3. Viewing South between the Nimitz Highway bridges.

Photo 17.4. Viewing North between the Nimitz Highway bridges.

AR00041248

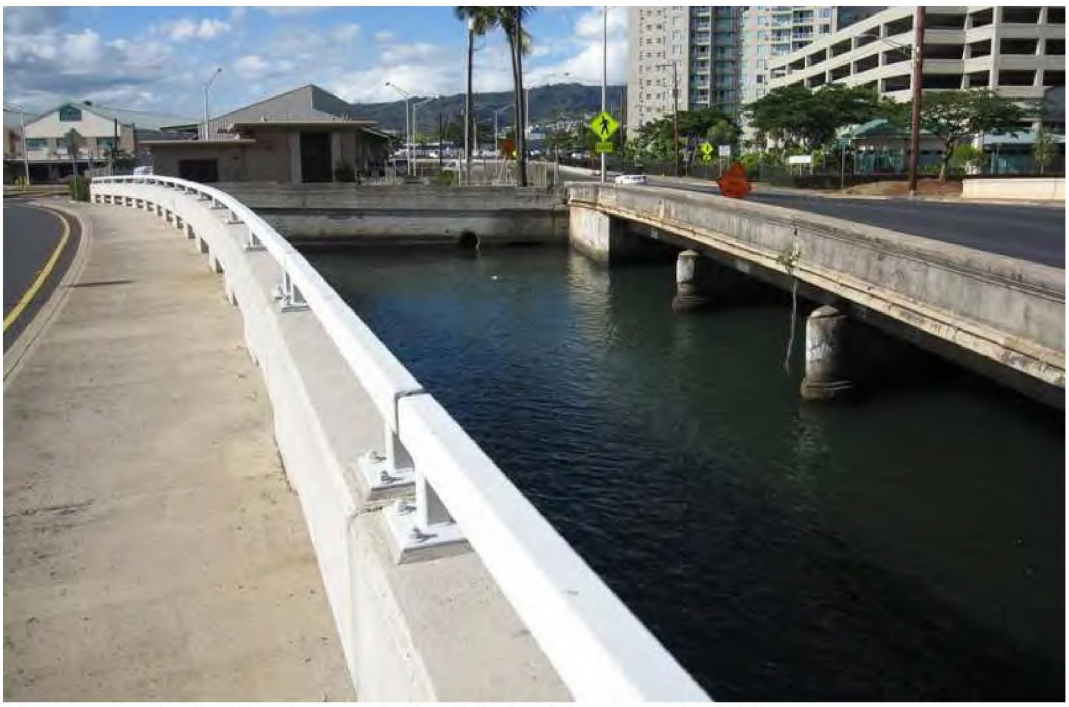


Photo 17.5. Viewing North towards the right bank between the bridges.

Photo 17.6. Viewing South at the location of the proposed HHCTCP column (to be located in the waterway in foreground).

Photo 17.7. Viewing upstream at the right bank abutment of the upper Nimitz highway bridge. The HHCTCP column would be located in the lee of the abutment (left in photo).

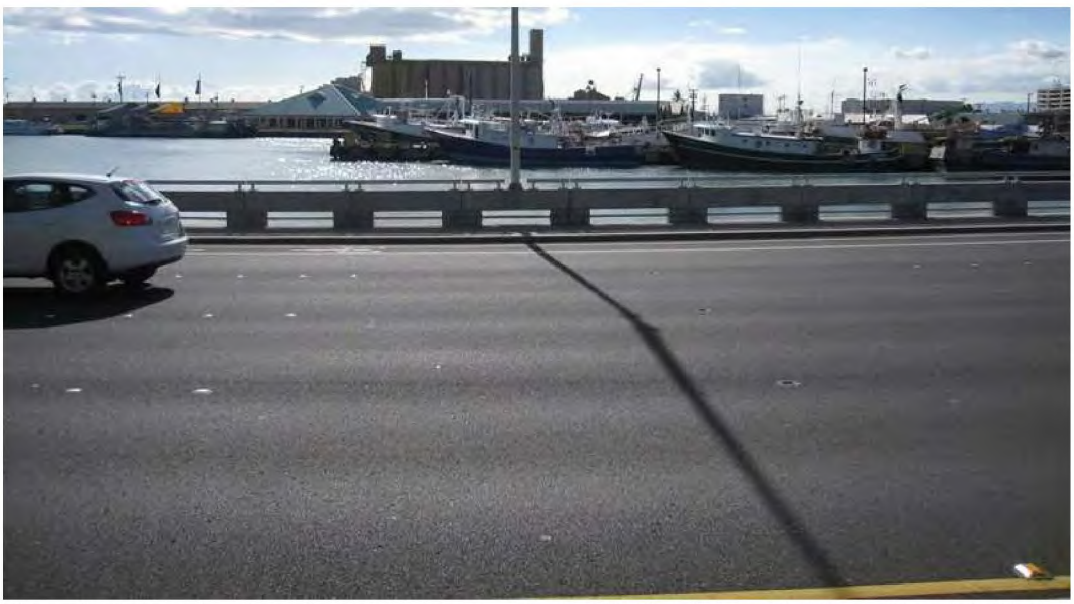
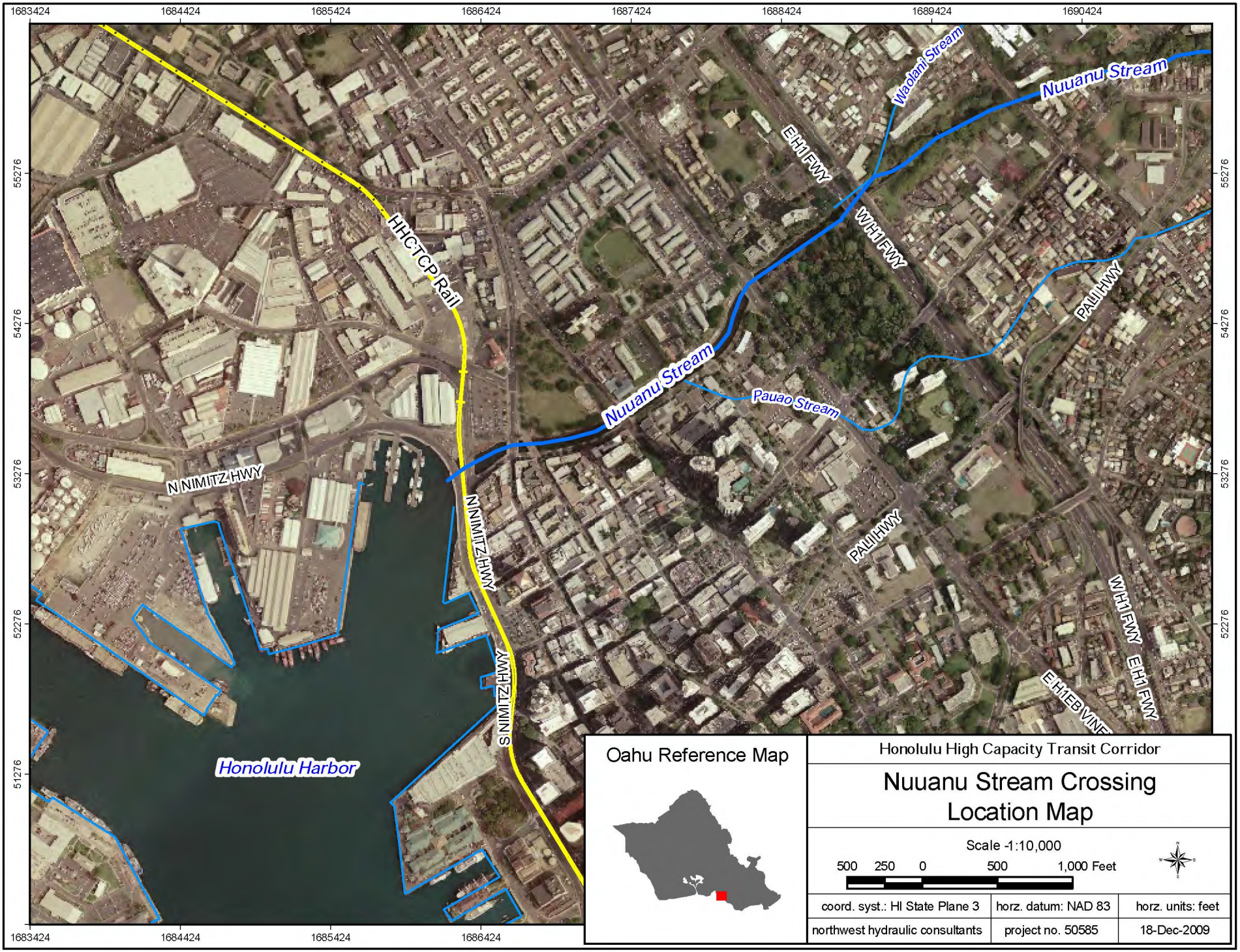


Photo 17.8. Viewing downstream of the downstream Nimitz Highway bridge to Honolulu Harbor.

AR00041250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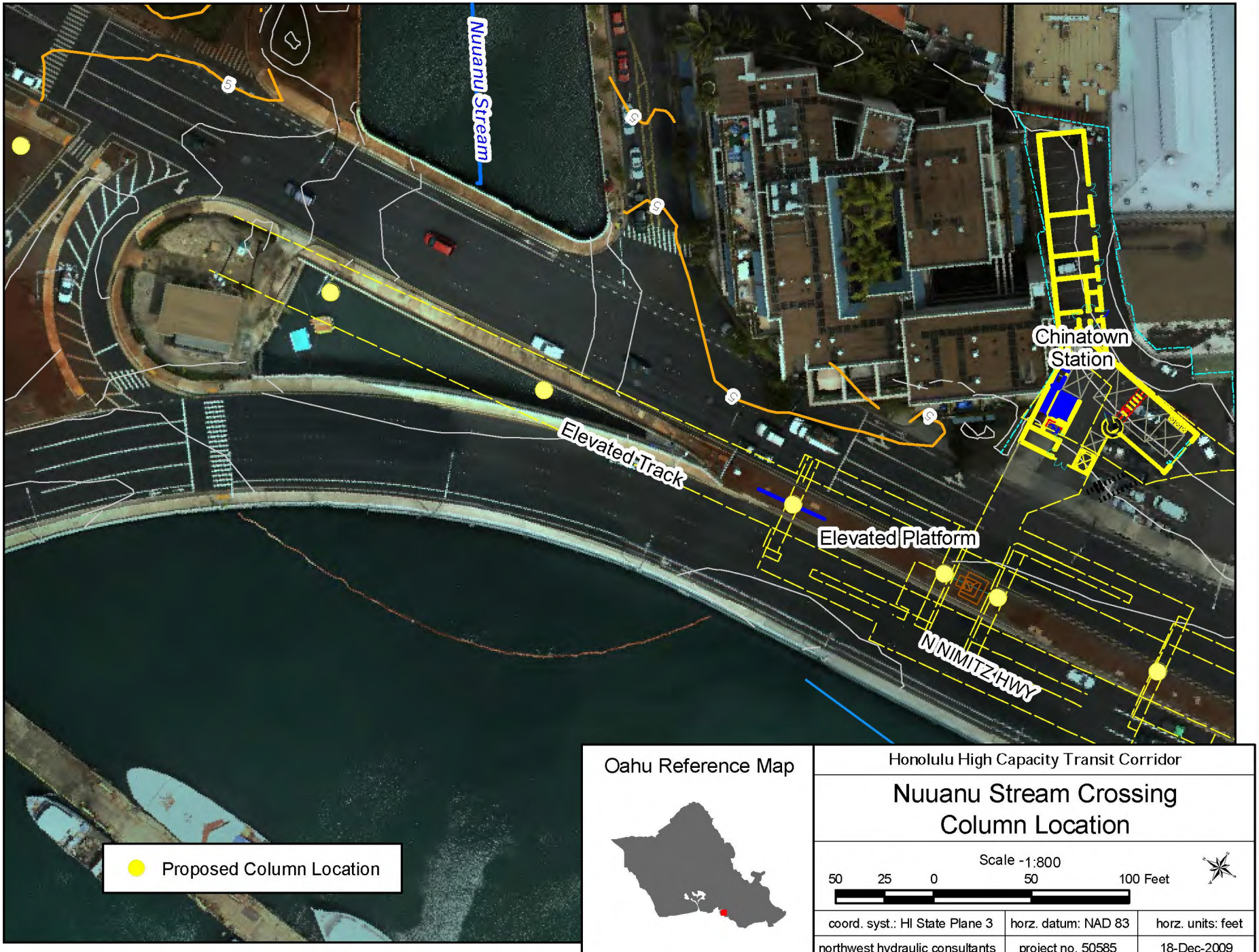
1688424
1689424
1690424
55276
54276
53276
52276
51276
Waolani Stream
Nuuanu Stream
E H1 FWY
W H1 FWY
HHCTCP Rail
PALI HWY
Nuuanu Stream
Pauao Stream
N NIMITZ HWY
N NIMITZ HWY
PALI HWY
S NIMITZ HWY
W H1 FWY
E H1 FWY
E H1 EB VINE
Honolulu Harbor
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Nuuanu Stream Crossing
Location Map
Scale -1:10,000
500 250 0 500 1,000 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
18-Dec-2009

Figure 17.1


Nuuanu Stream
Chinatown Station
Elevated Track
Elevated Platform
N NIMITZ HWY
Proposed Column Location
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Nuuanu Stream Crossing Column Location
Scale -1:800
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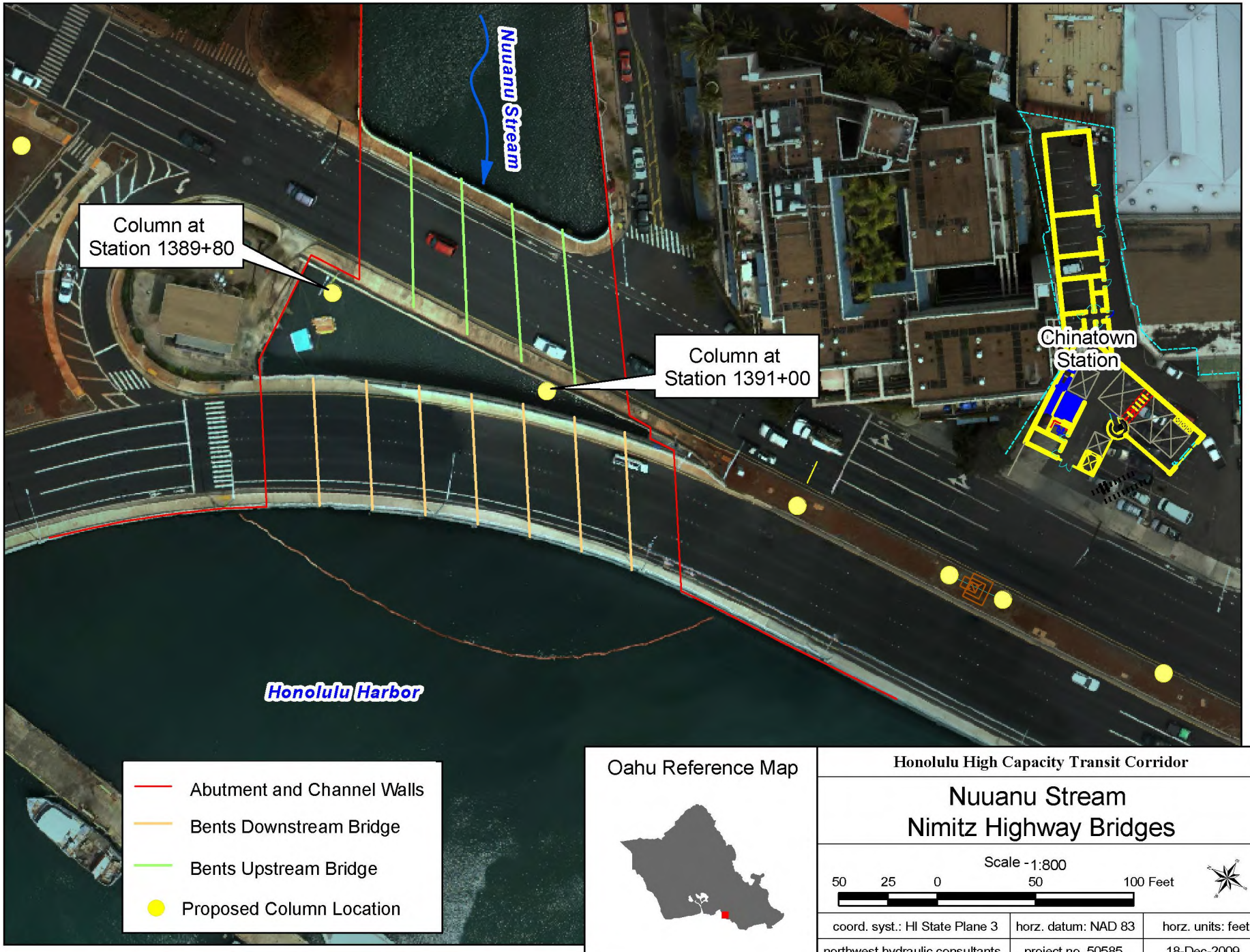
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
18-Dec-2009

Figure 17.2


Nuuanu Stream
Column at Station 1389+80
Column at Station 1391+00
Chinatown Station
Honolulu Harbor
Abutment and Channel Walls
Bents Downstream Bridge
Bents Upstream Bridge
Proposed Column Location
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Nuuanu Stream
Nimitz Highway Bridges
Scale - 1:800
50 25 0 50 100 Feet
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horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
18-Dec-2009

Figure 17.3

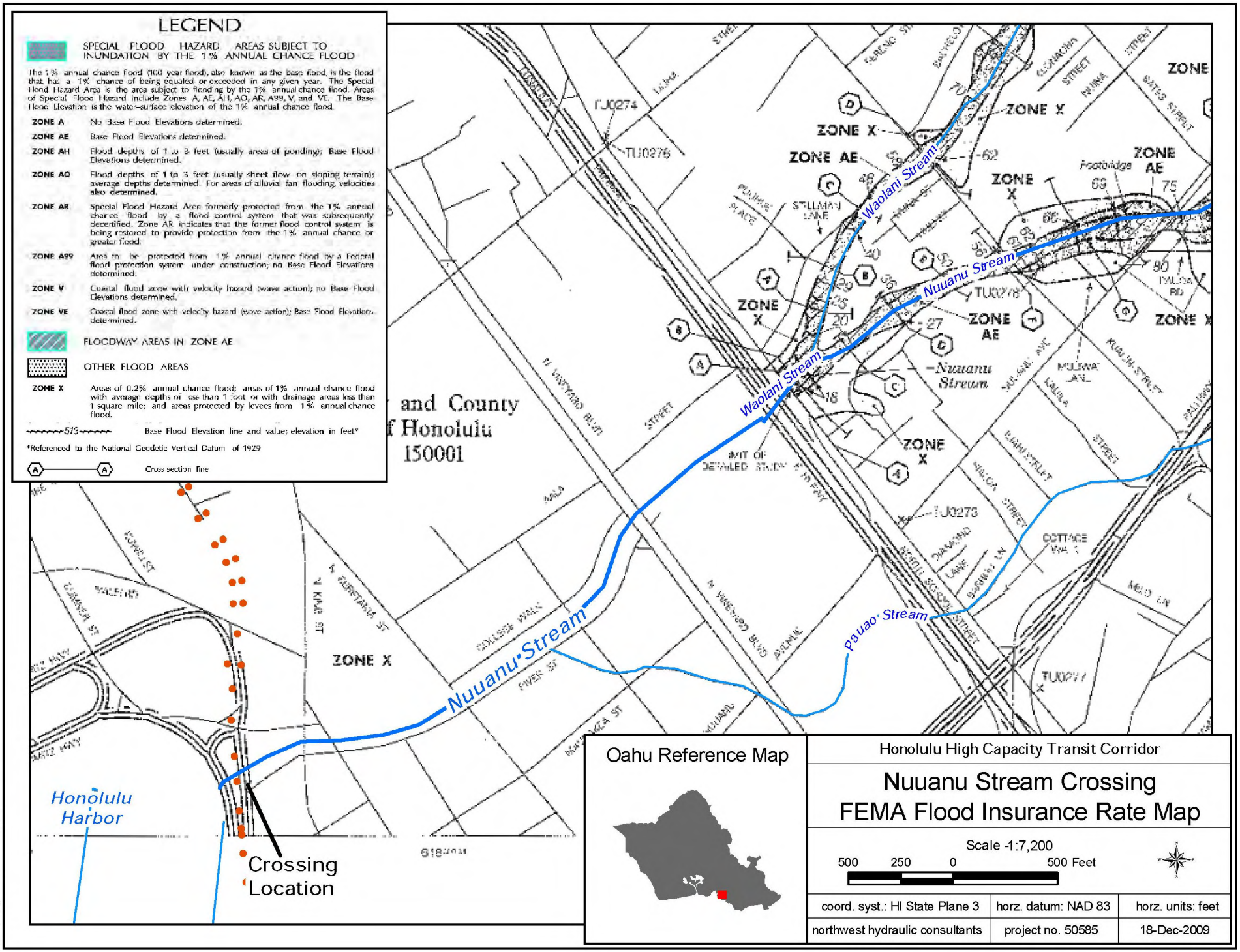
LEGEND
SPECIAL FLOOD HAZARD AREAS SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD
The 1% annual chance flood (100 year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V, and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.
ZONE A No Base Flood Elevations determined.
ZONE AE Base Flood Elevations determined.
ZONE AH Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.
ZONE AO Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
ZONE AR Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
ZONE A99 Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.
ZONE V Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.
ZONE VE Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.
FLOODWAY AREAS IN ZONE AE
OTHER FLOOD AREAS
ZONE X Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.
513 Base Flood Elevation line and value; elevation in feet*
*Referenced to the National Geodetic Vertical Datum of 1929
A A Cross section line
and County of Honolulu
150001
ZONE X
ZONE AE
Waolani Stream
Nuuanu Stream
Nuuanu Stream
Pauao Stream
Nuuanu Stream
Honolulu Harbor
Crossing Location
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Nuuanu Stream Crossing
FEMA Flood Insurance Rate Map
Scale -1:7,200
500 250 0 500 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
18-Dec-2009

Figure 17.4

18 - Ala Moana Coastal Zone

Site Description

A portion of Design Segment G of the HHCTCP near the Ala Moana Shopping Center is located within a mapped FEMA coastal floodplain that is designated as an approximate A Zone. Figure 18.1 shows the study area referred to as Site 18, Ala Moana Coastal Zone. The project will be constructed in a densely developed urban and industrial area of Honolulu. Approximately 4,800 linear feet of elevated guideway and two transit stations will be located in the approximate A Zone shown in Figure 18.1. The Kakaako Station located east of Ward Avenue and the Ala Moana Center Station located in the Ala Moana Shopping center will provide transit passengers with easy access to the commuter trains. There are several 6-foot square guideway support columns located along the track alignment. The Ala Moana Center Station has two entrance buildings; the Mauka Entrance Bldg. and the Makai Entrance Bldg. that will be integrated into other existing structures at the Ala Moana Shopping Center.

Because the project is located in a mapped A Zone, it must not cause a rise in floodplain water levels greater than one foot. Following is a summary of the analyses conducted for this study site.

Hydrologic Analysis and FEMA Study

The Ala Moana Flood Zone Area is mapped as an A Zone. This area was studied and mapped as part of FEMA's National Flood Insurance Program (NFIP) and state-wide flood studies for Oahu published in September 2004. The coast line areas along Ala Moana Park Dr (Figure 18.1) are mapped as an AE Zone with a Base Flood Elevation (BFE) of 4 feet on the west side and 5 feet on the east side. All the HHCTCP project features are located in the approximate A Zone. There is no information in the current Flood Insurance Study (FIS) for City and County of Honolulu, HI (2004) about the methods used to map the Ala Moana Area. NHC is aware that FEMA is in the process of updating the current floodplain mapping for this area, but that information has not been finalized and is not available to the public for use in technical studies.

Hydraulic Analysis

Because there is no information in the FIS about the methods used to map the Ala Moana Area and water levels are not specified in approximate A Zones, NHC used available information from the current Flood Insurance Rate Map (FIRM) to estimate 100-year water levels along the guideway alignment and the amount of floodplain storage volume that will be displaced by HHCTCP project features located in the mapped A Zone. The coast line area along Ala Moana Park Drive is mapped as AE Zone with a BFE of 4 feet on the west side and 5 feet on the east side. NHC used these published BFE elevations; extended them horizontally in the landward direction and generated a BFE surface for the study area that slopes up slightly, from 4 feet to 5 feet, from west to east to match the coastal BFEs. Water depths along the

AR00041255

project corridor were then determined by subtracting the ground surface (project LiDAR topo) from the landward extended BFE surface. The 1-foot contour lines and LIDAR spot elevations provided by Parsons Brinckerhoff (November, 2009) were used to create the ground surface at a 1-foot by 1-foot grid cell resolution.

Analyses determined that both entrance buildings are located above the currently estimated 100-year A Zone water level and that only three of the guideway support columns are inundated. Figure 18.2 shows the locations of the three inundated columns. Photo 18.1 below, shows a ground photo of the approximate location of the inundated column at the intersection of Kamakee and Queen Streets. Table 18.1 summarizes results of how much floodplain storage volume is displaced by the three support columns. Water depths at the center of these columns are estimated to be approximately 0.01 to 0.26 feet deep. These depths were multiplied by the column cross sectional area to develop displaced storage volumes associated with each column (Table 18.1). Based on available information and the methods used to estimate water levels in the approximate A Zone, only 10.44 cubic feet of floodplain storage is lost because of shallow flooding at the base of these three columns. Because the Ala Moana flood area is likely to have very shallow ponding with very low velocities, there can only be an extremely small effect on the 100-year water level (much less then a few hundredths of a foot).

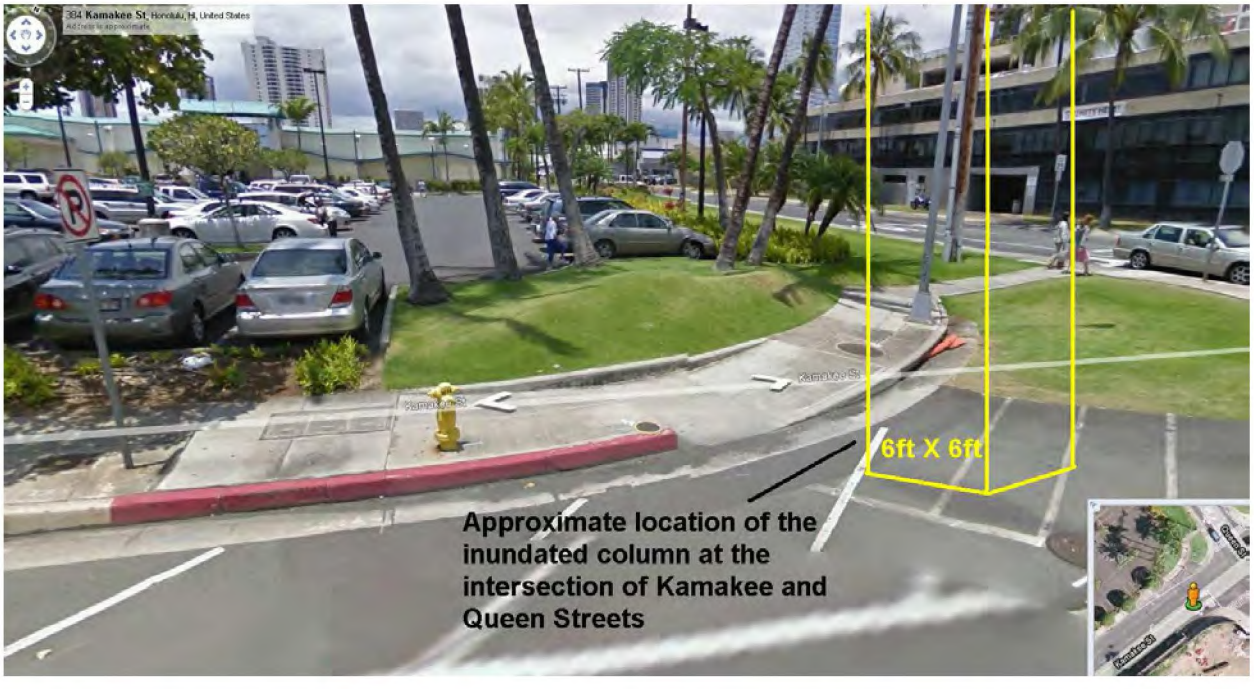

Photo 18.1: Approximate location of the inundated guideway support column at the intersection of Kamakee and Queen Streets.

According to NHC's estimates, the Makai Entrance Building at the Ala Moana Center Station is approximately 0.9 ft above the estimated 100-year flood elevation, and the Mauka Entrance Building is approximately 0.8 ft above the 100-year flood elevation. The Kakaako Station building is approximately 2.6 ft above the 100-year flood elevation. Therefore, neither of the entrance buildings will have any

effects on water levels. Additional analyses of the effects of flooding may be required in the future if FEMA's updated BFEs (water levels) are found to be higher then currently published values.

Table 18.1: Displaced Floodplain Storage Volume Calculations

Project Feature	Inundation Depth (ft)	Cross Sectional Area of Columns (sq ft)	Displaced Volume (cubic feet)
Column at Station: 1474+20	0.26	36	9.36
Northern Column at Station: 1485+00	0.01	36	0.36
Southern Column at Station: 1485+00	0.02	36	0.72
TOTAL VOLUME DISPLACED			**10.44**

Regulatory Compliance and Conclusions

Figure 18.1 shows all of the HHCTCP project features (station buildings and guideway columns) that are located within the current FEMA mapped A Zone. FEMA regulations require any structures built in mapped A Zone result in less then a foot rise in 100-year water elevation. Based on the preliminary hydraulic analyses presented herein, we conclude that the project features will not cause more than a foot rise in the 100-year flood elevation and only approximately 10.44 cubic feet of volume is displaced by the three support columns. This small amount of displaced flood storage volume can be easily compensated for locally through minor project feature modifications during final design.

Study Limitations

There is no information in the FIS for City and County of Honolulu, HI (2004) about the methods used to map the Ala Moana Area. Therefore, NHC used currently available information and simplified methods to estimate 100-year water levels along the HHCTCP guideway alignment located in the approximate A Zone. These results are based on currently published BFEs and information presented in FEMA's September 2004 FIS. If FEMA adopts new mapping for the Ala Moana Coastal flood area in the future, water levels, depths of inundation and volume compensation estimates reported herein may need to be revised based on new information and mapping made available at that time.

Reference

FEMA Flood Insurance Study (FIS) for City and County of Honolulu, HI. September, 2004. Flood Insurance Study Number 15003CV001A

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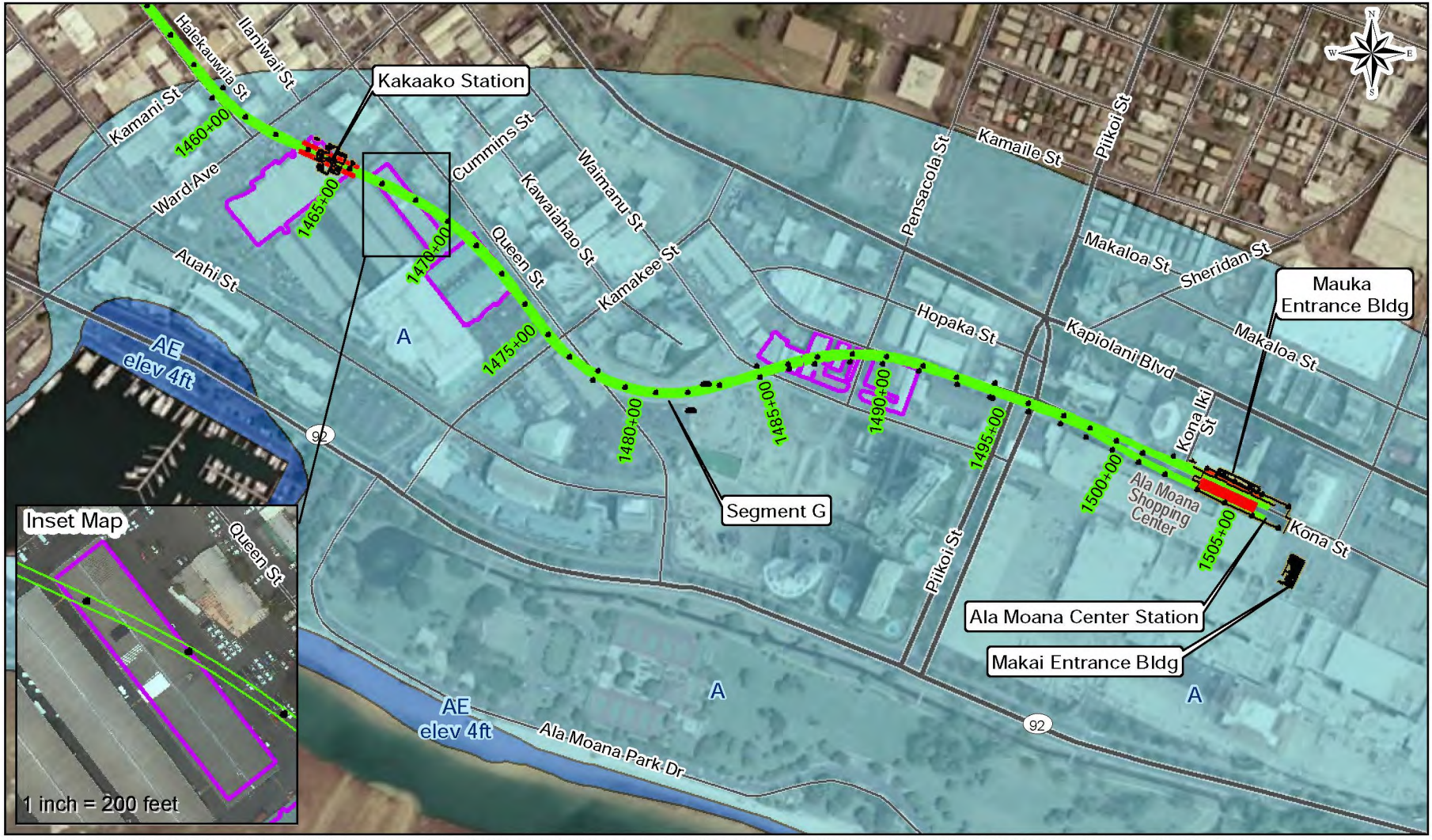


Data Sources: USGS Color Aerial Imagery, 2004. ESRI Roads, 2008. Project Buildings, Support Columns, Platforms, Transit Stations, and Track Alignments, 2009. FEMA Flood Hazard Areas and Streams, City & County of Honolulu, HI, 6/2005.

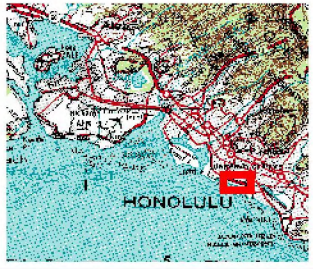
Legend

- Guideway Support Columns
- Platforms
- Track Alignment
- Highways
- Roads

FEMA Flood Hazard Zones
- AE
- A

Existing Buildings in Track Alignment

Oahu Reference Map



Honolulu High Capacity Transit Corridor

Site 18 Ala Moana Coastal Zone

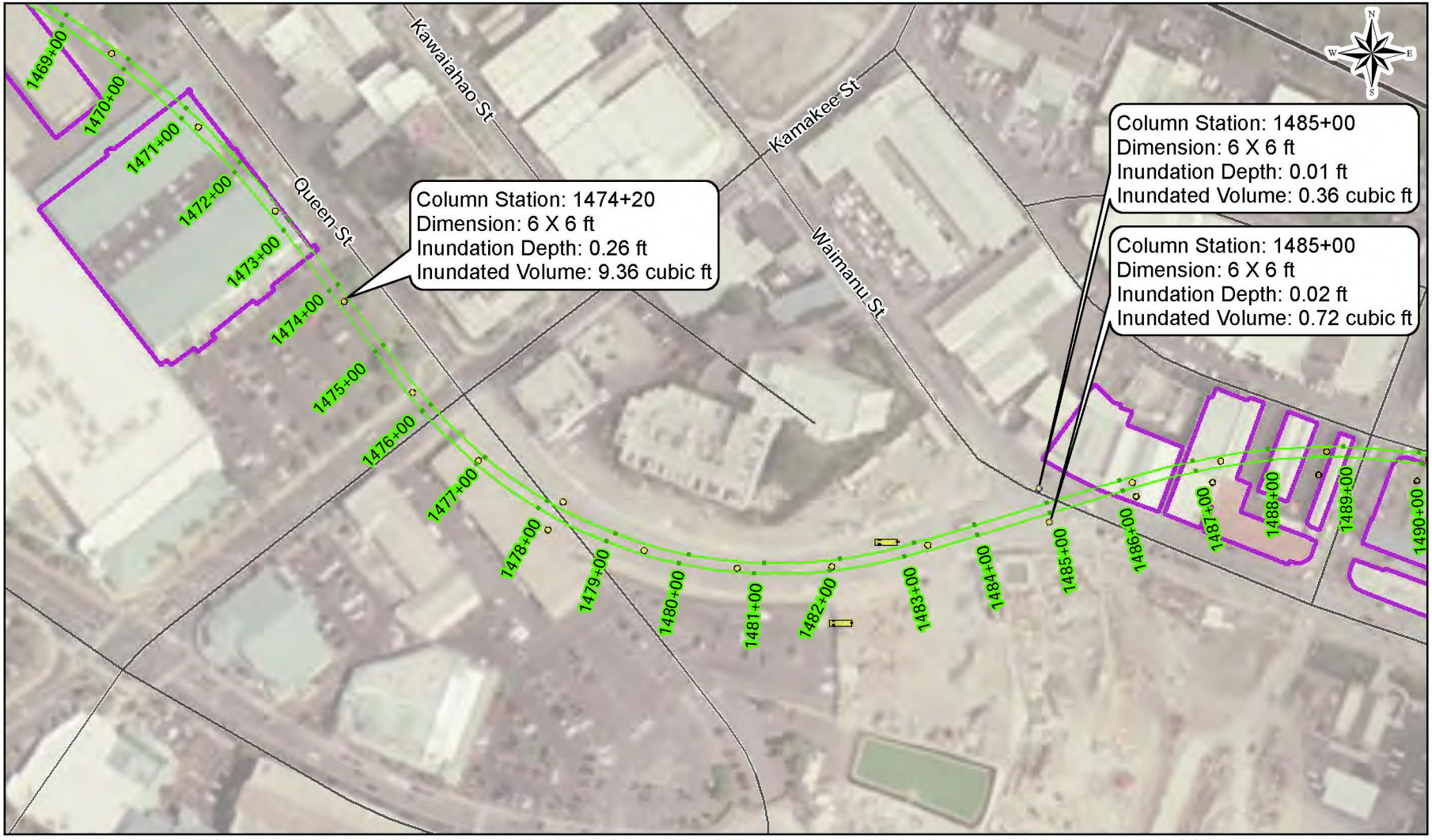
Scale -1:7,200

300 150 0 300 600 Feet

coord. syst.: HI State Plane 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50597	Apr-2010

Figure 18.1

AR00041258

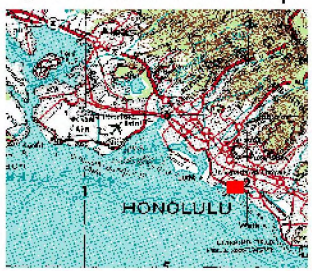


Data Sources: USGS Color Aerial Imagery, 2004. ESRI Roads, 2008. Project Buildings, Support Columns, Transit Stations, and Track Alignments, 2009. FEMA Flood Hazard Areas and Streams, City & County of Honolulu, HI, 6/2005.

Legend

- Guideway Support Columns
- Track Alignment
- Roads
- Buildings in Track Alignment

Oahu Reference Map



Honolulu High Capacity Transit Corridor

Location of Three Inundated Columns

Scale -1:2,400

100 50 0 100 200 Feet

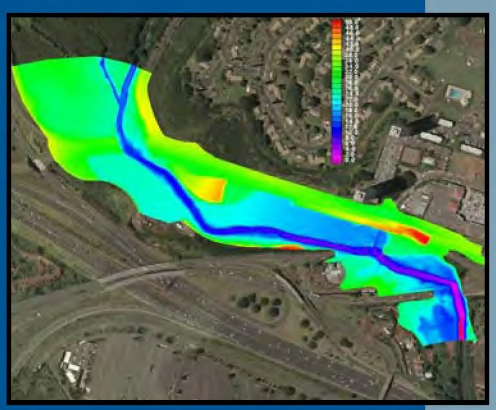
coord. syst.: HI State Plane 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50597	Apr-2010

Figure 18.2

AR00041259

ADVANCED FLOODPLAIN EVALUATIONS FOR PEARL HIGHLANDS STATION AND PARK-AND-RIDE FACILITY – WAIAWA STREAM

DRAFT



Prepared for:

Parsons Brinckerhoff Inc.
1001 Bishop Street, Suite 2400
Honolulu, HI 96813
808-531-7094

Prepared by:

Northwest Hydraulic Consultants Inc.
16300 Christensen Road, Suite 350
Seattle, WA 98188
206-241-6000

April 30, 2010

EXECUTIVE SUMMARY

The proposed Honolulu High-Capacity Transit Corridor Project (HHCTCP) is an elevated rail line that will provide rapid public transportation service in the City and County of Honolulu on the island of Oahu, Hawaii. Northwest Hydraulic Consultants (NHC) was retained to perform hydraulic evaluations with respect to and as required for planning and design of the proposed Pearl Highlands Station and Park-and-Ride Facility (PHSPF), part of the greater HHCTCP.

Portions of the proposed PHSPF are located within the floodway of Waiawa Stream. The City and County of Honolulu Department of Planning and Permitting (DPP) is the lead agency responsible for Flood Hazard District compliance within the City and County of Honolulu. As mandated by FEMA, DPP requires that all segments of the HHCTCP comply with a "No-rise" policy, i.e. have no impact on 100-year flood water levels within floodway zones.

After inspecting the project site, and evaluating the hydrology of Waiawa Stream, NHC developed two detailed hydraulic computer models of Waiawa stream through the proposed project site. The first model utilized the U.S. Army Corps of Engineers HEC-RAS program to create a one-dimensional model of the stream and project site. This model was used to provide a preliminary assessment of project impacts and to evaluate potential mitigation measures to offset those impacts. It was also used to demonstrate that the final project configuration will meet the City and FEMA "No-Rise" requirement. The second model was created using a software code referred to as FESWMS, which is a two-dimensional hydraulic model that was developed for the Federal Highway Administration. This model provides much more detailed hydraulic information than the HEC-RAS model and was used not only to refine the PHSPF design and mitigation scheme to minimize impacts, but also to provide the design team with an understanding of how flow will move through and interact with the project features. This model will be used in future project phases to develop detailed designs for scour and erosion protection.

NHC, working with the PHSPF design team, together have carefully considered how the proposed facility will interact with the stream during floods. The team has used this hydraulic data to help develop a design that will not only satisfy the City and FEMA "No-Rise" requirement, but will also minimize the risks imposed by the stream on the facility and public.

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Advanced Floodplain Evaluations for Pearl Highlands Station and Park-and-Ride Facility – Waiawa Stream

1. INTRODUCTION

The proposed Honolulu High-Capacity Transit Corridor Project (HHCTCP) is an elevated rail line that will provide rapid public transportation service in the City and County of Honolulu on the island of Oahu, Hawaii. Parsons Brinckerhoff (PB), the General Engineering Consultant (GEC) for the project, retained Lyon Associates (LAI) who contracted with Northwest Hydraulic Consultants (NHC) to perform hydraulic evaluations with respect to and as required for planning and design of the proposed Pearl Highlands Station and Park-and-Ride Facility (PHSPF), part of the greater HHCTCP.

According to the existing FEMA flood maps, the proposed PHSPF is in part located within the floodway of the Waiawa Stream. The City and County of Honolulu Department of Planning and Permitting (DPP) is the lead agency responsible for Flood Hazard District compliance within the City and County of Honolulu. As mandated by FEMA, DPP requires that all segments of the HHCTCP comply with "No-rise" policy, i.e. have no impact on water levels within floodway zones. The existing FEMA effective model of Waiawa Stream is generally too coarse to be able to accurately simulate the complex details of the proposed project, and to reliably assess the potential impacts of the proposed station and guideway support columns on area water levels. Furthermore, newer, more refined survey and LiDAR data are available to better represent the present geometric conditions of the channel and floodplain. It was therefore decided to develop new and more up-to-date hydraulic computer modeling to re-evaluate present hydraulic conditions and future with-project conditions.

Two modeling programs were utilized for NHC's evaluation of the project. In each case, steady-state assumptions were made, which is typical for bridge analyses and appropriate for this location in which floodplain storage is insignificant. In most flood insurance studies and for the purposes of completing a No-rise analysis for adherence to FEMA regulations, one-dimensional analysis is typically applied (e.g. the effective HEC-2 model). FEMA's guidelines were developed in a one-dimensional framework and are not particularly two-dimensional friendly. The review process is more streamlined and straight forward if based upon one-dimensional modeling, in particular by using the Army Corps of Engineers' successor to the HEC-2 software, HEC-RAS, which is the current accepted standard tool. Therefore, an HEC-RAS model was first developed to provide preliminary evaluation of initial project alternatives and ultimately to assess impacts of the project under a regulatory 100-yr flood event. Because of the complex hydraulics at the site, including an extremely skewed approach angle of the stream to the upper Farrington bridge just downstream of the project, and in order to accurately aid in the design and evaluation of project features, a two-dimensional model was subsequently also developed as a more detailed evaluation and design tool for the project. The FESWMS (FST2DH) model program by the Federal Highway Administration was applied at this site, due to its capabilities to simulate bridge features and secondarily because it is approved by FEMA. Results from the two-

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dimensional model were then used to help guide the placement and appropriate realignment of cross sections in the final HEC-RAS model, to ultimately demonstrate No-rise in the project.

The main objective of this study included the following:

(1) Evaluate existing flow conditions in the Waiawa Stream floodplain within the project reach;
(2) Assess potential impacts of the proposed project development alternatives for the PHSPF on the existing Waiawa Stream flow conditions; and
(3) Evaluate and help fine-tune project alternatives needed to reduce or eliminate impacts to water levels in the floodplain caused by the project, including mitigation as needed.

NHC's work involved field inspection of the project area, collection of the data needed for computer simulation of the flooding in the vicinity of the project station, hydrological analysis, hydraulic modeling, and evaluation of various project alternatives. Revision of the effective FEMA flood maps was outside the scope of this study. Following is a description of work and services performed by NHC.

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2. STUDY AREA

This report uses the standard cardinal or ordinal directions in its description and orientation of locations and features (i.e. the North, South, East and West compass points). Along Waiawa Stream, North is generally the upstream direction and analogous to Mauka, South is downstream or Makai, East is the left bank (facing downstream) and analogous to Diamondhead or Honolulu, and West is the right bank or Ewa. At the project site itself, the stream follows a slightly different alignment, whereby the left bank becomes somewhat more Mauka than Diamondhead and the right bank more Makai than Ewa. To avoid confusion, we simply adhere to the cardinal directions but also make use of upstream/downstream and left/right terminologies.

The proposed PHSPF transit station structures will be located alongside the Waiawa Stream, approximately 1.5 miles upstream of Middle Loch at the north end of Pearl Harbor, and above the influence of tidal fluctuations. While the proposed elevated rail line (guideway) alignment for the HHCTCP will generally follow the Farrington Highway alignment, the PHSPF facility itself will be located alongside the Kamehameha Highway in Pearl City, near its junction with Farrington Highway. The location is approximately 10 miles northwest of Honolulu on the Hawaiian island of Oahu (Figure 2.1). It will consist of an elevated fixed guideway transit system across Waiawa Stream, a transit center station, parking garage, and commuter drop-off areas. Structural components of the PHSPF, including guideway support columns (piers), retaining and building walls, and fill will be constructed within the 100-year regulatory floodplain and a portion of the floodway of the Waiawa Stream, downstream of the Kamehameha Highway bridge and upstream of Farrington Highway. Figure 2.2 depicts the features of the original design for the proposed PHSPF, including project footprint and a three-dimensional rendering.

The climate of the island of Oahu is generally mild, with fairly uniform temperatures. The wet season extends from October through April, the dry season from May through September. During heavy winter rainfalls, the Pearl City area is subject to frequent flooding from the local streams. The major source of overland flooding is Waiawa Stream and its tributaries.

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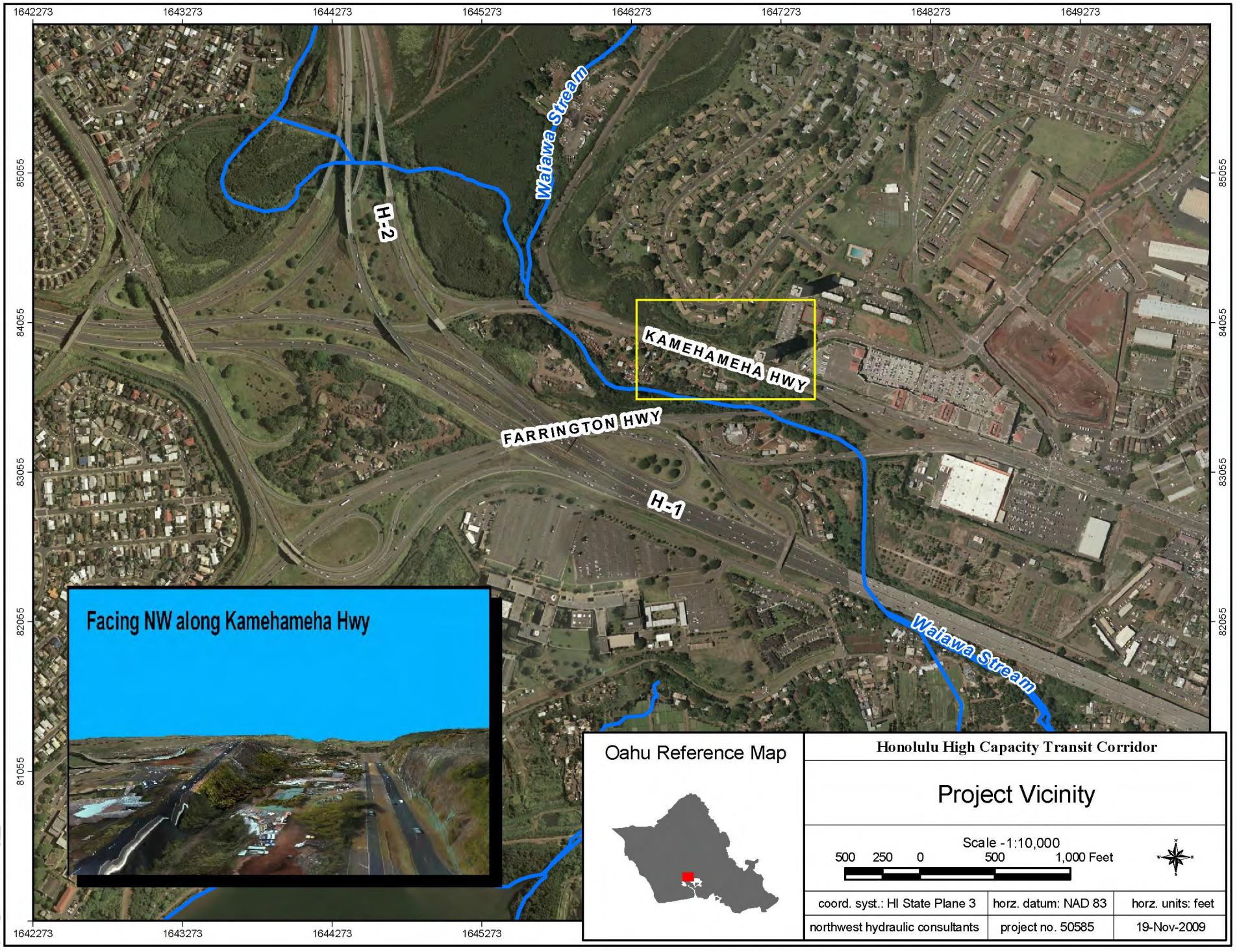
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84055
83055
82055
81055
Waiawa Stream
H-2
KAMEHAMEHA HWY
FARRINGTON HWY
H-1
Waiawa Stream
Facing NW along Kamehameha Hwy
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Project Vicinity
Scale -1:10,000
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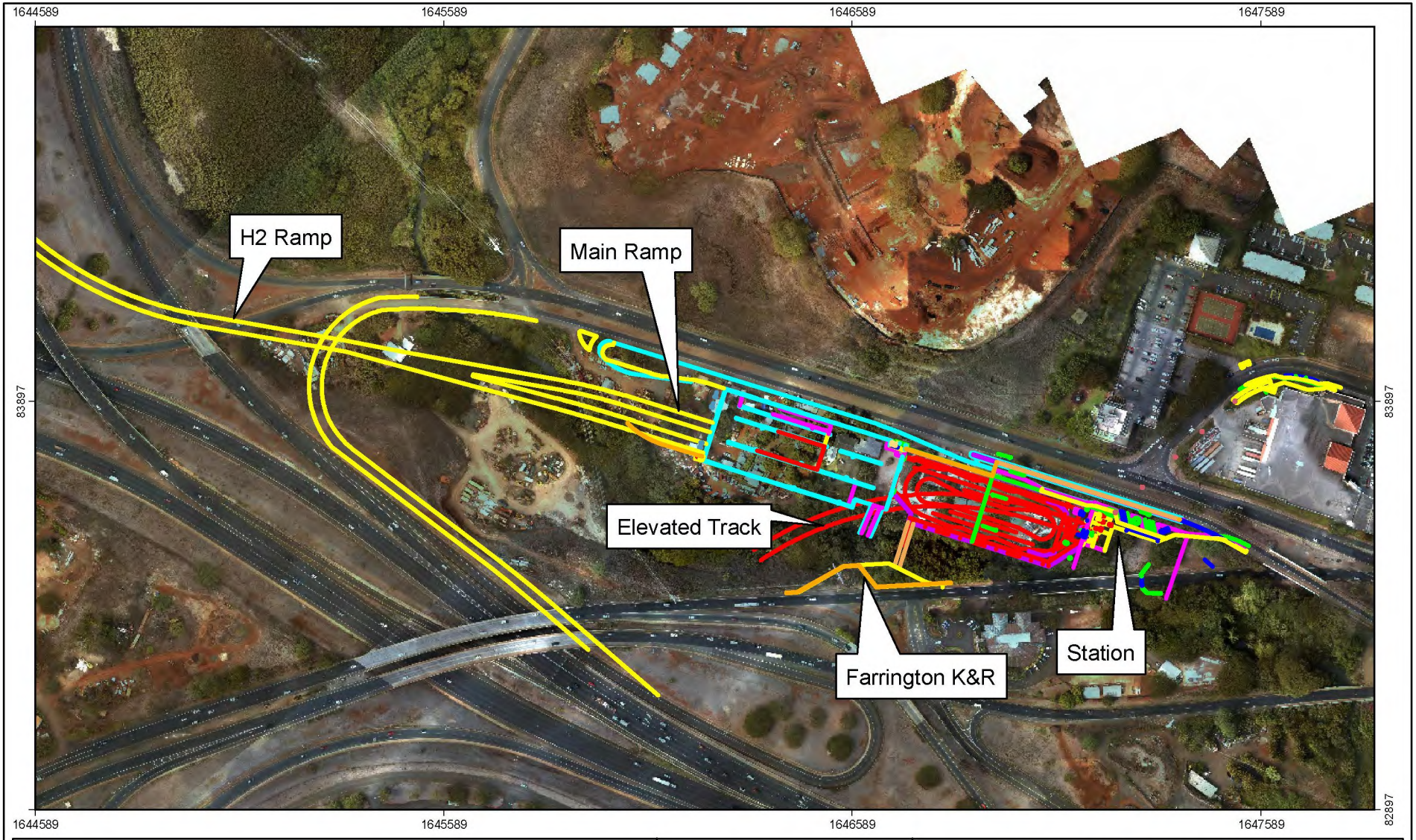
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
19-Nov-2009

Figure 2.1



Legend

- Elevated Transit Facility
- Elevated Parking Garage
- Ramps

Oahu Reference Map

Honolulu High Capacity Transit Corridor

HHCTCP Pearl Highlands Transit Station

Scale -1:4,200

200 100 0 200 400 Feet

coord. syst.: HI State Plane 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50585	19-Nov-2009

Figure 2.2

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3. FIELD INVESTIGATIONS AND DATA COLLECTION

This chapter presents results from a field inspection of the study area conducted jointly by PB, NHC, and LAI on January 15, 2009 and data collection activities undertaken to support development of the models of the study stream floodplain area.

3.1 Site Inspection

A field inspection of the study area was conducted on January 15, 2009. The purpose of the site visit was to document channel and floodplain conditions of the stream, pertinent hydraulic features, locations where ponding and ineffective flow are likely, and high water mark observations from past events. Results of the field inspection are discussed herein, with photographs presented in Appendix A.

Waiawa Stream drains a portion of the wet Koolau mountains, and generates the largest 100-year instantaneous peak streamflow on Oahu. In general, the upper watershed has narrow floodplains with steep mountain gradients that quickly convey flow to the project reach. The PHSPF reach of Waiawa is bounded by the two Kamehameha Highway bridges upstream and the two Farrington Highway bridges downstream (see Figure 3.1). The floodplain upstream of Kamehameha Highway consists of old cane fields and agriculture land. Panakauahi Gulch is a major tributary that joins Waiawa less than 500 feet upstream of the Kamehameha Highway bridges. The two Kamehameha Highway bridges were built in 1949 and 1951 and provide a significant constriction to Waiawa Stream. Based on residents' comments and the site visit, these bridges collect large amounts of debris on the bents and overtop during large floods (see Photos 1 and 2).

After crossing Kamehameha Highway, Waiawa Stream changes from a generally southerly to a southeasterly downstream flow direction, through the area known as the "Banana Patch". Through most of this reach the channel is located at the toe of a steep tall slope on the right bank (facing downstream); Farrington Highway and Interstate H-2 are located at the top of this slope. The left bank floodplain has been filled over the years with rocks and debris (see Photos 3 and 4) and developed with numerous residential and accessory buildings between the channel and Kamehameha Highway which parallels the stream. There is, however, a section of more natural land that has not been significantly filled located on the left bank just upstream of the Farrington Highway bridge (see Photo 5). The channel banks within the Banana Patch reach have been protected with various rocks, sticks, concrete pieces and car bodies (see Photos 6 and 7) and there are large trees on both banks that provide a relatively continuous canopy. Based on interviews with Banana Patch residents, debris from the channel is routinely cleaned following flood events (see Photos 8 and 9). Floodplain areas adjacent to the stream are occupied by dwellings, outbuildings, heavy equipment, scrap piles, etc.

Two Farrington Highway bridges cross Waiawa Stream immediately downstream of the Banana Patch (see Figure 3.1). The upstream bridge carries the westbound lanes of Farrington Highway and was constructed in 1933. This bridge is a significant constriction to Waiawa Stream due to

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its small opening, heavy debris collection, presence of fill downstream of the bridge, and large piers which are misaligned with the direction of flow through the bridge, resulting in a general skew of more than 50 degrees (see Photos 10 to 13). Thus, the upstream Farrington Highway bridge significantly controls water surface elevations in the PHSPF reach upstream. The flow in Waiawa channel is directed nearly parallel to the bridge on the upstream side until the flow hits the left bank abutment and is forced to turn through the bridge. These flow patterns have caused local erosion along the left bank upstream of the bridge and scour at the base of the two piers located nearest the left bank (Photos 13 and 14). On the downstream side of the bridge the right bank has been filled, which significantly blocks four of the six bridge spans from conveying much flow and has led to deposition under the bridge (Photos 15 and 16).

The reach between the two Farrington Highway bridges is approximately 600 ft long. The left overbank is mostly natural area and heavily vegetated (Photo 17), while the right bank has been filled and includes numerous buildings on both the Hawaii Laborers' Union Training Program and Naval properties. There is a submerged concrete weir that was used by the USGS for Waiawa Stream flow gauging from 1952 to 2004, located about 50 ft upstream of the downstream Farrington Highway bridge (Photo 17). The downstream eastbound Farrington bridge (Photo 18) was built in 1951 and also constricts flow in Waiawa Stream. The bridge deck partially interacts with large floods and the right bank approach fill blocks over 250 ft of floodplain until the road is overtopped. Downstream of this bridge is a relatively straight and natural reach (see Photo 19) for approximately 800 ft until Interstate H-1 crosses the stream.

During the site visit several residents were interviewed about historic flooding along Waiawa Stream. Figure 3.2 shows the locations of several observed anecdotal high water marks from the March 21, 1991 and December 11, 2008 flood events.

3.2 Channel Survey

Existing channel survey information along Waiawa Stream was needed to support model development and hydraulic analyses for the HHCTCP. Following the site inspection of the study area, NHC then developed a detailed channel survey program. The survey was conducted by ControlPoint Surveying, Inc. (ControlPoint) in the spring of 2009. The survey data included channel cross sections at various locations along the study reach, beginning upstream of Kamehameha Highway and extending downstream to the Interstate H-1 crossing. Altogether, approximately 20 cross sections were surveyed along Waiawa Stream. They also surveyed key feature elevations at the Kamehameha and Farrington Highway bridge crossings, elevations for the high water marks identified during the site visit, and miscellaneous floodplain feature elevations. The survey points are shown on a topographic map of the study reach in Figure 3.3.

3.3 Topographic Data

Bare ground surface topography was needed to support development of the two-dimensional model of the study area, and for mapping with-project inundation limits. Topographic data used in this study included 1 ft contour interval Light Detection and Ranging (LiDAR) based on

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topography data within the HHCTCP corridor obtained by John Chance Land Surveys, Inc. (JCLS) in 2007 (official data for the project) and 5 ft by 5 ft grid FEMA LiDAR data developed in 2006 for FEMA Hurricane Study and covering the entire island coast including Pearl Highlands and Waiawa Stream area. These topographic data sets were provided to the project team by the City and County of Honolulu.

The 2007 project corridor topography data and 2006 FEMA LiDAR data were compared in ArcGIS to ensure their consistency and compatibility. The comparison indicated a close agreement (within one foot) of ground elevations between these topographic data sets for most overlapping areas. A comparison was also made between the ground survey data along Waiawa Stream and the project topography. The difference in ground elevations in these datasets was small in overbank regions, typically within 0.5 ft. The ground survey data was used for the channels and around bridge structures, with the project topography used for overbank areas. The only location the FEMA LiDAR dataset was used for the Waiawa Stream analysis was downstream of the Farrington Highway bridges beyond the extents of the project topographic dataset. Shaded ground contours of the final merged TIN developed and used by NHC are shown in Figure 3.3.

3.4 Other Data Sources

Bridge plans were provided by PB staff for the four Kamehameha and Farrington Highway crossings of Waiawa Stream. FEMA effective Flood Insurance Study (FIS), DFIRM mapping, and HEC-2 hydraulic models were obtained directly from FEMA by NHC. Oceanit identified (flagged) and ControlPoint surveyed in the ordinary high water marks along the Waiawa Stream channel between Farrington and Kamehameha Highways. These marks were then connected by NHC to estimate the ordinary high water mark boundary along the entire study reach.

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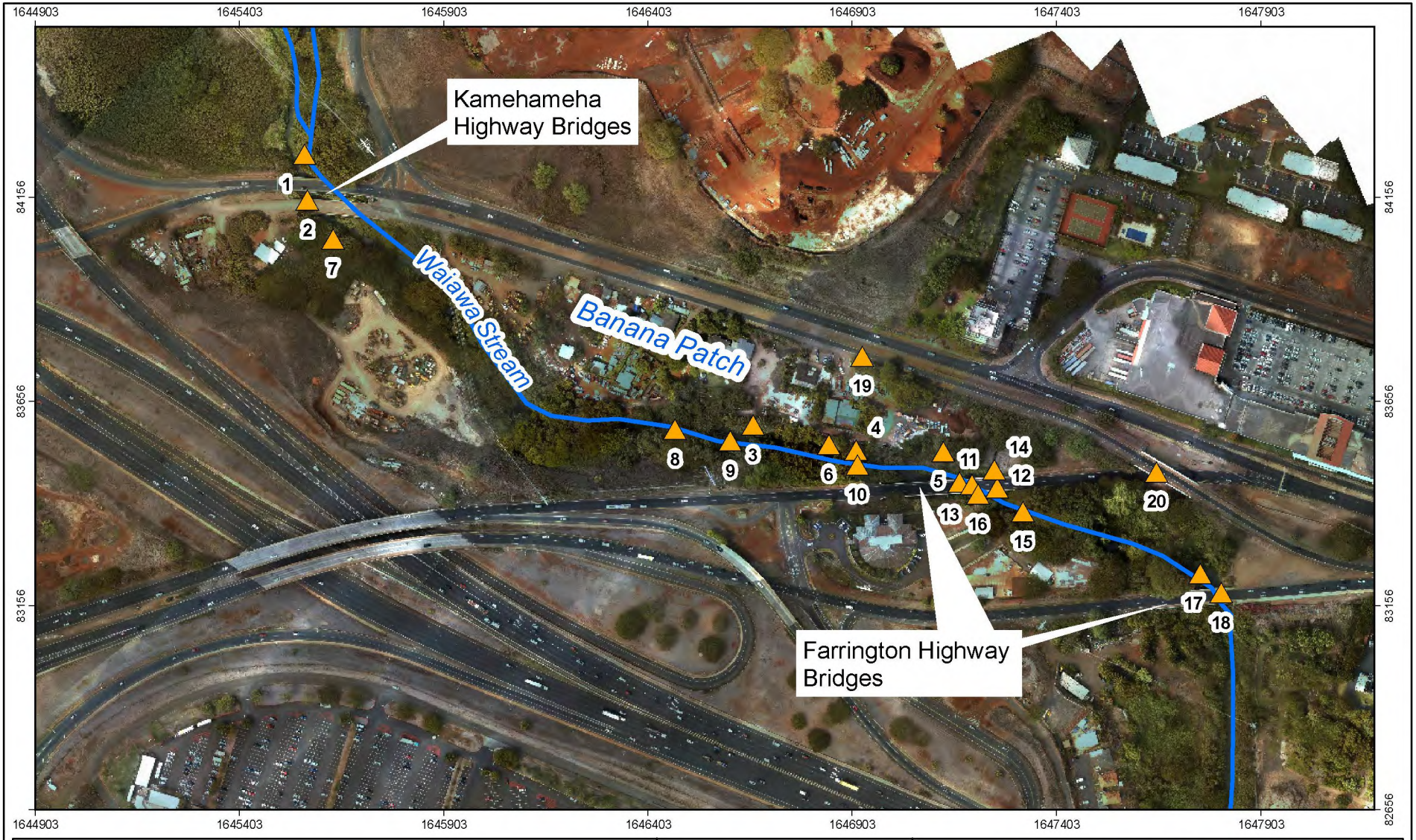

Legend

▲ Photograph Location

Oahu Reference Map

Honolulu High Capacity Transit Corridor

Map of Approximate Photographed Locations During Site Visit

Scale -1:4,200

200 100 0 200 400 Feet

coord. syst.: HI State Plane 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50585	19-Nov-2009

Figure 3.1

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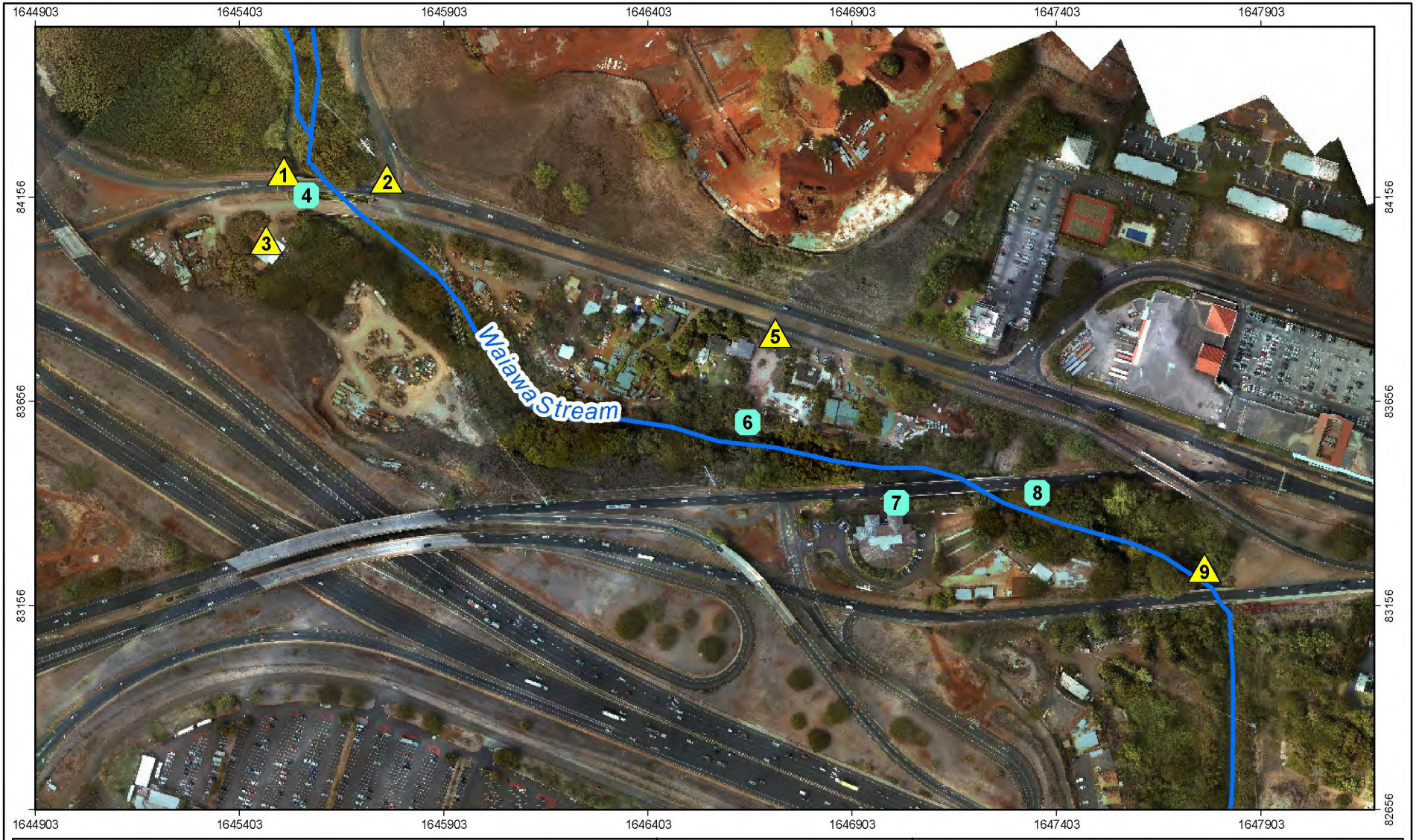

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High Water Marks

1991 Event

2008 Event

Oahu Reference Map

Honolulu High Capacity Transit Corridor

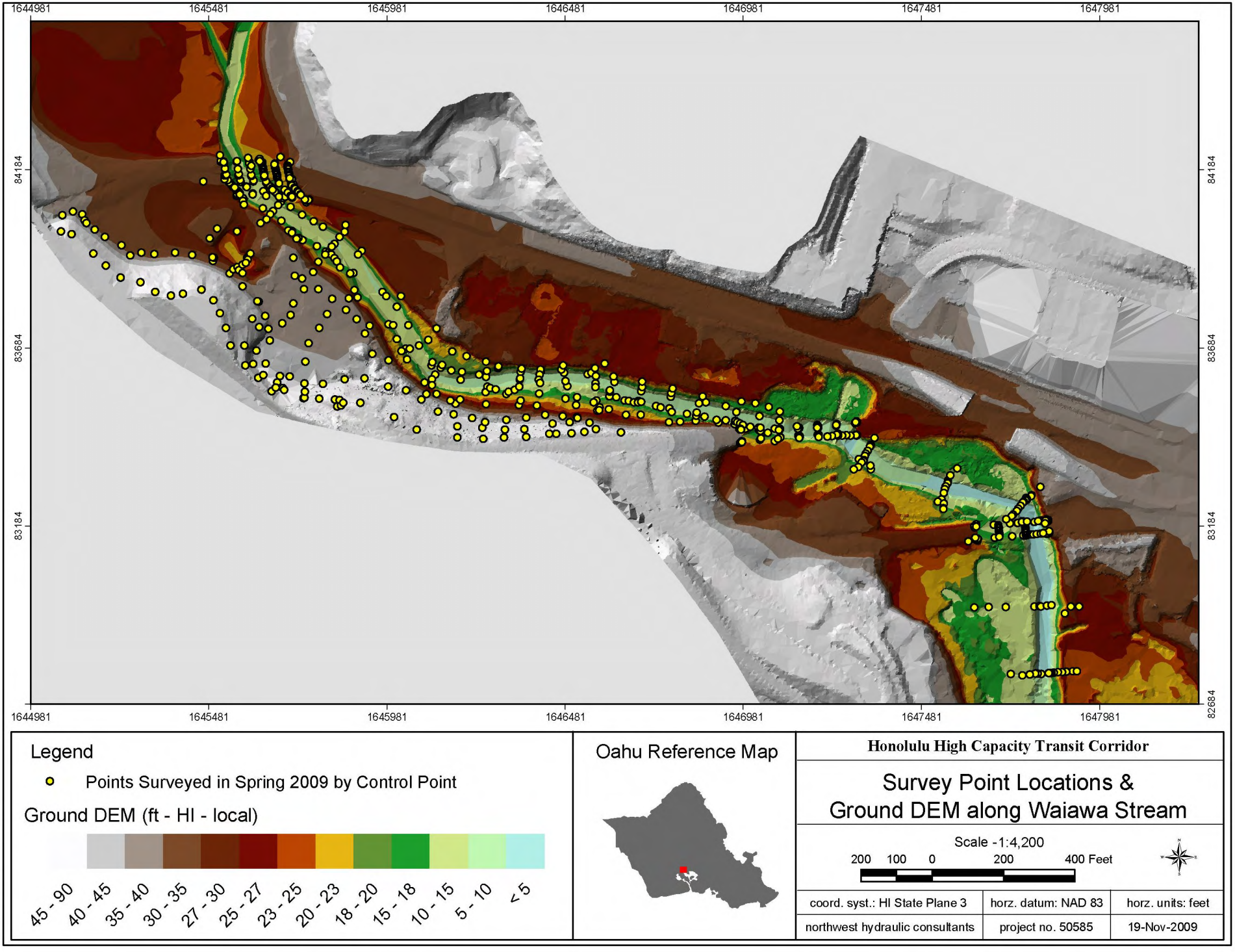
Locations of High Water Marks Obtained During Site Visit

Scale -1:4,200

200 100 0 200 400 Feet

coord. syst.: HI State Plane 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50585	19-Nov-2009

Figure 3.2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1645481
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84184
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Legend
Points Surveyed in Spring 2009 by Control Point
Ground DEM (ft - HI - local)
45 - 90
40 - 45
35 - 40
30 - 35
27 - 30
25 - 27
23 - 25
20 - 23
18 - 20
15 - 18
10 - 15
5 - 10
< 5
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Survey Point Locations &
Ground DEM along Waiawa Stream
Scale - 1:4,200
200 100 0 200 400 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
19-Nov-2009

Figure 3.3

4. HYDROLOGY

The purpose of the hydrologic analysis was to review and analyze existing discharge data from prior studies and stream gages, and to update these data as necessary to provide inputs to support the hydraulic analysis and modeling, including the published FIS peak flow quantiles for Waiawa Stream.

4.1 Review of FEMA Effective FIS Hydrologic Analysis

The effective FIS for the City and County of Honolulu (FEMA 2004) was originally published on November 20, 2000 and subsequently revised and re-issued on September 30, 2004. The FIS documents hydrologic and hydraulic analyses and provides maps for dozens of streams on the island of Oahu. These analyses utilized a variety of methods and data sources to compute flood quantiles and map the Base Flood Elevations (BFEs).

The effective FIS utilized a nomograph shown on Plate 6 of the City and County of Honolulu Storm Drainage Standards (DPP 2000), which estimates 100-year peak discharge for streams and conduits as a function of drainage area. Three lines represent peak discharge as a power function of drainage area for 3 distinct geographic regions, windward Oahu (highest peak flows per unit area), central Oahu (2nd highest peak flows per unit area) and west Oahu (lowest peak flows per unit area). According to a label, the nomograph is based on USGS data as of 1988. The effective FIS states that 10-year and 50-year peak flow values were not computed; however, the 500-year peak is described as having been estimated using the ratio of 500-year to 100-year peak flow derived from data from USGS Gage 16216000.

4.2 Re-Analysis of Waiawa Stream Frequency Curve

NHC used the entire available stream record of peak annual flows (52 years, water years spanning 1953-2004) from USGS gage 16216000 to fit a Log-Pearson III frequency curve. The USGS gage was located on the upstream side of the eastbound lanes of Farrington Highway (lower bridge) in very close proximity to proposed HHCTCP facilities.

The USGS data were fit according to Bulletin 17b procedures (WRC 1981) using the HEC-SSP computer program. Detailed output from the HEC-SSP frequency analysis is provided in Appendix B. Summary results are shown in Figure 4.1 and in Table 4.1. 100-year and 500-year quantile estimates from the re-analysis are within 5% of the values reported in the effective FIS. This suggests that the Waiawa record may have been used to formulate the City's flood nomograph for the Central Oahu region.

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4.3 Application of Hydrologic Analysis to Hydraulic Modeling of Waiawa Stream for HHCTCP Facilities

The hydraulic analysis required for the PHSPF examined flow, water level, and velocity pattern conditions for a range of flood discharges from the 2-year to the 100-year events. The flood quantiles shown in Table 4.1 (both the FIS and revised HHCTCP values) are suitable discharge inputs for steady state, one- or two-dimensional hydraulic modeling of the entire reach of Waiawa Stream that is expected to be affected by the proposed PHSPF. Because the FIS 100-year discharge is very close to the revised value (within 5%), the subsequent hydraulic analyses for both project design and No-rise compliance were therefore based upon the presently adopted FIS value of 34,000 cfs.

Table 4.1 Waiawa Stream Flood Quantile Estimates

Study	Location	Drainage Area (sq mi)	2-yr	5-yr	10-yr	50-yr	100-yr	500-yr	Source or Method
Effective FIS	"Mouth"	27.34					34,000	43,800	Plate 6, City and County of Honolulu Storm Drainage Standards, 1988 plus ratio from LP-III, Bulletin 17b.
HHCTCP (NHC), Effective FIS verification	USGS 16216000 (upstream of E.bound Farrington Hwy)	26.40	8,477	15,032	19,740	30,507	35,128	45,838	Bulletin 17B Frequency Analysis via HEC-SSP, 52 years of record. (WY 1953-2004)
Change from Effective FIS			N/A	N/A	N/A	N/A	3.3%	4.7%	

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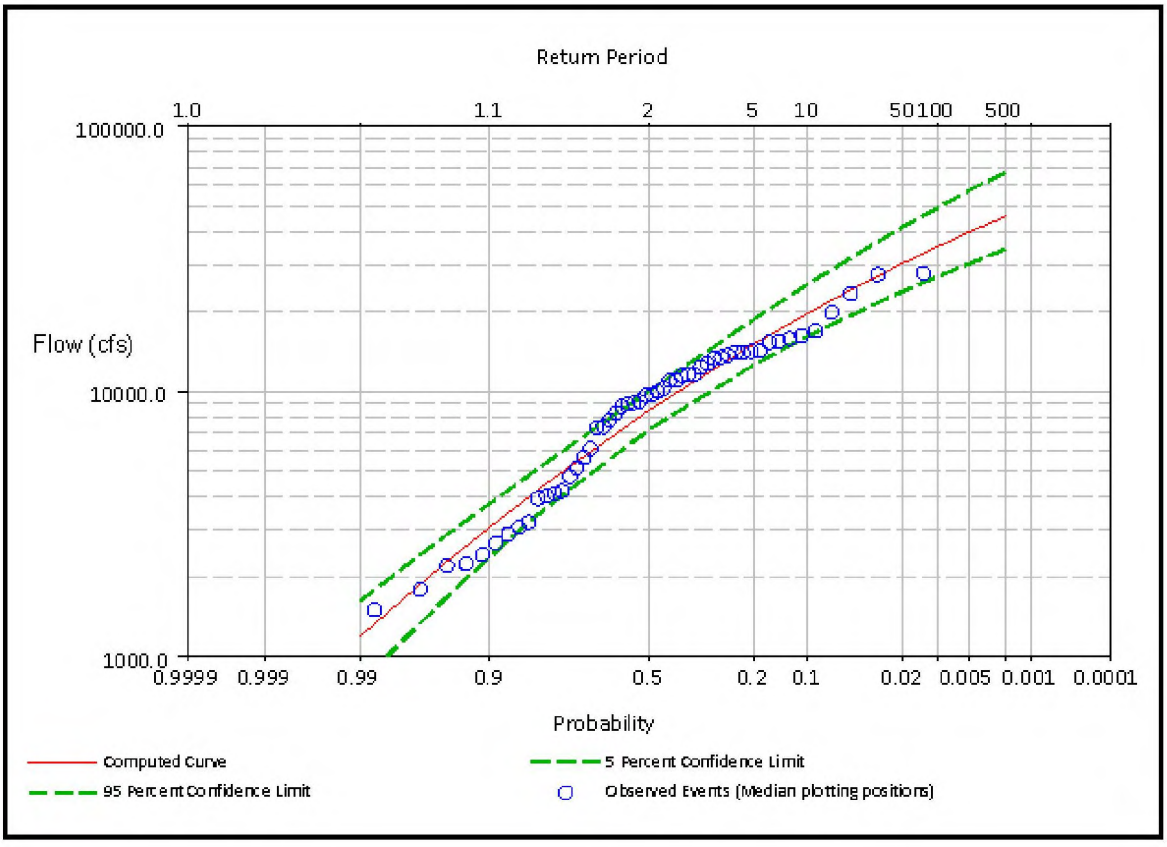
Figure 4.1 Flood Frequency Curve for Waiawa Stream near Pearl City, USGS Gage 16216000.



5. HEC-RAS MODEL DEVELOPMENT

5.1 FEMA Effective Model

The effective FEMA HEC-2 model for Waiawa Stream was obtained from FEMA and reviewed. The HEC-2 model was imported into and converted to HEC-RAS (USACE 2009), initially retaining the original cross sections and bridge geometries from the HEC-2 model. This HEC-RAS model is the "duplicate effective" model for this study. It was determined however that the model is too coarse to be a reliable and useful tool for assisting the design or evaluating impacts of the complex details of the PHSPF project. Furthermore, as detailed in Chapter 3, new topographic data are available for the channel and floodplain, which would be more up-to-date than the effective FEMA cross sections. Because of these reasons, it was determined that a more refined model was needed to provide a defensible tool for evaluating realistic impacts of project alternatives and demonstrating to agencies No-rise of the final project configuration. Therefore, using the new project topography along with the Waiawa channel cross-sectional surveys that were obtained, numerous cross sections were added to the duplicate effective HEC-RAS model (imported HEC-2 model) to create a much more detailed model of the project reach. This is referred to as the "refined" model in the next section.

5.2 Refined Model

The refined HEC-RAS model begins approximately 4,700 ft upstream (north) of Kamehameha Highway and extends 12,100 feet downstream to the Pearl Harbor Bike Path crossing. From the upstream end of the model to within approximately 1,000 ft of the Kamehameha bridges, cross sections from the HEC-2 model were used. From this point, extending through the project site, past the Farrington Highway bridges and Interstate H-1, new cross sections were used. Downstream from Interstate H-1, cross sections from the HEC-2 model were generally retained. The refined reach spans from downstream of cross section G to upstream of cross section L at the Kamehameha Highway bridges in the effective FIS. Bridge data was converted from the HEC-2 model, input into HEC-RAS and then adjusted as needed. The modeling of the bridges was not significantly modified from the effective HEC-2 model at this stage of the analysis. The model cross section locations and alignments used in the refined model through the project site are illustrated in Figure 5.1.

5.3 Calibration and Verification

Hydraulic model calibration is a process by which model input parameters, most typically coefficients such as roughness and other empirical inputs including coefficients for weir flow, bridge and contraction/expansion losses, are adjusted (within reasonable limits) in order to more closely match observed high water levels from past floods. This requires knowledge, or at least a reasonable estimation, of the actual discharge from the historic event. Ideally, high water and discharge data are available for more than one past event, in which case the adjusted coefficients

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from the calibration event can be tested against a second (or third, etc.) event. If the additional event(s) perform well, without requiring re-adjustment of the input parameters, the model is said to be verified.

Measured and anecdotal high water data are available from the 1991 flood, which measured 27,600 cfs at the USGS Waiawa gage 16216000 and is approximately a 25-year event and only slightly smaller than the 27,950 cfs event of record in 1982 (no high water marks are available for the 1982 flood). A more recent, but substantially smaller event, occurred in 2008 after the USGS gage ceased operation. The discharge for this event has been roughly estimated to be on the order of a 5-year event, or approximately 12,000 cfs. Table 5.1 lists the location and estimated elevations of the various measured high water marks for these two flood events. The high water mark locations are identified on Figure 3.2 (included with the figures in Chapter 3).

Table 5.1 High Water Mark Locations Used in Calibration

High Water Mark ID	Event	Location	Estimated Elevation (ft)
1	Mar 21, 1991	Overtopped guardrail on upstream Kamehameha Highway bridge	36.6
2	Mar 21, 1991	Light pole at intersection was submerged	34.8
3	Mar 21, 1991	Water above the third step of Irwin Kawano's staircase	33.1
4	Dec 11, 2008	Debris on low chord of upstream Kamehameha Highway bridge	29.5
5	Mar 21, 1991	Kamehameha Highway was not significantly overtopped	32.0 to 34.0
6	Dec 11, 2008	High bank was not overtopped	24.5
7	Dec 11, 2008	Debris downstream of bridge	20.4
8	Dec 11, 2008	Debris downstream of bridge	20.8
9	Mar 21, 1991	USGS Gage at concrete weir	24.2

Variables adjusted in the HEC-RAS model during the calibration included bridge routine selection, roughness values, cross section orientation, and contraction and expansion losses. Table 5.2 lists typical roughness values used in the calibrated model. Refer to the tables in Chapter 7 for the final calibrated values of contraction and expansion losses. Simulated water surface profiles and observed high water marks for the calibration events are shown in Figure 5.2. One will notice that the flood profiles are slightly higher than a number of the high water marks. There is significant uncertainty in the accuracy of many of the marks because they are based upon the very approximate anecdotal information provided by local residents (the 1991 event occurred many years ago). It was determined that for design purposes, it would be best for the HEC-RAS model to compute flood profiles that are slightly conservative.

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Table 5.2 Calibrated Manning's Roughness Values in HEC-RAS Model	
Land Use	**n Value**
Road or Paved	0.015
Channel	0.025
Shade	0.035
Mostly Open	0.040
Light Vegetation	0.060
Heavy Vegetation	0.070
Dense Vegetation	0.080

5.4 HEC-RAS FEMA Floodway Model

Once the refined HEC-RAS model was complete, NHC modified the channel cross sections to confine flows within the limits of the effective FEMA floodway. This was done for two reasons. First, it was to ensure that the PHSPF was designed to accommodate future development within the flood fringe (i.e. on the floodplain landward of the floodway boundary). Under current FEMA rules, the portion of the floodplain landward of the floodway boundary could theoretically be completely filled which would eliminate flood conveyance in the flood fringe. This in-turn would elevate flood levels within the stream corridor which needs to be accounted for in the facility design. Second, when a structure will be built within the FEMA floodway, one is required to show that the features will have no impact on FEMA floodway water surface elevations.

To ensure that the cross sections in the refined model include the correct floodway boundaries, NHC identified the boundary limits using data from the existing FEMA Digital Flood Insurance Rate Map (DFIRM) for the Waiawa Stream, Flood Insurance Study (FIS), and the effective HEC-2 floodway model.

Once the refined floodway model was complete, the 100-year floodway water surface profile was computed and compared to the baseline 100-year floodplain profile. Refer to Chapter 7 for the final comparison of floodway and 100-year floodplain water surface profiles.

5.5 Preliminary Evaluation of Project Alternatives

Once the refined HEC-RAS model was complete, NHC used the model to evaluate the original PHSPF layout. The model revealed that the project, as initially proposed, would have caused an unacceptable impact in the 100-year water surface and flood inundation. NHC then used the model to make some very rough estimates of the amount of excavation that would need to occur along the project site to eliminate the impact of the proposed facility. At this point, NHC set the

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HEC-RAS model aside and developed a two-dimensional model of the project area in order to further refine the project and mitigation measures. Development and use of the two-dimensional model is presented in the next chapter.

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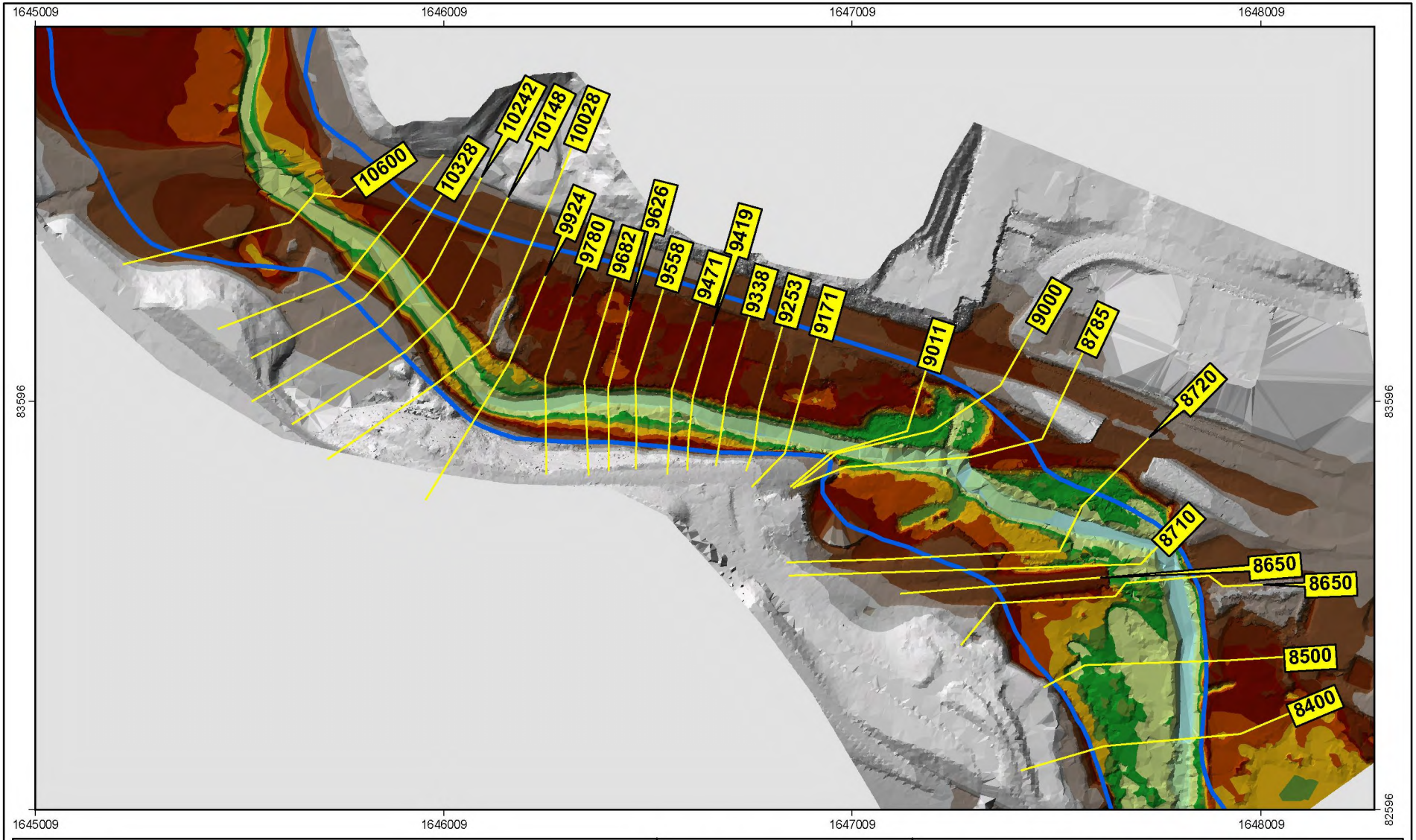


Legend

Model Cross-Sections — FEMA Floodway

Ground DEM (ft - HI - local)

45 - 90 | 40 - 45 | 35 - 40 | 30 - 35 | 27 - 30 | 25 - 27 | 23 - 25 | 20 - 23 | 18 - 20 | 15 - 18 | 10 - 15 | 5 - 10 | < 5

Oahu Reference Map

Honolulu High Capacity Transit Corridor

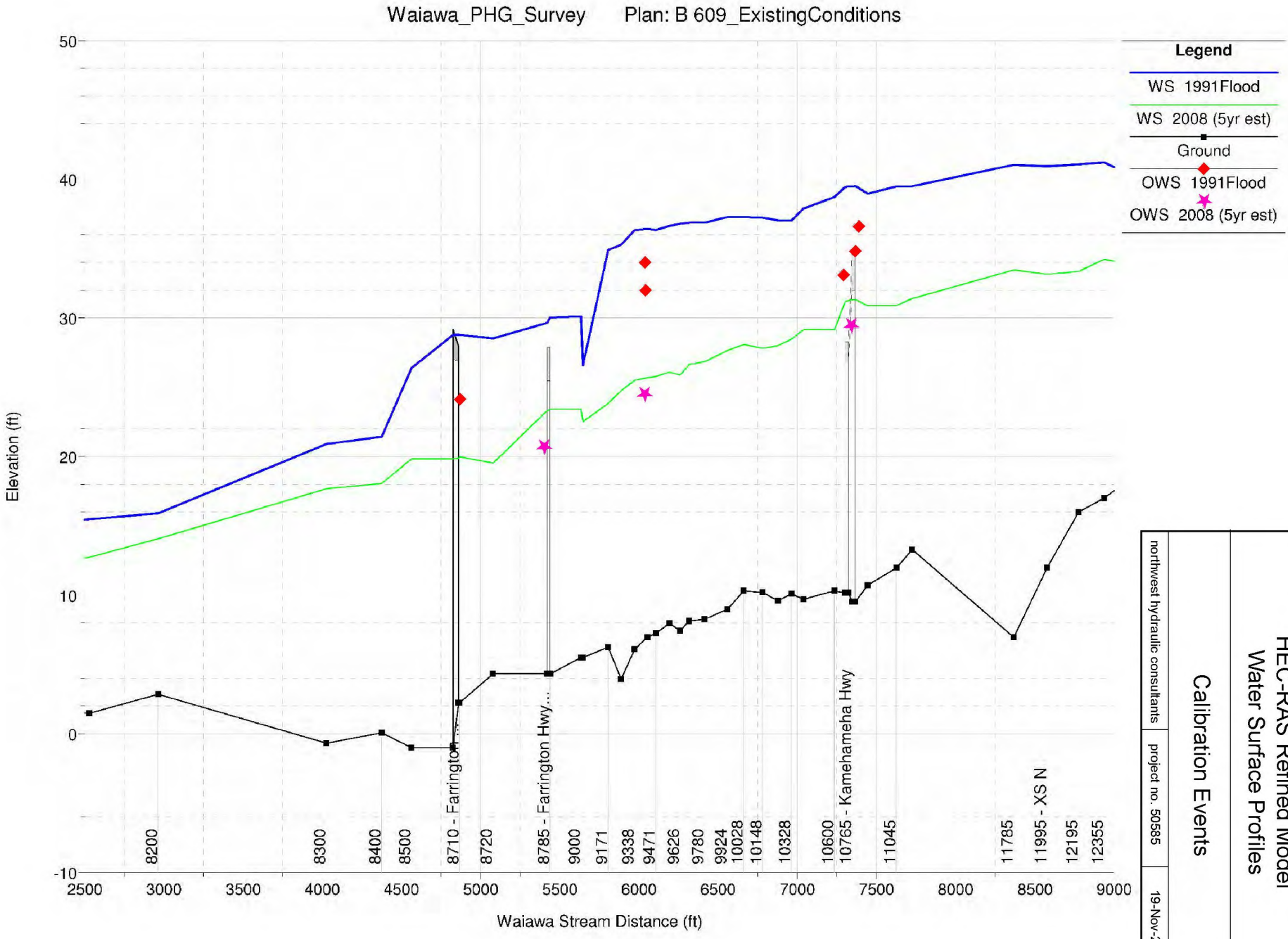
Refined HEC-RAS Model Cross-Section Locations

Scale - 1:4,200

200 100 0 200 400 Feet

coord. syst.: HI State Plane 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50585	19-Nov-2009

Figure 5.1



Honolulu High Capacity Transit Corridor

HEC-RAS Refined Model
Water Surface Profiles

Calibration Events

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Figure 5.2

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6. FESWMS MODEL DEVELOPMENT

6.1 Model Geometry

Because of the complex hydraulics at the site and in order to most accurately aid in the design and evaluation of project features, a two-dimensional FESWMS model (FHWA 2003) was developed. Using the ground TIN based upon the LiDAR topography and Waiawa channel cross-sectional surveys obtained for the project, a detailed computational mesh for the two-dimensional model of Waiawa Stream was developed. GIS methods using ESRI products and the SMS program by Aquaveo were utilized to process the TIN and formulate the mesh, which contains the ground elevations at every computational point in the model. The existing conditions FESWMS mesh and topographic shading is shown in Figure 6.1. The model domain extends approximately 3600 ft along Waiawa Stream and through the proposed PHSPF. The modeled reach begins about 600 ft upstream (north) of Kamehameha Highway, extends through the project site, then past the upper and lower (westbound and eastbound, respectively) Farrington Highway bridges to a downstream boundary approximately 400 ft upstream (north) of Interstate H-1. The chosen model boundary locations are located a sufficient distance from the project site to minimize the effects of assumed boundary conditions on the model solution in the study reach.

The model incorporates and simulates all existing fill and structures. Several of the bridges overtop significantly and become highly submerged. At the Kamehameha Highway crossing, the approach fills to the bridge will overtop during large floods and are therefore included in the computational mesh. On the east side approach, the overtopping flow will continue to be conveyed down the highway (also meshed) and towards the Farrington crossing for some distance before re-entering the Waiawa Stream. At the upper westbound Farrington bridge, the roadway will overtop extensively at the bridge and along its east approach, which is therefore included in the mesh. The approach fills to the lower eastbound Farrington bridge however, are higher. The east approach will remain high and dry; however, the west approach may overtop at very large events but would not completely submerge the roadway. Along the west approach, therefore, weir segments were specified in the FESWMS model to calculate overtopping using the standard empirical weir formula. At each bridge, piers were also coded into the model to account for their respective hydraulic losses.

Roughness coefficients, in terms of Manning's n values, were initially assigned based upon bed material and vegetation cover, based on site visit observation and analysis of aerial imagery. Adjustments were made to account for blockages due to existing buildings. Further adjustments were made during calibration of the model, in order to better replicate measured or estimated high water marks from recent flood events.

6.2 Calibration

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Observed high water mark data were discussed previously and presented in Table 5.1 for flood events occurring in 1991 and 2008, with 1991 nearly the event of record at 27,600 cfs (approximately a 25-year event) and the 2008 event considerably smaller with an estimated discharge of 12,000 cfs (on the order of a 5-year event). Given the uncertainty of the discharge estimate for 2008 and that it is considerably smaller than the design 100-year event, it was decided not to perform a two-dimensional simulation for this event. Even for the one-dimensional HEC-RAS modeling, the benefit of the 2008 simulation is arguable; however, it is a simple effort to include the second event. With FESMWS, however, it is not a trivial exercise to simulate an additional event of considerably lower discharge, requiring gradual ramping down of the upstream inflow and many adjustments to the elements that dry out in the process.

Some of the hydraulic losses that are reflected in the Manning's n roughness coefficient of one-dimensional models including HEC-RAS (such as some transition losses and other losses of two-dimensional nature such as sinuosity) are implicitly accounted for in the equations and solution of the FESMWS model. Therefore, roughness coefficients can be less in a calibrated two-dimensional model such as FESMWS, primarily reflecting bed and vegetation roughness alone and not other losses that the one-dimensional HEC-RAS model cannot implicitly handle. This became evident during the calibration of the FESMWS no-project model, as it became necessary to reduce some of the HEC-RAS calibrated n values in order to match the observed data from the 1991 flood. Table 6.1 lists the Manning's n roughness coefficients resulting from the calibrated two-dimensional model. Figure 6.2 plots the computed water surface contours for the FESMWS simulation of the 1991 flood.

Table 6.1 Calibrated Manning's Roughness Values in FESWMS Model

Land Use	n Value
Road or Paved	0.015
Bridge	0.020
Channel	0.025
Bridge Overbank	0.030
Mostly Open	0.035
Light Vegetation	0.045
Heavy Vegetation	0.055
House	0.500
Upper Farrington Bridge	0.700

During the calibration process, a range of Manning's n values was tested throughout the model domain. In addition to calibration, various other tests and sensitivity runs were also performed with the model, to ensure the integrity of the input and reliability of the output. The upper (westbound) Farrington Highway bridge significantly overtops and experiences pressure flow. Various methods were tested in FESWMS to account for the additional head loss due to the pressure flow. Specifying ceiling (low chord) elevations within FESWMS, which is the

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prescribed approach for the program, proved to be unreliable, and there have been documented problems with the pressure flow option in FESWMS. Other methods tested to increase the head loss included increasing the pier blockage and the roughness coefficients under the bridge. The final model relies upon the roughness method, with the Manning's n at upper Farrington increased such that the bridge loss matches that predicted with HEC-RAS (see Table 6.1). The lower Farrington bridge is higher and does not experience full pressure flow. Upstream of the project, the Kamehameha bridges become completely submerged under large floods due to backwater ponding such that pressure losses are not significant. After initial model testing and calibration was completed, final model runs for existing (no-project) conditions were then conducted.

6.3 Existing Condition 100-Year Flood Characteristics

The calibrated existing (no-project) conditions FESWMS model was simulated for the FIS 100-year flow conditions (34,000 cfs, see Table 4.1). This model was used to evaluate the existing flow conditions along the Waiawa Stream project reach during the regulatory 100-year peak flow. Water surface elevations, flood depths, and velocities are presented in Figures 6.3, 6.4, and 6.5, respectively. The water surface elevation compares reasonably well to the calibrated HEC-RAS model, but with much greater detail in terms of local variation in water surface across the channel and floodplain than is provided by the HEC-RAS model. The flood depths indicate widespread and deep flooding, including on several of the roadways and bridges as mentioned in the previous section (Kamehameha highway at the bridge and eastward, and the upper Farrington Highway).

6.4 FESWMS FEMA Floodway Model

The presently defined FEMA floodway was also simulated using the FESWMS two-dimensional model (see section 5.4 for a discussion on the development of the effective floodway). This was accomplished by disabling (turning off) computational elements outside of the floodway, which is akin to a complete blocked obstruction of each element (i.e. complete encroachment). Water surface elevations were computed for the with FEMA floodway scenario, and are presented in Figure 6.6. Figure 6.7 plots the difference, in terms of increase in flood depth, due to the floodway compared to the existing condition FESWMS 100-year baseline (no floodway) simulation. With the exception of a few isolated areas away from the project, the results show compliance to FEMA's maximum one-foot rise criteria.

6.5 With Project 100-Year Flood Characteristics

After the existing conditions FESWMS model was created, and the FEMA floodway confirmed, the two-dimensional model was used to evaluate the proposed project. The project, as initially proposed, would have caused an unacceptable impact in the 100-year water surface and flood inundation. Therefore, the model was used to evaluate refinements to the design of the PHSPF

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site features. NHC provided input to the design team during the project alternative refinement process. Figure 6.8 plots the computed floodway water surface elevations with the final project design in place but without any mitigation (i.e. compensatory excavation), while Figure 6.9 plots the difference, in terms of increase in floodway water depths, due to the proposed PHSPF project. FEMA requires No-rise in the regulatory floodway. As Figure 6.9 shows, the proposed project without mitigation would cause a rise in the floodway water surface. Therefore, it is necessary to mitigate the floodway impact, which is discussed in the following section.

6.6 No-Rise Mitigation Design Features

It became clear from the modeling (Figure 6.9), that additional mitigation features would become necessary in order to eliminate unacceptable impacts to the flow conditions through the project area or elsewhere along Waiawa Stream, in particular an increase to the 100-year floodway water surface elevation. Mitigation alternatives, primarily in the way of excavation and fill removal, were suggested and selected per consultation with the design team. Incremental refinements to the mitigation measures were evaluated and fine tuned with the FESWMS model, per agreement with the design team, until a satisfactory and feasible solution was obtained which eliminated project impacts. All mitigation elements were confined to above and outside of the ordinary high water mark boundary as defined by Oceanit.

Components of the final mitigation scheme are depicted in Figure 6.10, and include the following features:

- Less fill / more piers at Kiss & Ride
- Less fill / more piers along Kamehameha Highway
- No shear walls to obstruct flow
- Restore floodplain bench on right bank just downstream of Kamehameha bridges
- Restore floodplain bench on left bank under entire transit facility/garage
- Restore floodplain bench on right bank at westbound Farrington bridge
- Restore floodplain bench on left bank between Farrington Highway bridges

6.7 Final Project Alternative

Figure 6.11 plots the computed floodway elevations for the final project alternative, with mitigation, showing the lateral extent of flooding through the modeled reach. Figure 6.12 plots the difference, in terms of increase in floodway water depths, due to the PHSPF project. It is evident from this plot that, for all practical purposes, the project with proposed mitigation is predicted to not cause any significant increase in the 100-year floodway water surface. Small localized increases do persist, but guidelines from FEMA and the preference of the City is to use the one-dimensional HEC-RAS model to evaluate and certify the final no-rise condition.

When evaluating impacts with a two-dimensional model, it is not unusual for spurious nodes to show numerical increases, and even for other localized areas to demonstrate nominal impact. Recommended procedure is to "average" these localized impacts with the change in water

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surface across the entire channel and floodplain (i.e. cross-sectionally average the two-dimensional results). Under such an approach, the small rises shown in Figure 6.12 would disappear.

In general, it is shown that in most areas the project with mitigation results in a significant reduction in the 100-year floodway water surface. Note that other factors were involved in the overall design of the mitigation program, including keeping the water surface below the low chord of the first floor of the parking garage. Also, in order to produce, practically speaking, No-rise in certain areas, other areas actually show a noticeable decrease (Figure 6.12).

Computed 100-year floodway depths and velocities, with the final project alternative and required mitigation, are depicted in Figures 6.13 and 6.14, respectively. The velocities in Figure 6.14 were compared to existing condition velocities (Figure 6.5) to produce a velocity difference map which is presented in Figure 6.15. These figures can be used to further assess scour and erosion potential, and channel stability issues with the project in place; which will be performed in the future during detailed project design.

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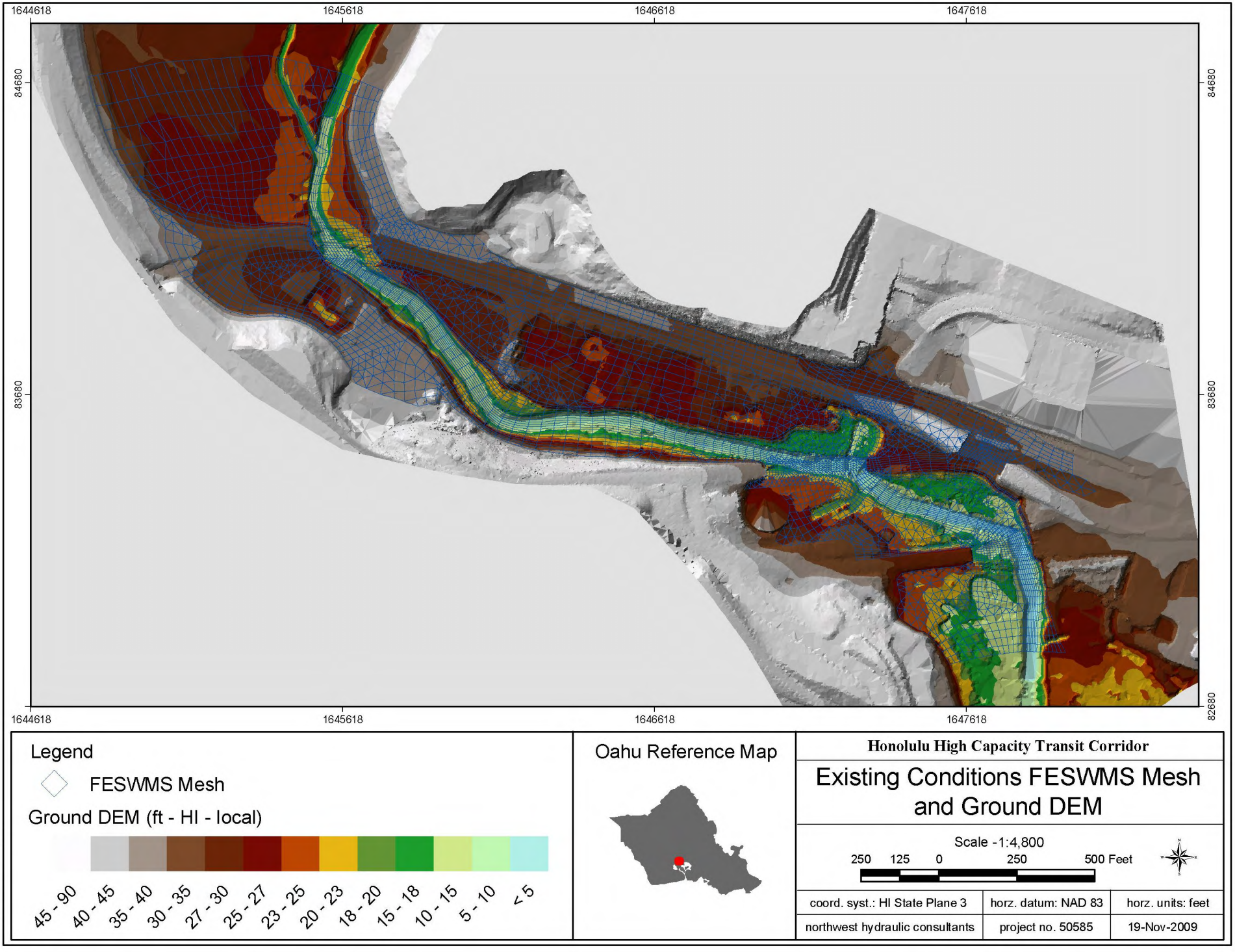

1644618
1645618
1646618
1647618
84680
83680
82680
Legend
FESWMS Mesh
Ground DEM (ft - HI - local)
45 - 90
40 - 45
35 - 40
30 - 35
27 - 30
25 - 27
23 - 25
20 - 23
18 - 20
15 - 18
10 - 15
5 - 10
< 5
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Existing Conditions FESWMS Mesh and Ground DEM
Scale - 1:4,800
250 125 0 250 500 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
19-Nov-2009

Figure 6.1

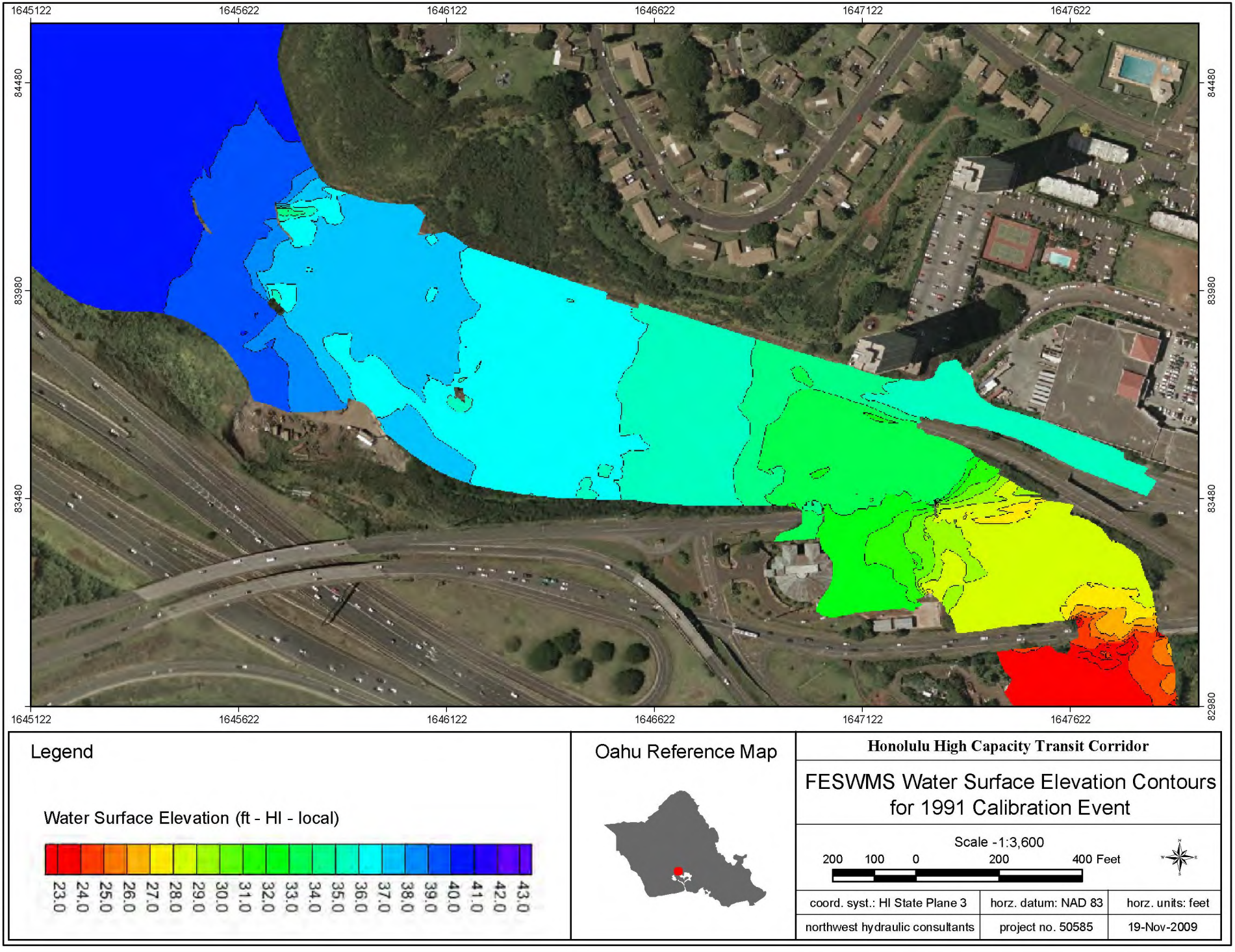
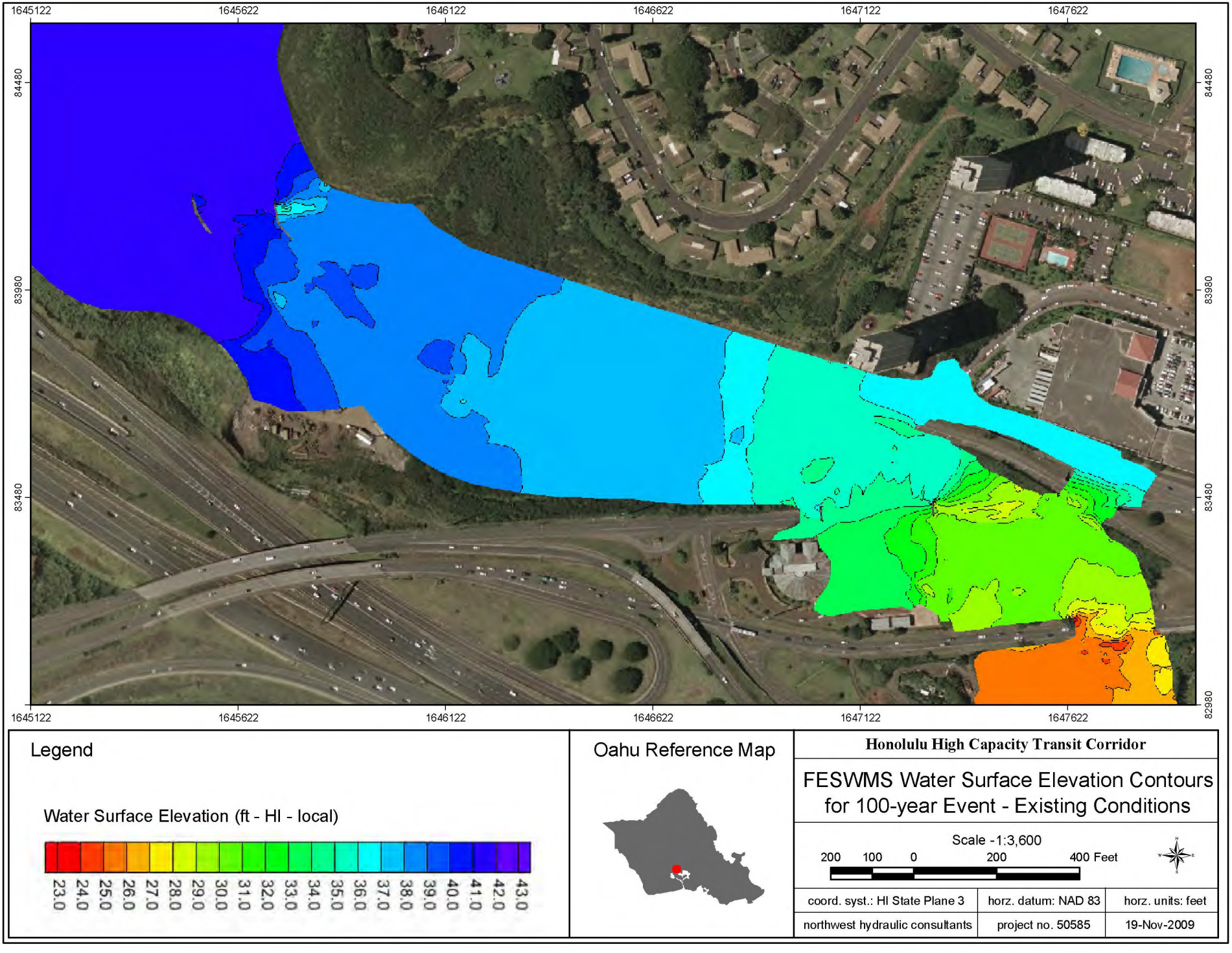

1645122
1645622
1646122
1646622
1647122
1647622
84480
83980
83480
82980
Legend
Water Surface Elevation (ft - HI - local)
23.0
24.0
25.0
26.0
27.0
28.0
29.0
30.0
31.0
32.0
33.0
34.0
35.0
36.0
37.0
38.0
39.0
40.0
41.0
42.0
43.0
Oahu Reference Map
Honolulu High Capacity Transit Corridor
FESWMS Water Surface Elevation Contours for 1991 Calibration Event
Scale -1:3,600
200 100 0 200 400 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
19-Nov-2009

Figure 6.2


1645122
1645622
1646122
1646622
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1647622
84480
83980
83480
82980
Legend
Water Surface Elevation (ft - HI - local)
23.0
24.0
25.0
26.0
27.0
28.0
29.0
30.0
31.0
32.0
33.0
34.0
35.0
36.0
37.0
38.0
39.0
40.0
41.0
42.0
43.0
Oahu Reference Map
Honolulu High Capacity Transit Corridor
FESWMS Water Surface Elevation Contours for 100-year Event - Existing Conditions
Scale -1:3,600
200 100 0 200 400 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
19-Nov-2009

Figure 6.3

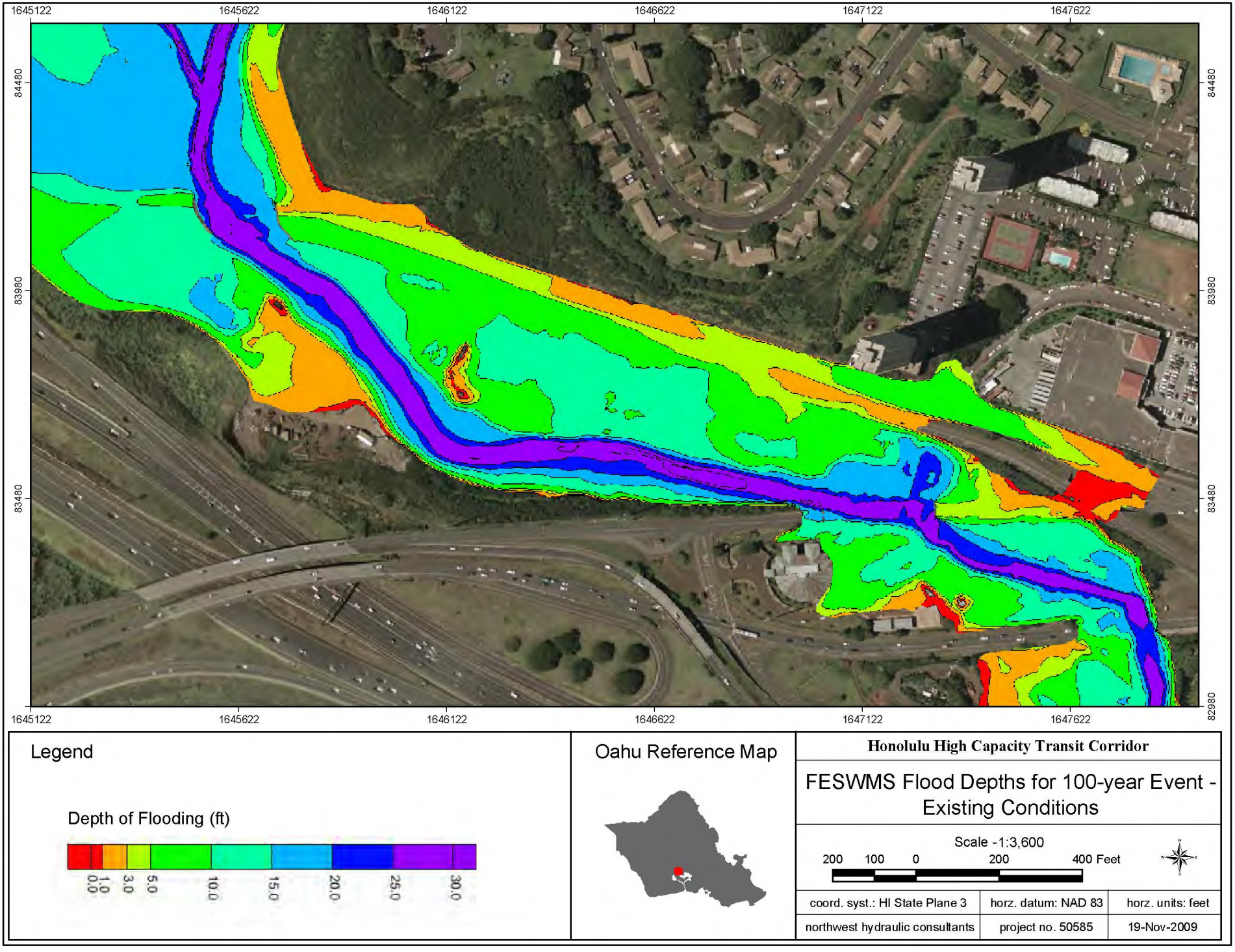
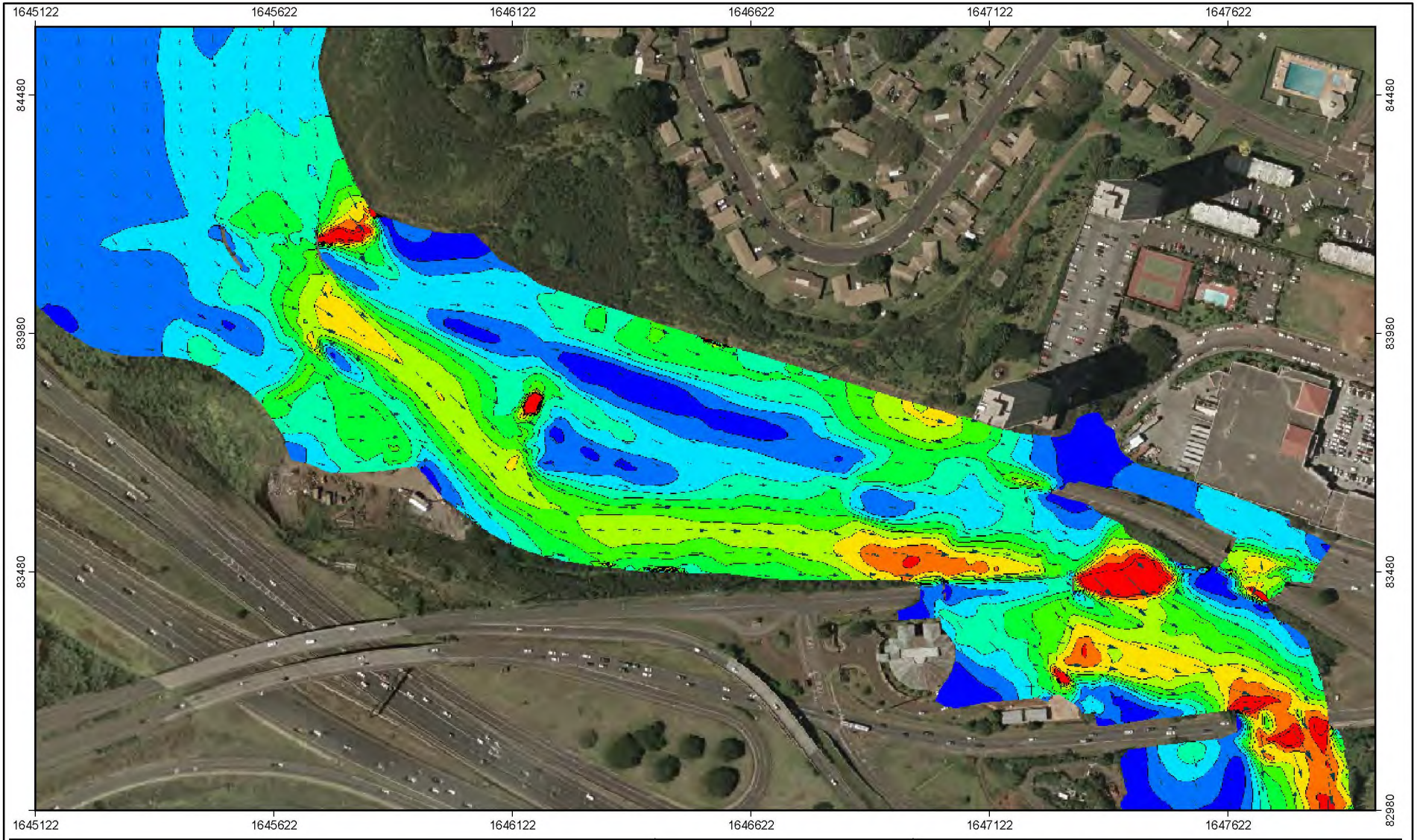

1645122
1645622
1646122
1646622
1647122
1647622
84480
83980
83480
82980
Legend
Depth of Flooding (ft)
0.0
1.0
3.0
5.0
10.0
15.0
20.0
25.0
30.0
Oahu Reference Map
Honolulu High Capacity Transit Corridor
FESWMS Flood Depths for 100-year Event - Existing Conditions
Scale -1:3,600
200 100 0 200 400 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
19-Nov-2009

Figure 6.4



Legend

Direction and Magnitude of Velocity

Velocity (ft/s)

0.0 1.5 3.0 4.5 6.0 7.5 9.0 10.5 12.0 13.5 15.0

Oahu Reference Map

Honolulu High Capacity Transit Corridor

FESWMS Velocities for 100-year Event - Existing Conditions

Scale -1:3,600

200 100 0 200 400 Feet

coord. syst.: HI State Plane 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50585	19-Nov-2009

Figure 6.5

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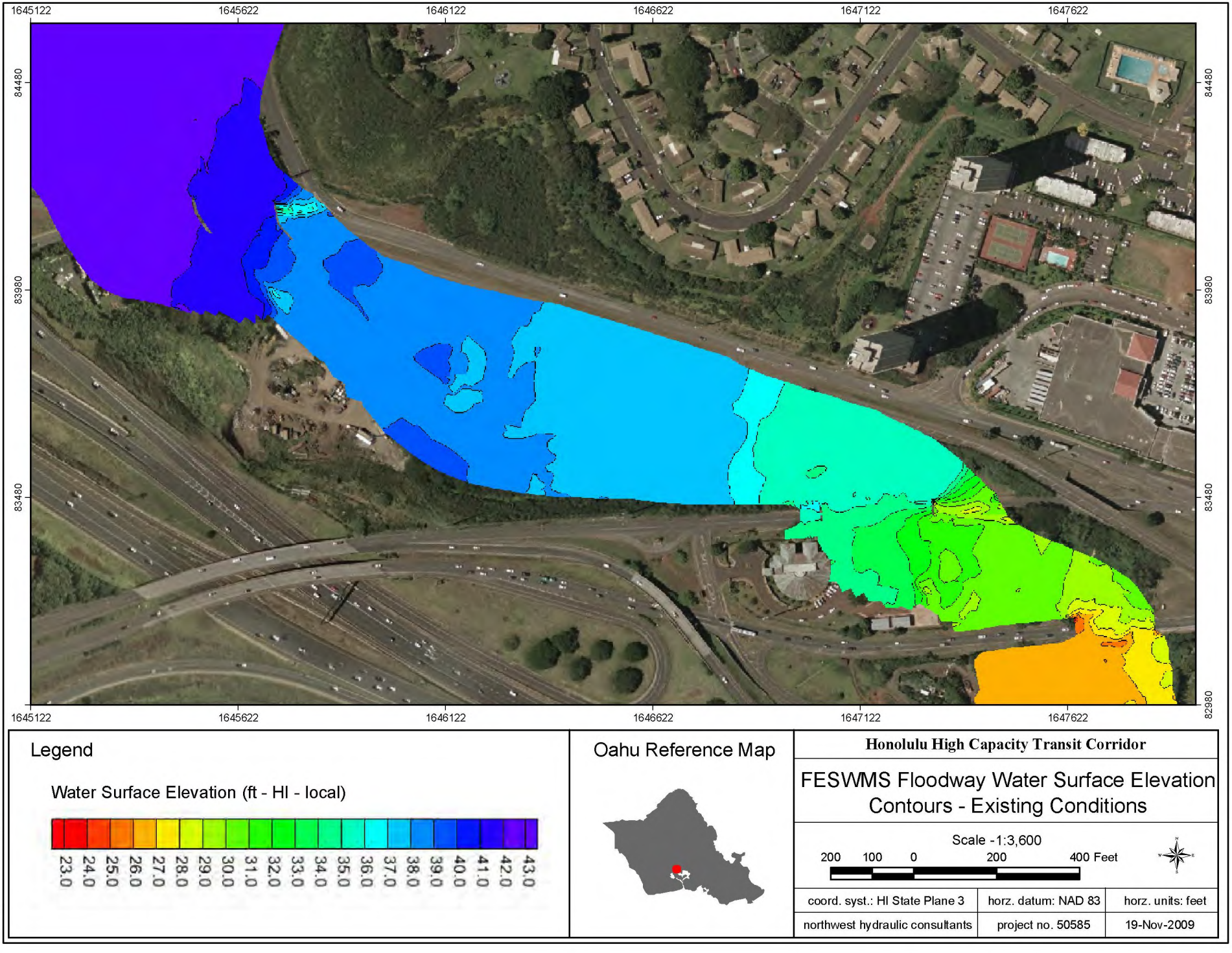
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83980
83480
82980
Legend
Water Surface Elevation (ft - HI - local)
23.0
24.0
25.0
26.0
27.0
28.0
29.0
30.0
31.0
32.0
33.0
34.0
35.0
36.0
37.0
38.0
39.0
40.0
41.0
42.0
43.0
Oahu Reference Map
Honolulu High Capacity Transit Corridor
FESWMS Floodway Water Surface Elevation Contours - Existing Conditions
Scale -1:3,600
200 100 0 200 400 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
19-Nov-2009

Figure 6.6

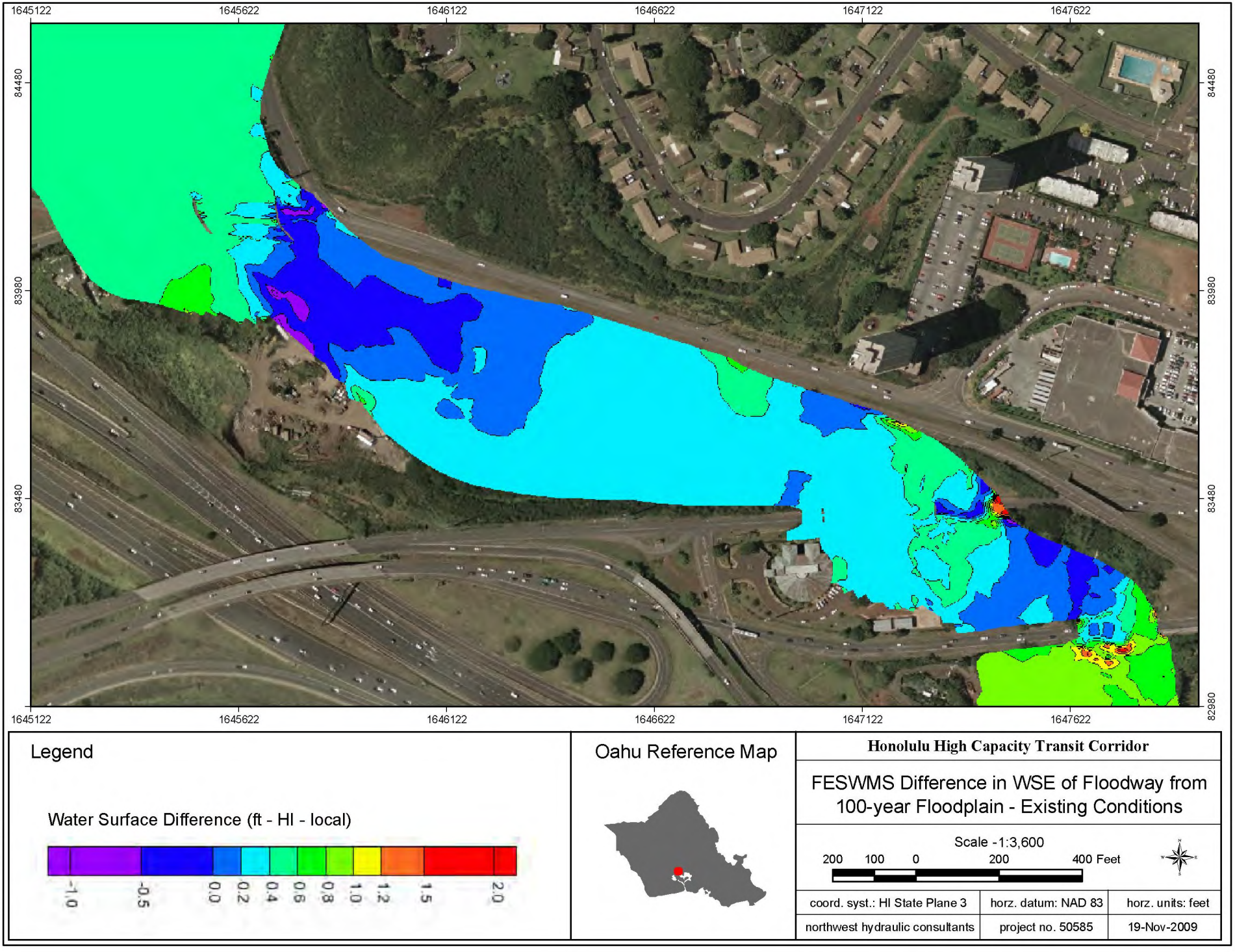


Figure 6.7

AR00041296

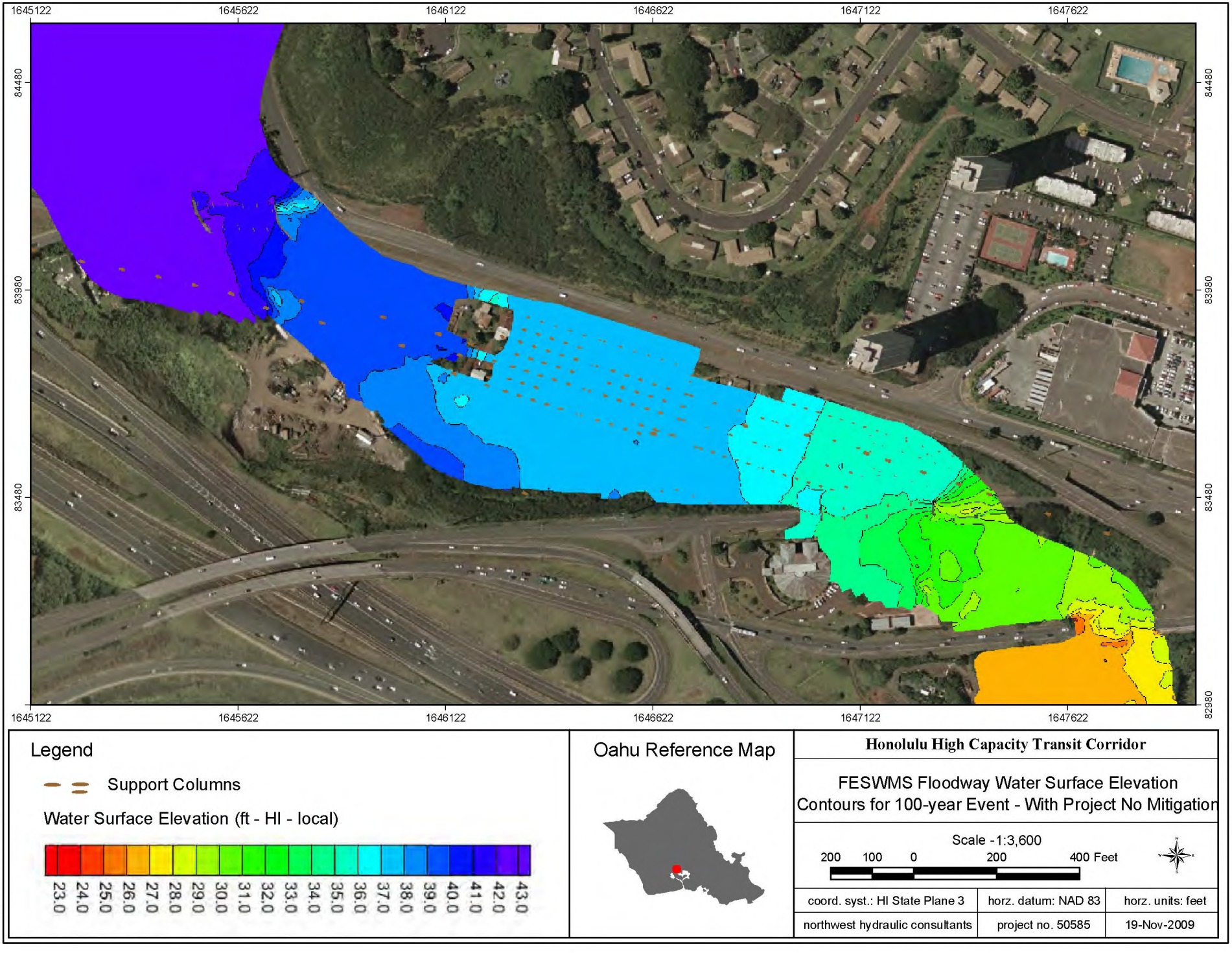
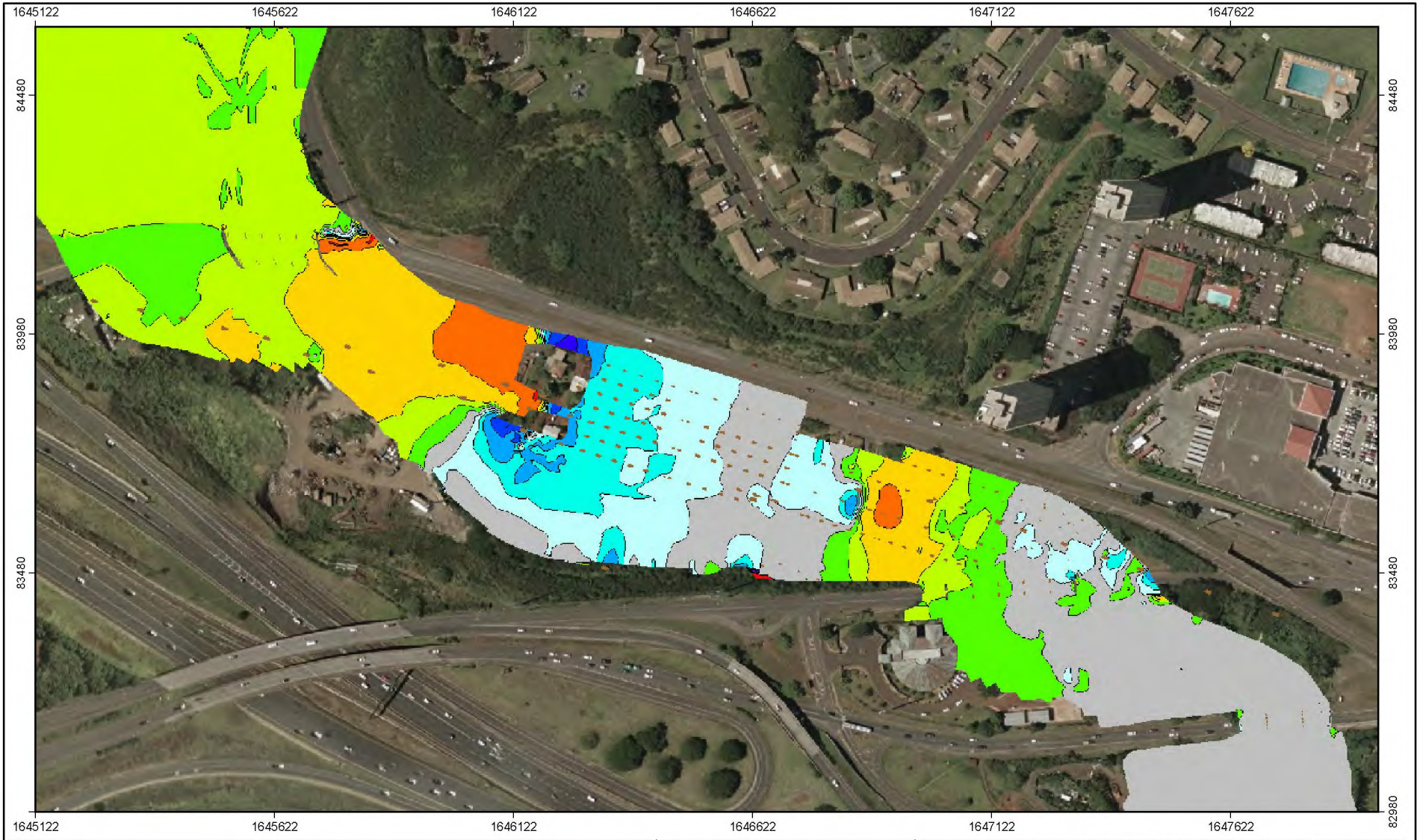

1645122
1645622
1646122
1646622
1647122
1647622
84480
83980
83480
82980
Legend
Support Columns
Water Surface Elevation (ft - HI - local)
23.0
24.0
25.0
26.0
27.0
28.0
29.0
30.0
31.0
32.0
33.0
34.0
35.0
36.0
37.0
38.0
39.0
40.0
41.0
42.0
43.0
Oahu Reference Map
Honolulu High Capacity Transit Corridor
FESWMS Floodway Water Surface Elevation
Contours for 100-year Event - With Project No Mitigation
Scale -1:3,600
200 100 0 200 400 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
19-Nov-2009

Figure 6.8



Legend

Support Columns

Water Surface Difference (ft - HI - local)

-5.0 -3.0 -2.0 -1.0 -0.5 -0.3 -0.1 0.1 0.3 0.5 1.0 2.0 3.0

Oahu Reference Map

Honolulu High Capacity Transit Corridor

FESWMS Difference in Floodway WSE - Project no Mitigation vs. Existing Conditions

Scale - 1:3,600

200 100 0 200 400 Feet

coord. syst.: HI State Plane 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50585	19-Nov-2009

Figure 6.9

AR00041298

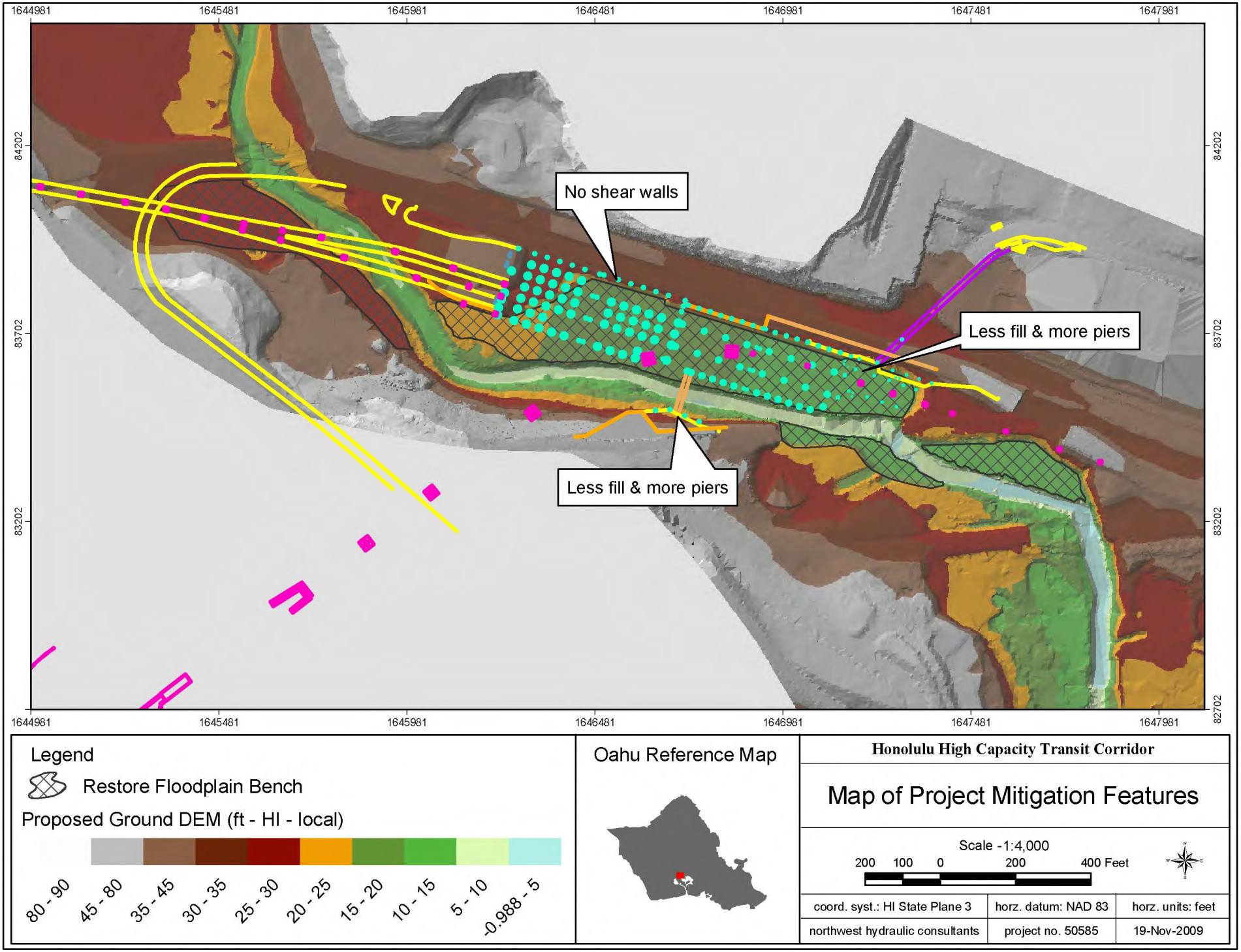

1644981
1645481
1645981
1646481
1646981
1647481
1647981
84202
83702
83202
82702
No shear walls
Less fill & more piers
Less fill & more piers
Legend
Restore Floodplain Bench
Proposed Ground DEM (ft - HI - local)
80 - 90
45 - 80
35 - 45
30 - 35
25 - 30
20 - 25
15 - 20
10 - 15
5 - 10
-0.988 - 5
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Map of Project Mitigation Features
Scale - 1:4,000
200 100 0 200 400 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
19-Nov-2009

Figure 6.10

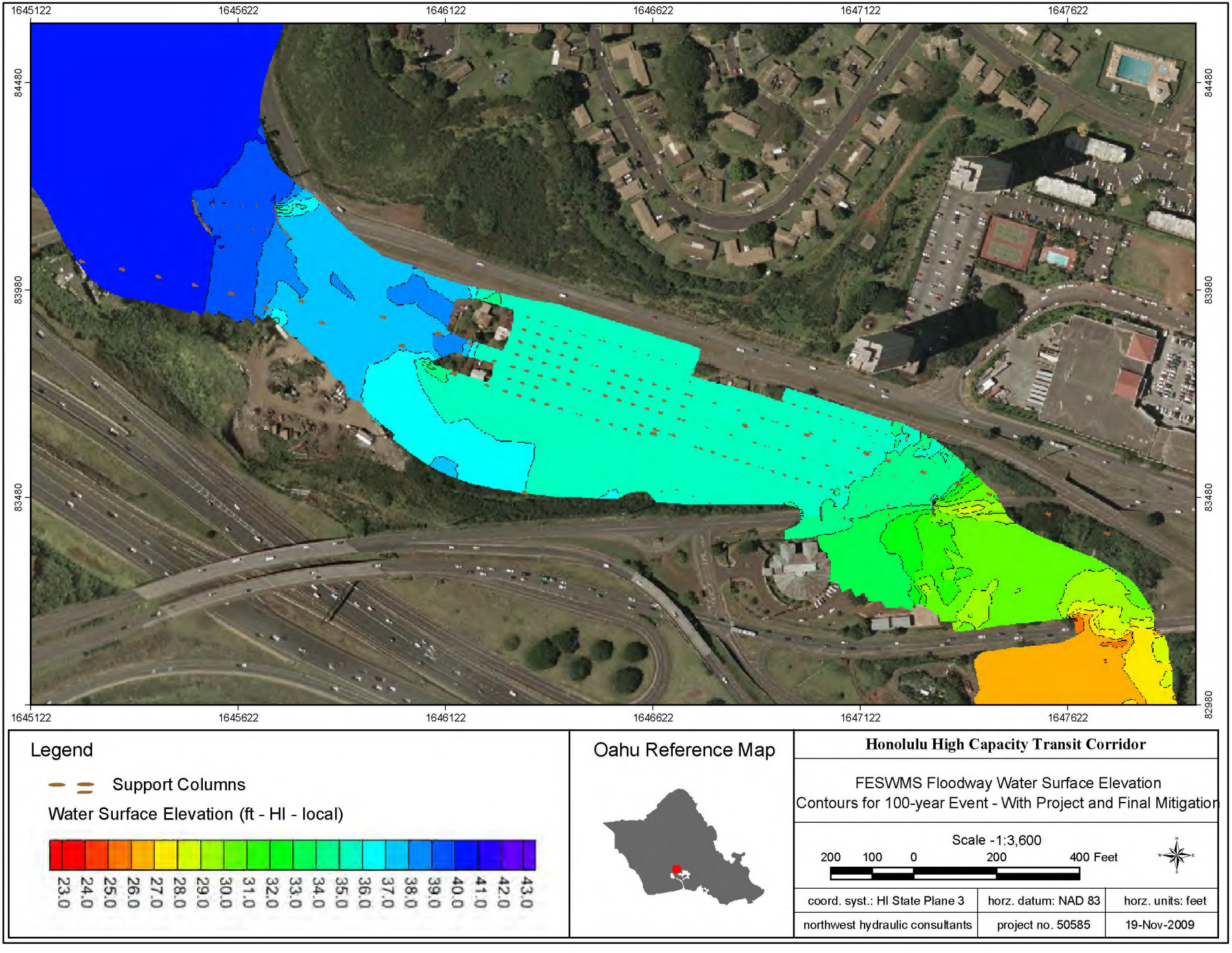
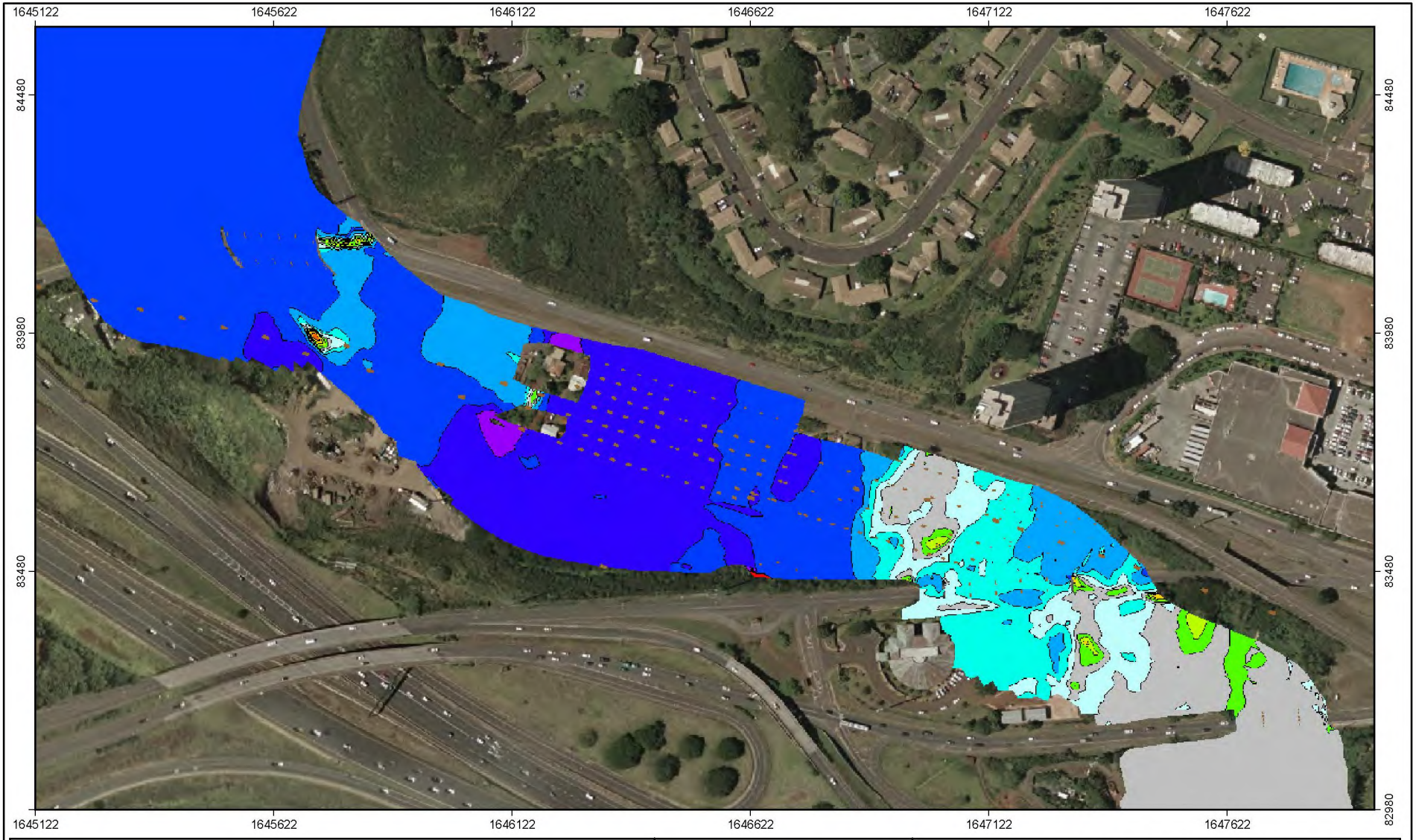


Figure 6.11

AR00041300



Legend

Support Columns

Water Surface Difference (ft - HI - local)

-5.0 -3.0 -2.0 -1.0 -0.5 -0.3 -0.1 0.1 0.3 0.5 1.0 2.0 3.0

Oahu Reference Map

Honolulu High Capacity Transit Corridor

FESWMS Difference in Floodway WSE -
Final Project with Mitigation vs. Existing Conditions

Scale - 1:3,600

200 100 0 200 400 Feet

coord. syst.: HI State Plane 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50585	19-Nov-2009

Figure 6.12

AR00041301

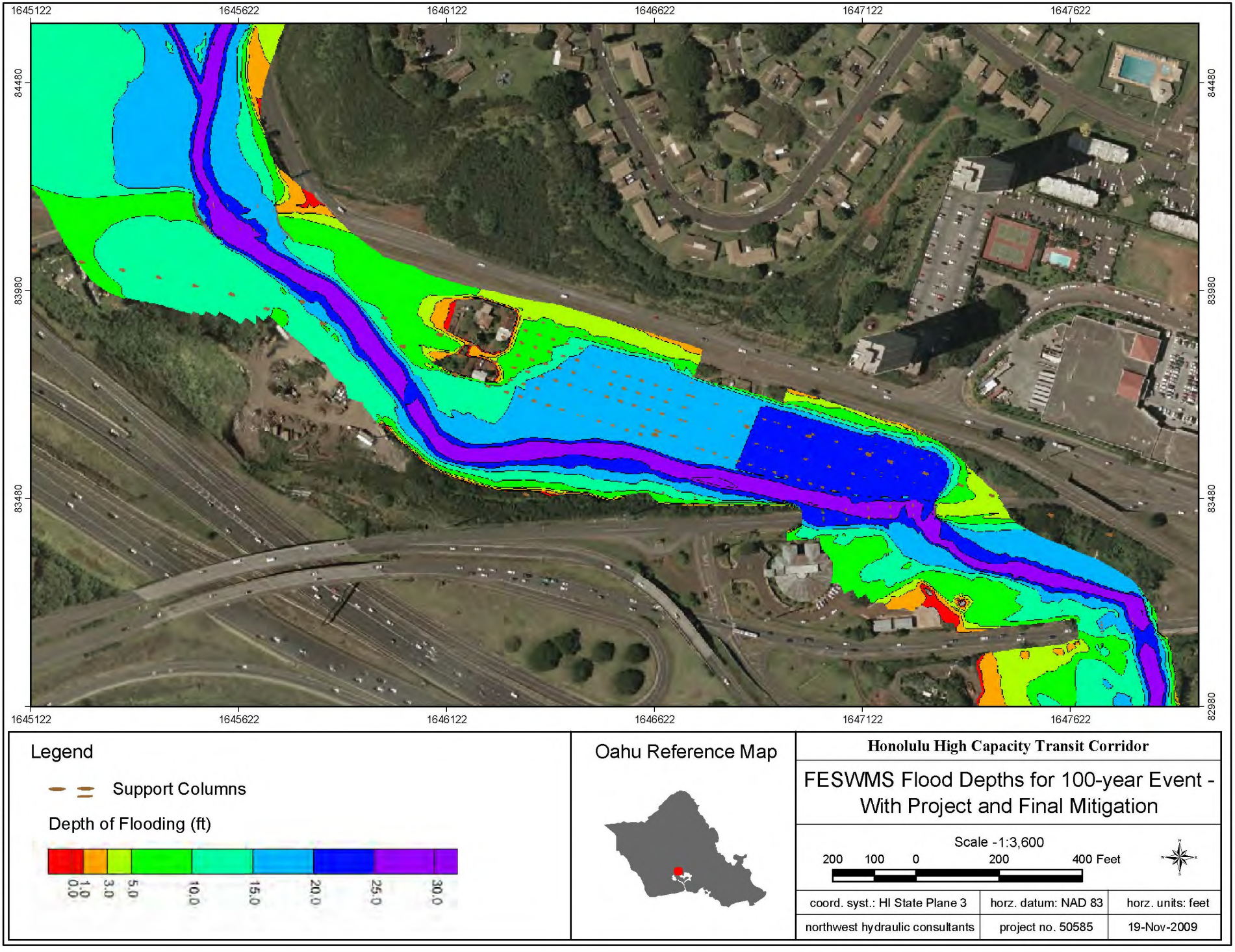
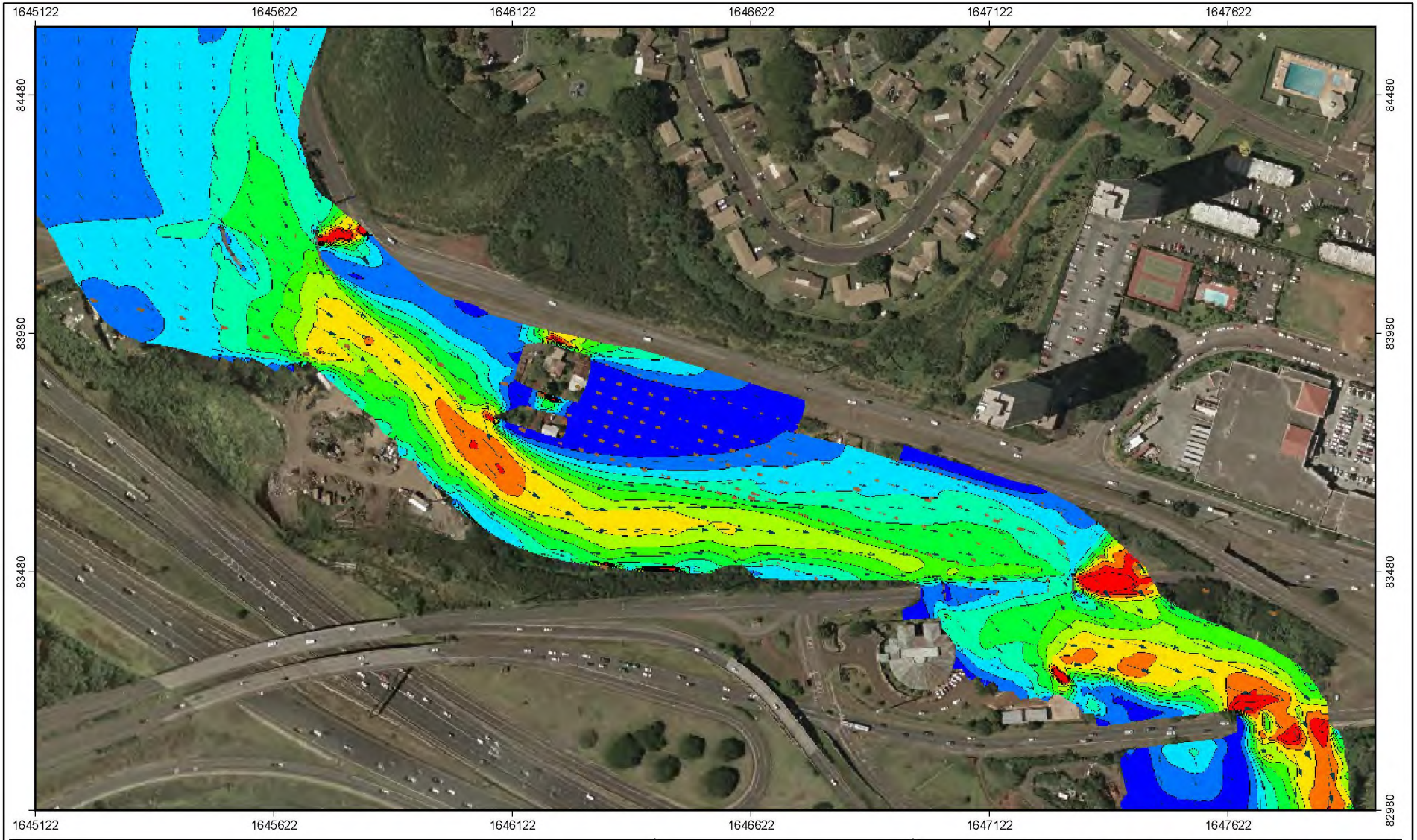


Figure 6.13

AR00041302



Legend

Support Columns

Direction and Magnitude of Velocity

Velocity (ft/s)

0.0 1.5 3.0 4.5 6.0 7.5 9.0 10.5 12.0 13.5 15.0

Oahu Reference Map

Honolulu High Capacity Transit Corridor

FESWMS Velocities for 100-year Event - With Project and Final Mitigation

Scale -1:3,600

200 100 0 200 400 Feet

coord. syst.: HI State Plane 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50585	19-Nov-2009

Figure 6.14

AR00041303

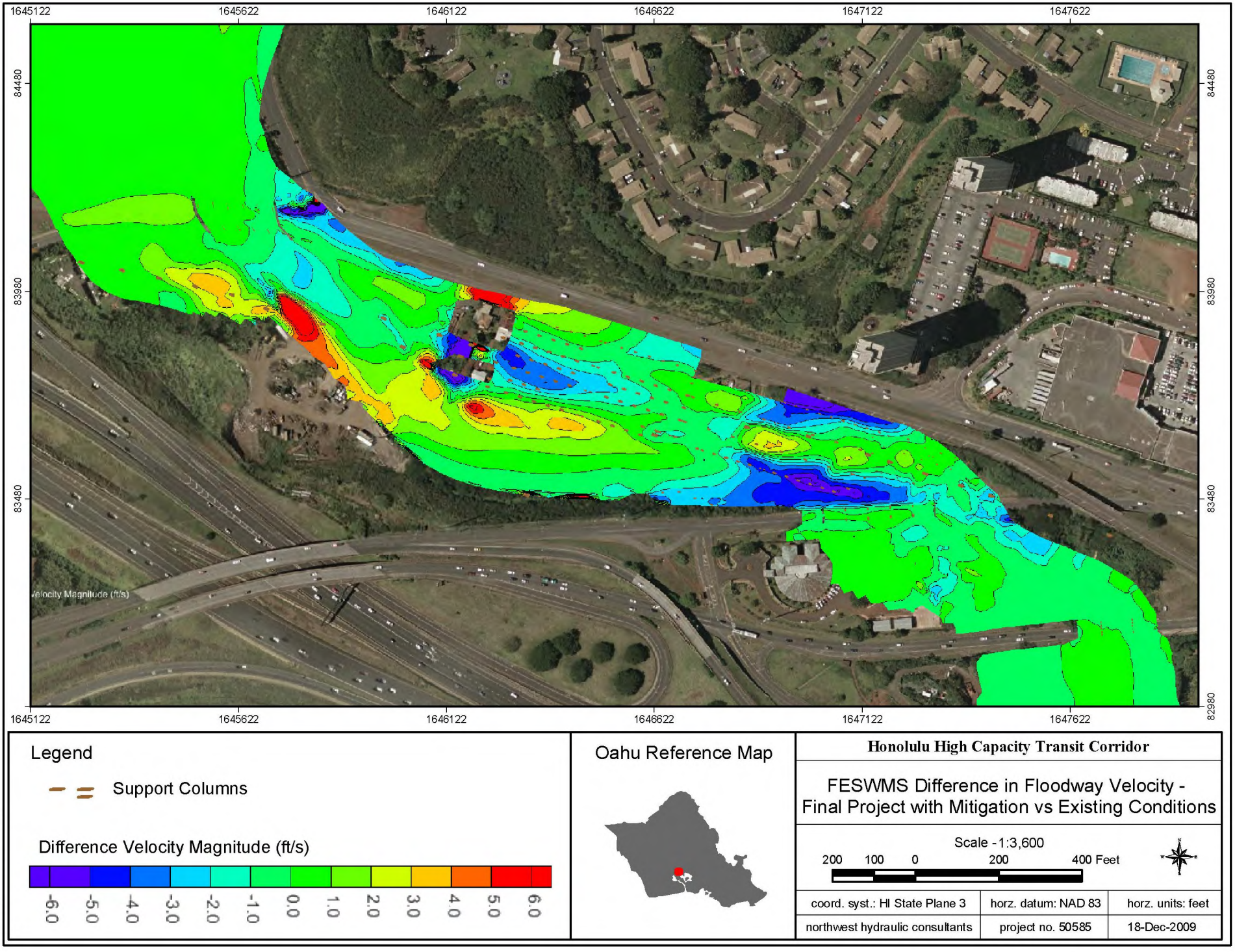

1645122
1645622
1646122
1646622
1647122
1647622
84480
83980
83480
82980
Velocity Magnitude (ft/s)
Legend
Support Columns
Difference Velocity Magnitude (ft/s)
-6.0
-5.0
-4.0
-3.0
-2.0
-1.0
0.0
1.0
2.0
3.0
4.0
5.0
6.0
Oahu Reference Map
Honolulu High Capacity Transit Corridor
FESWMS Difference in Floodway Velocity - Final Project with Mitigation vs Existing Conditions
Scale - 1:3,600
200 100 0 200 400 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
18-Dec-2009

Figure 6.15

7. "NO-RISE" CERTIFICATION AND DOCUMENTATION

City and County of Honolulu flood hazard district regulation 21-9.10.5(b) states that construction of features within a FEMA designated floodway are:

"…prohibited unless certification and supporting data including hydrologic and hydraulic analyses performed in accordance with standard engineering practice are provided by a licensed engineer demonstrating that the proposed encroachment will not cause any increase in regulatory flood elevations during the occurrence of the regulatory flood."

To demonstrate compliance with this condition, the City and County requires the project proponent to fill out a document called "Certification of a "No-Rise" Determination for a Proposed Floodway Development", a form that was created by FEMA (Appendix C). Also appended to this is a document titled "Certification Requirement for Simple Floodway Encroachments", also prepared by FEMA (Appendix D). It spells out the data that need to be prepared and submitted to demonstrate compliance with the "No-Rise" condition. For the PHSPF project the following hydraulic data are required:

1. Hydraulic backwater model of the 100-year flood and floodway water-surface profiles for the following:
    a. Duplicate of the effective Flood Insurance Study (FIS) model.
    b. Existing-conditions (effective FIS) model modified to include cross sections through the project site. Cross sections must reflect condition prior to construction of the project.
    c. Post-project conditions model. This model must include cross section through the project site reflecting floodplain conditions after construction for the project.
2. A copy of the appropriate NFIP map showing the existing floodway and indicating the project area.
3. Topographic mapping of the entire project area indicating the locations of all cross sections used in the modified hydraulic model and a plan view of all project elements.
4. Construction plans, certified by a registered professional engineer, for all project elements, including those measures employed to provide additional effective conveyance.

Data demonstrating that the proposed PHSPF project complies with the "No-Rise" requirement are presented below.

7.1 No-Rise Certification

A signed and stamped copy of the FEMA form "Certification of a "No-Rise" Determination for a Proposed Floodway Development" is presented in Appendix C.

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7.2 Hydraulic Backwater Model

As stated in Chapter 5, the FEMA floodplain maps for the Waiawa stream were created using HEC-2, a simple one-dimensional steady state water surface profile computer model. The first step in developing the data required to demonstrate No-rise is to convert the HEC-2 model to HEC-RAS to create a "duplicate effective model". This new model must then be modified to create both an "existing conditions model" and a "post-project conditions model" which together are used to demonstrate No-rise. NHC has completed these steps as explained below.

7.2.1 Duplicate Effective Model

As described in Chapter 5, NHC obtained the original FEMA HEC-2 effective model from FEMA and converted it to HEC-RAS to create a "duplicate effective model".

7.2.2 Existing Conditions Model

As described in Chapter 5, NHC refined the "duplicate effective model" to contain sufficient detail to appropriately evaluate the impact of the proposed PHSPF. This model was referred to as the "refined model". The "refined model" needed additional modifications to make it appropriate for use as the "existing condition model". Modifications included a revised cross section layout, updated geometry and modeling of the four Kamehameha and Farrington Highway bridges, and further calibration of ineffective areas, roughness, and contraction and expansion losses. The updated model reach ties into the effective FIS at cross section J (7581) downstream and cross section M (11145) upstream. Tables 7.1 and 7.2 list the final calibrated values used for roughness, and parameters for contraction and expansion losses. The roughness values presented in Table 7.1 are the same as previously given in Table 5.2. However, the areas where these were applied on certain cross sections changed during the continued calibration undertaken during "existing condition model" development. The final cross section layout and model reach of the existing conditions HEC-RAS model are illustrated in Figure 7.1. The water surface profiles for the final calibration existing conditions HEC-RAS model are shown in Figure 7.2.

Table 7.1 Calibrated Manning's Roughness Values in HEC-RAS Model

Land Use	n Value
Road or Paved	0.015
Channel	0.025
Shade	0.035
Mostly Open	0.040
Light Vegetation	0.060
Heavy Vegetation	0.070
Dense Vegetation	0.080

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Table 7.2 Calibrated Contraction and Expansion Coefficients in HEC-RAS Model					
HEC-RAS Cross Section	Contraction	Expansion	HEC-RAS Cross Section	Contraction	Expansion
7581	0.10	0.30	9419	0.10	0.30
8400	0.10	0.30	9471	0.10	0.30
8450	0.10	0.30	9558	0.10	0.30
8500	0.10	0.30	9626	0.10	0.30
8550	0.10	0.30	9682	0.10	0.30
8600	0.10	0.30	9780	0.10	0.30
8650	0.10	0.30	9924	0.10	0.30
8690	0.10	0.30	10028	0.10	0.30
8720	0.30	0.50	10148	0.10	0.30
8750	0.30	0.50	10242	0.10	0.30
8780	0.30	0.50	10328	0.10	0.30
8810	0.30	0.50	10404	0.10	0.30
8840	0.30	0.50	10500	0.10	0.30
8870	0.30	0.50	10600	0.10	0.30
8900	0.15	0.40	10720	0.15	0.50
8950	0.15	0.40	10740	0.15	0.50
8980	0.15	0.40	10760	0.15	0.50
9000	0.15	0.40	10780	0.15	0.50
9011	0.15	0.40	10850	0.10	0.30
9050	0.15	0.40	10900	0.10	0.30
9090	0.15	0.40	10950	0.10	0.30
9130	0.15	0.40	11045	0.10	0.30
9171	0.15	0.40	11145	0.10	0.30
9253	0.15	0.40	11785	0.10	0.30
9338	0.15	0.40	11995	0.10	0.30

7.2.3 Post-Project Conditions Model

Once NHC and the PB team settled on a final design for the PHSPF as described in Chapter 6, NHC modified the "existing condition model" to include the project with mitigation (see Figure 6.10), and thus created a "post-project conditions model". Cross section locations were not altered from the "existing conditions model" (Figure 7.1).

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7.2.4 Flood Profiles and No-Rise Confirmation

Water surface profiles were computed using the existing conditions model for the 100-year floodplain and floodway. These profiles are compared in Figure 7.3, and the actual computed water surface elevations at each cross section are compared in Table 7.3.

Table 7.3 HEC-RAS Water Surface Elevation Comparison for Existing Condition Simulations

HEC-RAS Cross Section	100-year WSE	Floodway WSE	WS Increase Caused by Floodway		HEC-RAS Cross Section	100-year WSE	Floodway WSE	WS Increase Caused by Floodway
7581	26.920	27.500	0.580		9419	36.635	37.183	0.548
8400	27.327	27.693	0.366		9471	36.474	37.061	0.587
8450	27.718	28.427	0.709		9558	36.813	37.370	0.557
8500	28.631	29.261	0.630		9626	37.065	37.592	0.527
8550	28.051	28.754	0.703		9682	37.040	37.571	0.531
8600	26.417	27.414	0.997		9780	36.949	37.465	0.516
8650	28.671	29.570	0.899		9924	37.390	37.819	0.429
8690	29.312	29.889	0.577		10028	37.029	37.364	0.335
8720	29.820	30.043	0.223		10148	36.893	37.166	0.273
8750	29.832	30.180	0.348		10242	36.664	36.923	0.259
8780	29.405	29.840	0.435		10328	36.659	36.993	0.334
8810	31.942	32.304	0.362		10404	36.751	37.218	0.467
8840	31.898	32.276	0.378		10500	38.080	38.107	0.027
8870	31.916	32.395	0.479		10600	38.343	38.934	0.591
8900	32.657	33.101	0.444		10720	38.940	39.517	0.577
8950	35.432	36.093	0.661		10740	39.108	39.672	0.564
8980	35.120	35.811	0.691		10760	39.156	39.710	0.554
9000	35.533	36.179	0.646		10780	39.363	39.981	0.618
9011	35.307	35.888	0.581		10850	39.063	39.729	0.666
9050	35.128	35.709	0.581		10900	39.036	39.660	0.624
9090	34.537	35.147	0.610		10950	39.187	39.782	0.595
9130	34.492	35.168	0.676		11045	39.286	39.819	0.533
9171	34.515	35.221	0.706		11145	38.220	38.955	0.735
9253	35.151	35.828	0.677		11785	40.607	41.119	0.512
9338	36.453	36.993	0.540		11995	40.333	40.874	0.541

The existing condition versus post-project 100-year floodplain (without floodway) and floodway water surface profiles are compared in Figures 7.4 and 7.5, respectively. The actual computed water surface elevations at each cross section are compared in Tables 7.4 and 7.5. The figures

AR00041308

and tables demonstrate that in no place does the post-project profile extend above the existing condition profile, both for the 100-year (without floodway) and for the with floodway case; demonstrating that the project meets the "No-rise" requirement.

Table 7.4 HEC-RAS Water Surface Elevation Comparison of 100-year Floodplain (without Floodway) Simulations

HEC-RAS Cross Section	Existing Conditions	Proposed Conditions	WS Increase Caused by Proposed Conditions		HEC-RAS Cross Section	Existing Conditions	Proposed Conditions	WS Increase Caused by Proposed Conditions
7581	26.920	26.920	0.000		9419	36.635	34.199	-2.436
8400	27.327	27.327	0.000		9471	36.474	34.604	-1.870
8450	27.718	27.718	0.000		9558	36.813	34.813	-2.000
8500	28.631	28.631	0.000		9626	37.065	34.713	-2.352
8550	28.051	28.051	0.000		9682	37.040	34.671	-2.369
8600	26.417	26.417	0.000		9780	36.949	34.443	-2.506
8650	28.671	28.671	0.000		9924	37.390	34.125	-3.265
8690	29.312	29.287	-0.025		10028	37.029	34.653	-2.376
8720	29.820	29.439	-0.381		10148	36.893	33.212	-3.681
8750	29.832	29.522	-0.310		10242	36.664	35.074	-1.590
8780	29.405	29.357	-0.048		10328	36.659	35.009	-1.650
8810	31.942	30.762	-1.180		10404	36.751	35.000	-1.751
8840	31.898	30.609	-1.289		10500	38.080	37.419	-0.661
8870	31.916	30.857	-1.059		10600	38.343	37.256	-1.087
8900	32.657	31.979	-0.678		10720	38.940	38.310	-0.630
8950	35.432	33.816	-1.616		10740	39.108	38.468	-0.640
8980	35.120	33.550	-1.570		10760	39.156	38.506	-0.650
9000	35.533	33.712	-1.821		10780	39.363	38.775	-0.588
9011	35.307	33.256	-2.051		10850	39.063	38.415	-0.648
9050	35.128	33.635	-1.493		10900	39.036	38.417	-0.619
9090	34.537	34.181	-0.356		10950	39.187	38.621	-0.566
9130	34.492	34.328	-0.164		11045	39.286	38.738	-0.548
9171	34.515	34.224	-0.291		11145	38.220	37.242	-0.978
9253	35.151	34.155	-0.996		11785	40.607	40.425	-0.182
9338	36.453	34.185	-2.268		11995	40.333	40.129	-0.204

Table 7.5 HEC-RAS Water Surface Elevation Comparison of Floodway Simulations							
HEC-RAS Cross Section	Existing Conditions	Proposed Conditions	WS Increase Caused by Proposed Conditions	HEC-RAS Cross Section	Existing Conditions	Proposed Conditions	WS Increase Caused by Proposed Conditions
7581	27.500	27.500	0.000	9419	37.183	35.003	-2.180
8400	27.693	27.693	0.000	9471	37.061	35.368	-1.693
8450	28.427	28.427	0.000	9558	37.370	35.559	-1.811
8500	29.261	29.261	0.000	9626	37.592	35.472	-2.120
8550	28.754	28.754	0.000	9682	37.571	35.433	-2.138
8600	27.414	27.414	0.000	9780	37.465	35.245	-2.220
8650	29.570	29.570	0.000	9924	37.819	34.941	-2.878
8690	29.889	29.842	-0.047	10028	37.364	35.285	-2.079
8720	30.043	29.742	-0.301	10148	37.166	34.069	-3.097
8750	30.180	29.755	-0.425	10242	36.923	35.384	-1.539
8780	29.840	29.689	-0.151	10328	36.993	35.265	-1.728
8810	32.304	31.540	-0.764	10404	37.218	35.578	-1.640
8840	32.276	31.432	-0.844	10500	38.107	37.268	-0.839
8870	32.395	31.797	-0.598	10600	38.934	37.465	-1.469
8900	33.101	32.792	-0.309	10720	39.517	38.520	-0.997
8950	36.093	34.669	-1.424	10740	39.672	38.672	-1.000
8980	35.811	34.423	-1.388	10760	39.710	38.702	-1.008
9000	36.179	34.566	-1.613	10780	39.981	39.083	-0.898
9011	35.888	34.149	-1.739	10850	39.729	38.764	-0.965
9050	35.709	34.376	-1.333	10900	39.660	38.718	-0.942
9090	35.147	34.922	-0.225	10950	39.782	38.901	-0.881
9130	35.168	35.087	-0.081	11045	39.819	38.937	-0.882
9171	35.221	34.994	-0.227	11145	38.955	37.625	-1.330
9253	35.828	34.955	-0.873	11785	41.119	40.680	-0.439
9338	36.993	34.999	-1.994	11995	40.874	40.393	-0.481

7.3 NFIP Existing Floodway Map

A copy of the existing FEMA DFIRM Floodway Map for the project area is attached as Figure 7.6.

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7.4 Existing Topographic Map with Cross Section Locations

A current topographic map of the project site is included with the HEC-RAS cross section locations shown on Figure 7.1. The proposed project configuration is illustrated in Figure 6.10.

7.5 Construction Plans

Construction plans for the PHSPF will be submitted by PB as a separate document.

7.6 Compensatory Volume Requirement

When evaluating the impact a structure may have on flood levels there are two issues that need to be addressed – loss of the ability for the stream to convey flow downstream and loss of floodplain storage. In this project, conveyance would be reduced by the obstructions created by the columns and fills. However, the loss of conveyance will be mitigated by excavating and restoring the floodplain as illustrated in Figure 6.10. The HEC-RAS modeling described in Section 7.2 appropriately evaluated this conveyance issue and demonstrated that there will be no net loss of conveyance.

Loss of storage volume in the floodway is not addressed by the steady-state HEC-RAS modeling that was completed for this investigation. When obstructions are placed within a floodway one must demonstrate that the proposed project will not reduce existing storage volume. If storage is lost, flood levels downstream may rise. The easiest way to prevent downstream impacts is to remove sufficient fill to compensate for the obstruction. This has been carefully considered in the design of the PHSPF facility and the proposed excavation volume will significantly exceed the volume lost due to the columns and fill that will be placed when the facility is constructed. Proposed facility features will displace approximately 13200 cu yards of water within the floodway during a 100-year flood, while the excavation will remove approximately 79700 cu yards of existing material inside the floodway. This more than compensates for the storage volume that will be eliminated by the proposed facility features.

7.7 Phased Construction – Guideway Only

The No-Rise analysis described in this report is for the entire project, which includes the elevated guideway as well as the Station / Park-and-Ride Facility. The current plan is to construct the guideway portion of the project as part of the West Oahu / Farrington Design-Build (WOF D-B) Contract, prior to the Station and Park-and-Ride Facility. The No-Rise analysis for the WOF D-B Contract only is presented in a separate report titled "No-Rise Analysis for HHCTCP Guideway Columns – Waiawa Stream" (April 30, 2010).

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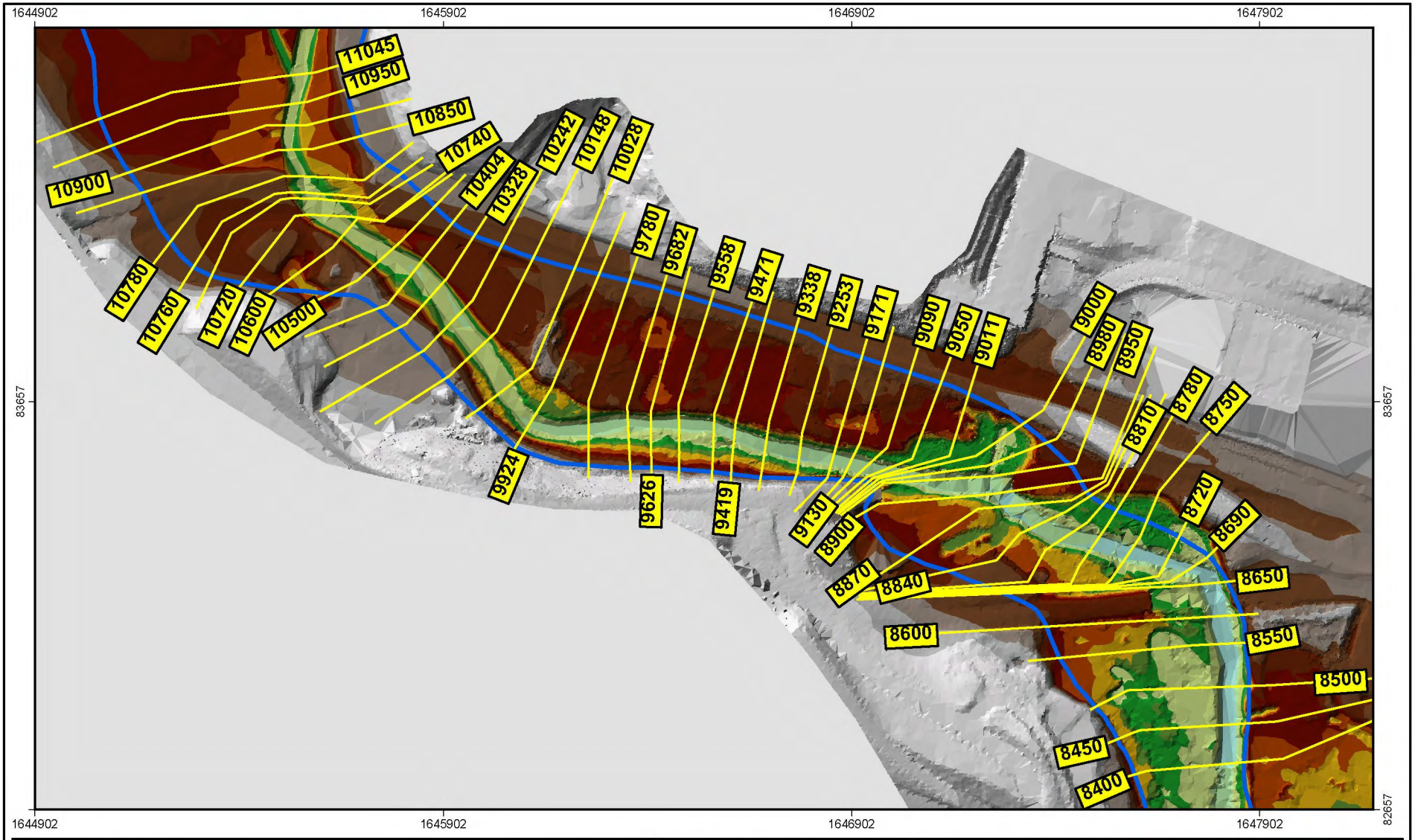
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10028
9780
9682
9558
9471
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9253
9171
9090
9050
9011
9000
8980
8950
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9626
9419
9130
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8600
8550
8500
8450
8400
Legend
Model Cross-Sections
FEMA Floodway
Ground DEM (ft - HI - local)
45 - 90
40 - 45
35 - 40
30 - 35
27 - 30
25 - 27
23 - 25
20 - 23
18 - 20
15 - 18
10 - 15
5 - 10
< 5
Oahu Reference Map
Honolulu High Capacity Transit Corridor
No-Rise HEC-RAS Model
Cross-Section Locations
Scale - 1:4,200
200 100 0 200 400 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
18-Dec-2009

Figure 7.1

Honolulu High Capacity Transit Corridor		
Existing Conditions HEC-RAS Model Water Surface Profiles		
Final Calibration		
northwest hydraulic consultants	project no. 50585	18-Dec-2009

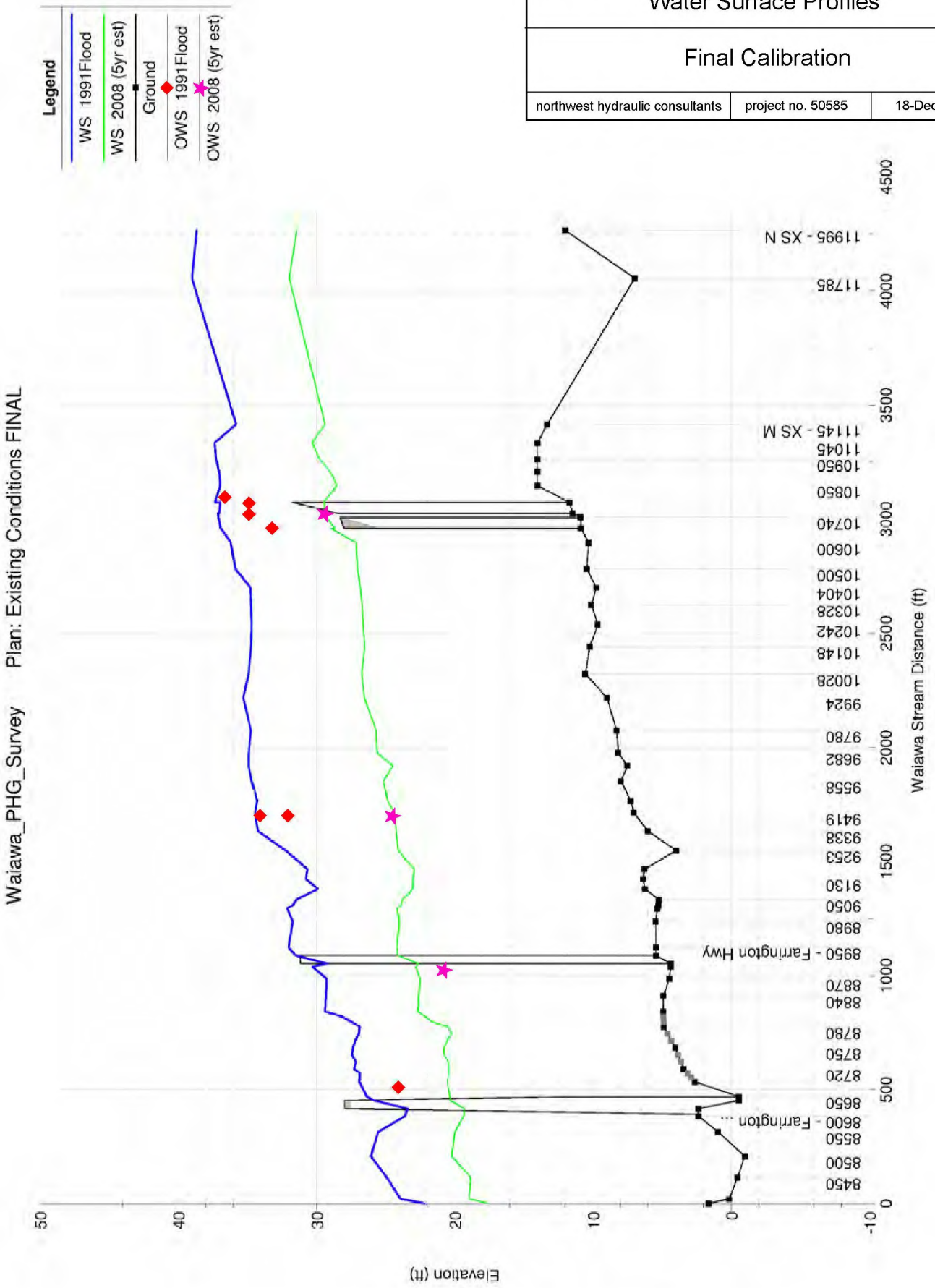
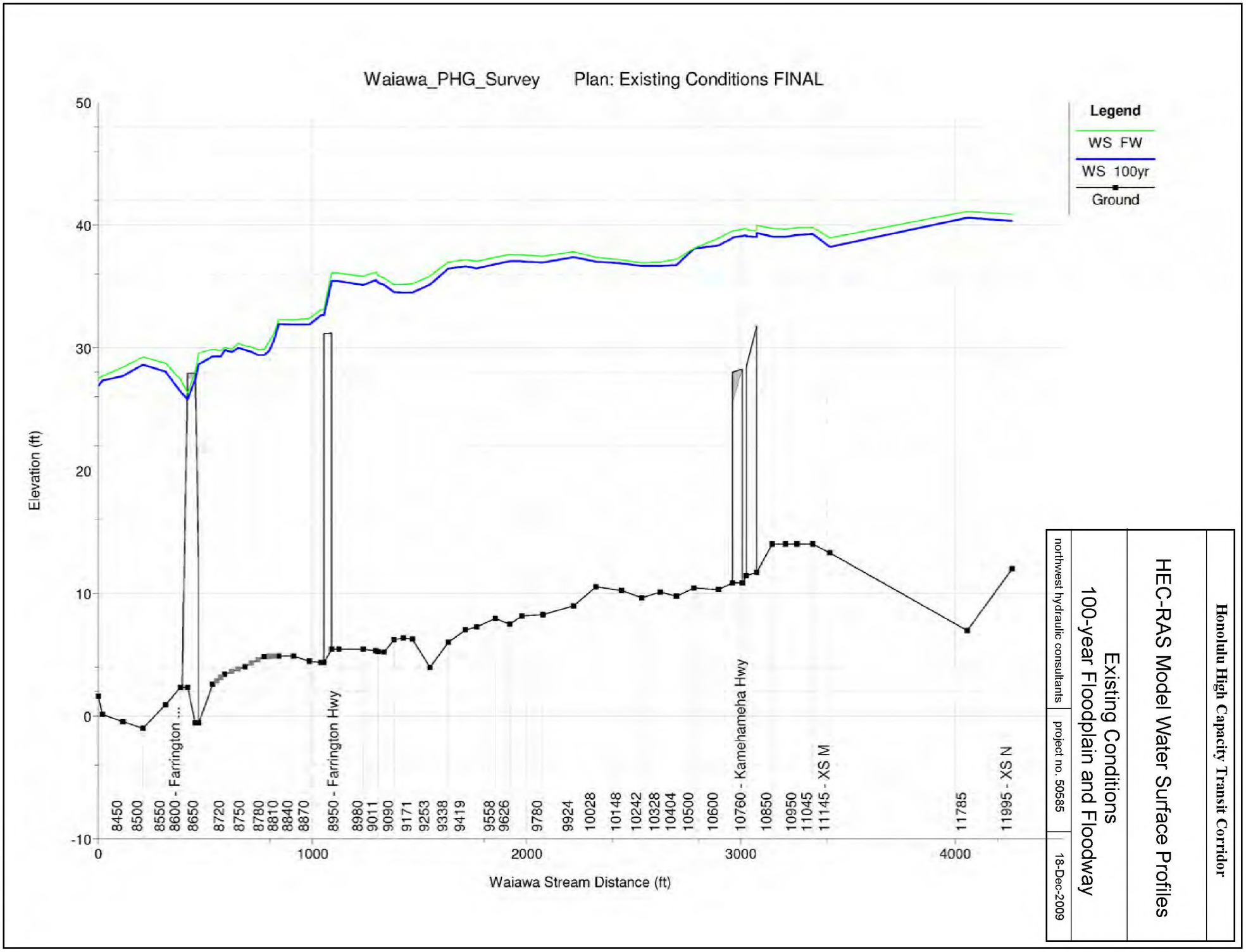


Figure 7.2



Honolulu High Capacity Transit Corridor

HEC-RAS Model Water Surface Profiles

Existing Conditions
100-year Floodplain and Floodway

northwest hydraulic consultants	project no. 50585	18-Dec-2009

Figure 7.3

AR00041314

Honolulu High Capacity Transit Corridor		
HEC-RAS Model Water Surface Profiles		
100-year Floodplain - Existing and Proposed Conditions		
northwest hydraulic consultants	project no. 50585	18-Dec-2009

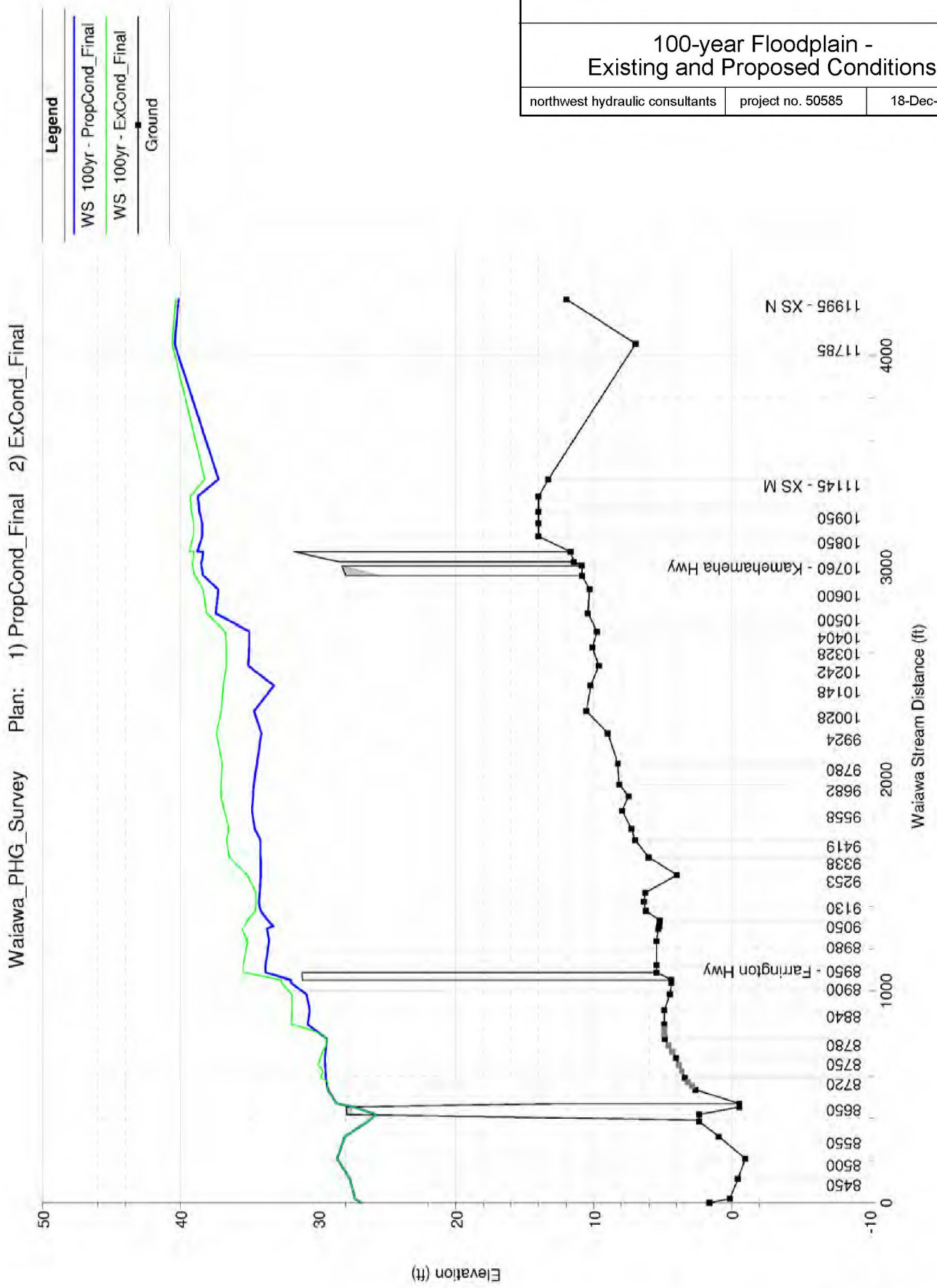


Figure 7.4

Honolulu High Capacity Transit Corridor		
HEC-RAS Model Water Surface Profiles		
Floodway Run - Existing and Proposed Conditions		
northwest hydraulic consultants	project no. 50585	18-Dec-2009

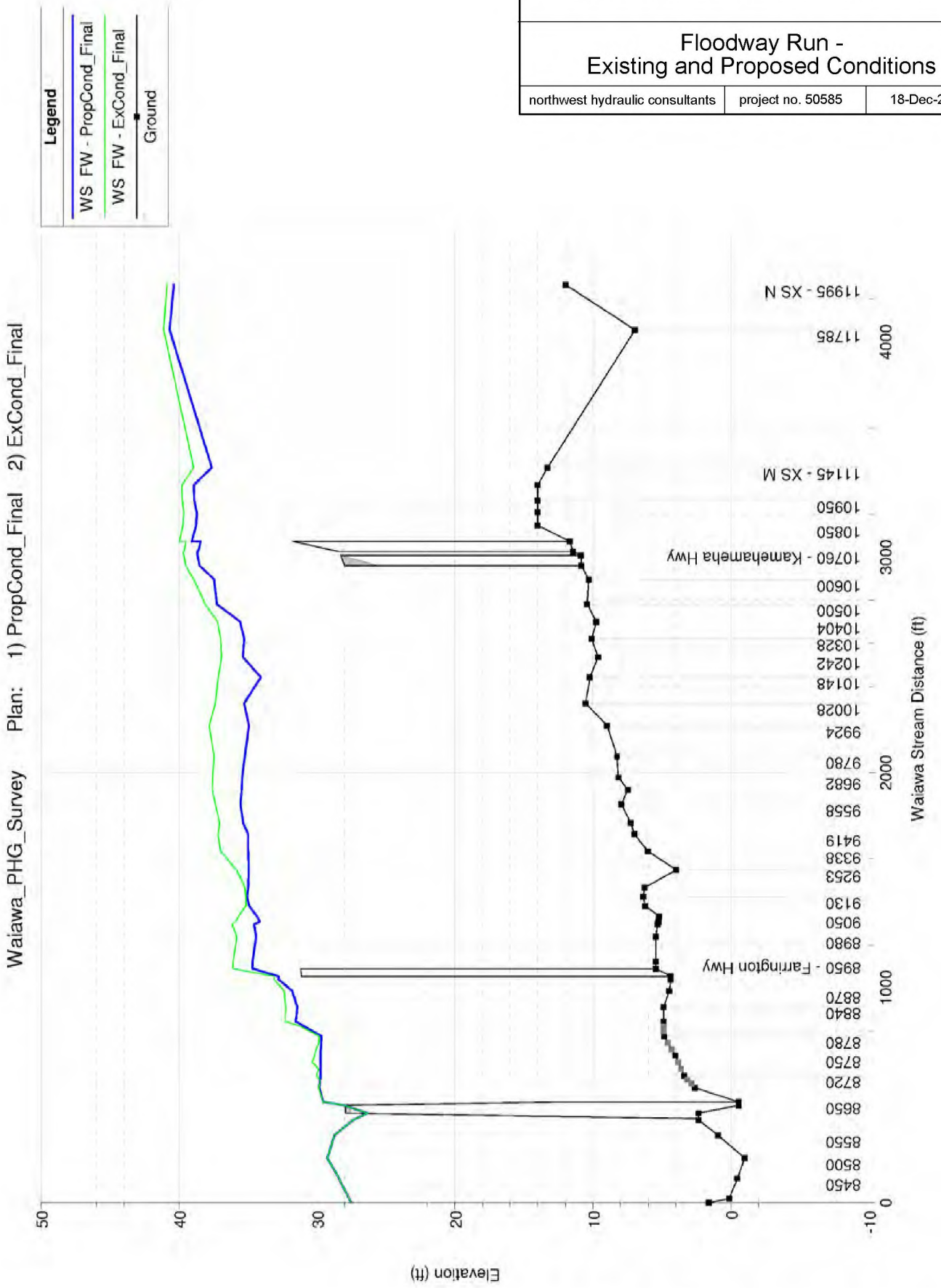


Figure 7.5

Honolulu High Capacity Transit Cooridor

Effective FEMA DFIRM Map

FIRM Panel 239 of 395 - Map # 15003C0239F
Map Revised September 30, 2004

Scale - 1:7,000

coord. syst.: HI State Plane 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50585	18-Dec-2009

Oahu Reference Map

LEGEND

SPECIAL FLOOD HAZARD AREAS SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD

The 1% annual chance flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V, and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.

ZONE A	No Base Flood Elevations determined.
ZONE AE	Base Flood Elevations determined.
ZONE AH	Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.
ZONE AO	Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
ZONE AR	Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
ZONE A99	Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.
ZONE V	Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.
ZONE VE	Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE

Figure 7.6

AR00041317

8. CONCLUSION

The proposed PHSPF is in part located within the floodway of the Waiawa Stream, a system that generates the largest peak discharges on the island of Oahu. NHC, working with the PHSPF design team, together have developed a design for the facility that not only successfully mitigates impacts the project may have on 100-year flood levels but also minimizes the risk imposed on the facility by the stream.

AR00041318

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AR00041319

APPENDIX A

Photos

AR00041320

Photo 1. Viewing downstream to the Kamehameha Highway bridges.

Photo 2. Debris on Kamehameha Highway bridge piers and deck.

Photo 3. Viewing the across the channel at the left bank fill in Banana Patch reach.

Photo 4. Viewing the across the channel at the left bank fill in Banana Patch reach.

Photo 5. Viewing vegetated "natural" area of left bank floodplain upstream of west-bound Farrington highway bridge; fill in background.

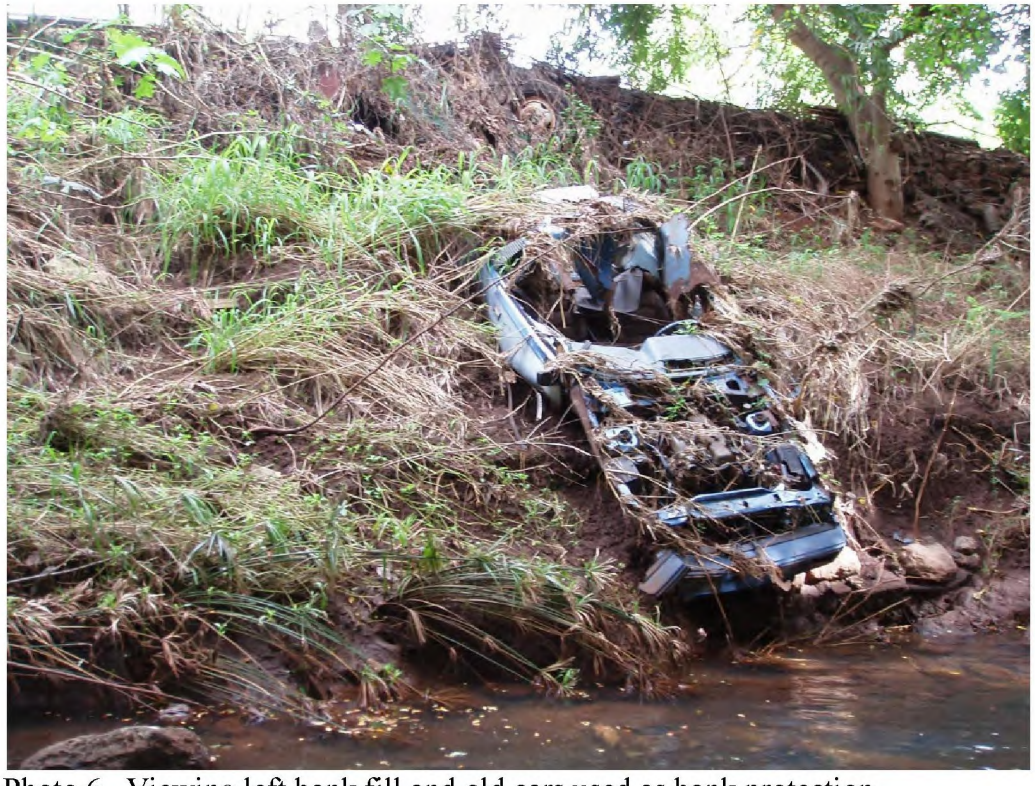

Photo 6. Viewing left bank fill and old cars used as bank protection.

AR00041323

Photo 7. Rocks have been placed along the banks in many locations in the project reach.

Photo 8. Viewing upstream at the Waiawa Stream channel in the Banana Patch reach.

AR00041324

Photo 9. Viewing downstream at the Waiawa Stream channel in the Banana Patch reach.

Photo 10. Viewing downstream to the west-bound Farrington Highway bridge.

Photo 11. Viewing upstream along Waiawa Stream from underneath the west-bound Farrington Highway bridge; debris caught on pier in foreground.

Photo 12. Viewing downstream from underneath the west-bound Farrington Highway bridge.

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Photo 13. Viewing towards the left bank under the west-bound Farrington Highway bridge; note the deposition bars and scouring of the banks.

Photo 14. Local left bank erosion upstream of west-bound Farrington Highway bridge.

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Photo 15. Viewing upstream to the west-bound Farrington highway bridge; note the fill downstream of the bridge (left in photo) blocking all but two bridge spans.

Photo 16. Viewing the fill on the right bank under and downstream of the west-bound Farrington Highway bridge.

Photo 17. Viewing upstream from the east-bound Farrington Highway bridge. The USGS gage was located at the concrete weir in the foreground.

Photo 18. Viewing the upstream right bank of the east-bound Farrington Highway bridge.

Photo 19. Viewing downstream along Waiawa Stream towards Interstate H-1 from underneath the east-bound Farrington Highway bridge.

AR00041330

APPENDIX B

HEC-SSP Output from Re-analysis of Waiawa Stream Record

AR00041331

Appendix B. HEC-SSP Output from Re-Analysis of Waiawa Stream Record

```
------------------------------
Bulletin 17B Frequency Analysis
    06 Oct 2008   08:43 AM
------------------------------


--- Input Data ---

Analysis Name: test
Description:

Data Set Name: waiawa
DSS File Name: C:\ssptest\test\test.dss
DSS Pathname: ///FLOW-PEAK/01jan1900/IR-CENTURY//

Report File Name: C:\ssptest\test\Bulletin17bResults\test\test.rpt
XML File Name: C:\ssptest\test\Bulletin17bResults\test\test.xml

Start Date:
End Date:

Skew Option: Use Weighted Skew
Regional Skew: -0.05
Regional Skew MSE: 0.302

Plotting Position Type: Median

Upper Confidence Level: 0.05
Lower Confidence Level: 0.95

Display ordinate values using 0 digits in fraction part of value

--- End of Input Data ---



----------------------
<< Low Outlier Test >>
----------------------
 Based on 52 events, 10 percent outlier test value K(N) = 2.783

       0 low outlier(s) identified below test value of 1,063.85



----------------------
<< High Outlier Test >>
----------------------
 Based on 52 events, 10 percent outlier test value K(N) = 2.783

     0 high outlier(s) identified above test value of 61,156.18
```

--- Final Results ---

<< Plotting Positions >>
waiawa

Events Analyzed			Ordered Events			
Day Mon Year	FLOW cfs		Rank	Water Year	FLOW cfs	Median Plot Pos
19 Nov 1952	1,790		1	1982	27,950	1.34
01 Jan 1954	1,500		2	1991	27,600	3.24
28 Nov 1954	16,900		3	1968	23,400	5.15
25 Feb 1956	13,500		4	1980	19,900	7.06
21 Jan 1957	6,080		5	1955	16,900	8.97
05 Mar 1958	7,810		6	1988	16,200	10.88
23 Oct 1958	7,320		7	1966	15,800	12.79
14 May 1960	11,500		8	1963	15,500	14.69
13 Feb 1961	3,220		9	1970	15,400	16.60
12 Mar 1962	2,240		10	1994	14,270	18.51
14 May 1963	15,500		11	1992	14,200	20.42
22 Mar 1964	2,690		12	1996	14,000	22.33
02 May 1965	14,000		13	1965	14,000	24.24
14 Nov 1965	15,800		14	1974	13,700	26.15
11 Oct 1966	11,000		15	1956	13,500	28.05
05 Jan 1968	23,400		16	1969	13,300	29.96
03 Jan 1969	13,300		17	1989	12,700	31.87
25 Jul 1970	15,400		18	1972	12,400	33.78
26 Nov 1970	9,080		19	2000	11,600	35.69
15 Apr 1972	12,400		20	1987	11,600	37.60
09 Jul 1973	2,900		21	1960	11,500	39.50
19 Apr 1974	13,700		22	1979	11,000	41.41
21 Nov 1974	8,760		23	1967	11,000	43.32
27 Nov 1975	8,300		24	1997	10,300	45.23
09 Jun 1977	5,640		25	2004	10,000	47.14
28 Jun 1978	4,780		26	1990	9,720	49.05
10 Feb 1979	11,000		27	2002	9,680	50.95
18 Mar 1980	19,900		28	1993	9,180	52.86
04 Aug 1981	7,380		29	1971	9,080	54.77
28 Oct 1981	27,950		30	1986	9,000	56.68
28 Oct 1982	5,170		31	1975	8,760	58.59
19 Apr 1984	3,950		32	1976	8,300	60.50
27 Nov 1984	4,130		33	1958	7,810	62.40
20 Oct 1985	9,000		34	1981	7,380	64.31
12 Jun 1987	11,600		35	1959	7,320	66.22
31 Dec 1987	16,200		36	1957	6,080	68.13
21 Jul 1989	12,700		37	1977	5,640	70.04
03 Oct 1989	9,720		38	1983	5,170	71.95
21 Mar 1991	27,600		39	1978	4,780	73.85
03 Sep 1992	14,200		40	2003	4,220	75.76
26 Dec 1992	9,180		41	1985	4,130	77.67
18 Sep 1994	14,270		42	1995	4,050	79.58
23 Aug 1995	4,050		43	1984	3,950	81.49
25 Jan 1996	14,000		44	1961	3,220	83.40
03 Jan 1997	10,300		45	1999	3,080	85.31
03 Oct 1997	2,200		46	1973	2,900	87.21
07 Jan 1999	3,080		47	1964	2,690	89.12
02 Dec 1999	11,600		48	2001	2,430	91.03
29 Oct 2000	2,430		49	1962	2,240	92.94
06 May 2002	9,680		50	1998	2,200	94.85
10 Apr 2003	4,220		51	1953	1,790	96.76
04 Aug 2004	10,000		52	1954	1,500	98.66

<< Skew Weighting >>

Based on 52 events, mean-square error of station skew = 0.142
Mean-square error of regional skew = 0.302

<< Frequency Curve >>
waiawa

Computed Curve FLOW, cfs	Expected Probability	Percent Chance Exceedance	Confidence Limits 0.05 FLOW, cfs	0.95
45,838	---	0.2	67,245	34,349
39,745	---	0.5	56,905	30,280
35,128	---	1.0	49,267	27,137
30,507	---	2.0	41,813	23,931
24,389	---	5.0	32,280	19,572
19,740	---	10.0	25,338	16,148
15,032	---	20.0	18,635	12,544
8,477	---	50.0	10,046	7,172
4,455	---	80.0	5,330	3,606
3,092	---	90.0	3,800	2,387
2,252	---	95.0	2,854	1,657
1,195	---	99.0	1,628	791

<< Systematic Statistics >>
waiawa

Log Transform: FLOW, cfs		Number of Events	
Mean	3.9067	Historic Events	0
Standard Dev	0.3161	High Outliers	0
Station Skew	-0.5800	Low Outliers	0
Regional Skew	-0.0500	Zero Events	0
Weighted Skew	-0.4107	Missing Events	0
Adopted Skew	-0.4107	Systematic Events	52

APPENDIX C

FEMA Certification of a No-Rise Determination

AR00041335

Federal Emergency Management Agency

Washington, D.C. 20472

CERTIFICATION OF A "NO-RISE" DETERMINATION

FOR A PROPOSED FLOODWAY DEVELOPMENT

City and County of Honolulu, Hawaii
Community Name

Pearl Highlands Station and Parking Facility
Development Name

Lot/Property Designation

Property Owner

I hereby certify that the proposed remedial measures, in combination with the property development designated above, will result in no loss of flow conveyance during the occurrence of the 1 percent annual chance of exceedence (100-year flood) discharge.

I further certify that the data submitted herewith in support of this request are accurate to the best of my knowledge, that the analyses have been performed correctly and in accordance with sound engineering practice, and that the proposed structural works are designed in accordance with sound engineering practice.

April 21, 2010
Date

Robert C. MacArthur
Registered Professional Engineer

Seal

AR00041336

APPENDIX D

FEMA Certification Requirements for Simple Floodway Encroachments

AR00041337

Federal Emergency Management Agency

Washington, D.C. 20472

CERTIFICATION REQUIREMENTS FOR SIMPLE FLOODWAY ENCROACHMENTS

Introduction

This document is intended to provide guidance to local floodplain administrators in evaluating requests for the placement of fill at a single location, a building, or another simple encroachment within an adopted regulatory floodway. The procedure contained in this document is not intended to evaluate complex encroachments, such as extensive fills, multiple structures, bridges, or levees, where flow expansion and contraction losses may be significant. In such cases, full hydraulic analyses by computer backwater models should be employed. The minimum floodplain management requirements for communities participating in the National Flood Insurance Program (NFIP) in which a regulatory floodway has been designated prohibit any floodway development that would result in an increase in flood levels within the community during the occurrence of the base (100-year) flood discharge. This requirement is outlined in Paragraph 60.3(d)(3) of the NFIP regulations.

Because floodway development is contradictory to the tenets of sound floodplain management, such development is discouraged by the Federal Emergency Management Agency (FEMA). Therefore, these certification requirements assume that all practical alternatives to floodway development have been investigated thoroughly and have been deemed not feasible.

In accordance with the NFIP regulations, it is ultimately the responsibility of each community participating in the NFIP to prohibit floodway development that would result in increases in 100-year flood levels. Communities must make determinations of this type and maintain backup calculations and certifications in their files for review by FEMA personnel upon request.

This document also does not address the many cases and situations requiring the actual revision of the floodway via redelineation of the floodway boundary, the criteria for which are presented in Section 65.7 of the NFIP regulations. A FEMA document, entitled "Conditions and Criteria for Floodway Revisions," and dated August 27, 1984, addresses these issues.

Definition of "No-Rise"

It is important that the concept of "no rise" be clarified and understood. The actual wording of Paragraph 60.3(d)(3) of the NFIP regulations is that the community shall "prohibit ... any increase in flood levels during the occurrence of the base flood discharge." If an adopted regulatory floodway has been computed and is displayed on the effective NFIP map, all areas within the floodway are considered to be effective in conveying the 100-year flood discharge. Therefore, no obstruction, regardless of size, can be

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placed within the floodway without obstructing flow and causing some increase in the base flood elevation (BFE). Such increases may be localized and may seem insignificant; if modeled, they may yield changes on the order of hundredths or thousandths of a foot.

There has been a tendency to misinterpret the "no-rise" criterion to include rounding allowances and also to conclude that an increase in computed water-surface elevation of 0.01 foot or less, as determined by a backwater computer model, is sufficient evidence to support the acceptability of development in a floodway.

Although the backwater computer model output may show little change in water-surface elevation, closer examination will, in all likelihood, reveal changes in other variables (e.g., topwidth, flow area, velocity). These changes can be translated into increases in water-surface elevation that may not be considered significant by themselves. However, the long-term cumulative effects of such increases will eventually result in significant changes. Therefore, no development in the floodway should occur without proper compensation for the lost conveyance. This is the intent of Paragraph 60.3(d)(3) of the NFIP regulations.

It is FEMA's position that this regulation is to be interpreted exactly, and strictly, as written; that is, "no" rise above the BFE will be permitted. Therefore, nothing that offers any resistance to the flow of floodwaters may be placed within a regulatory floodway unless compensatory action is taken to restore the lost conveyance.

Loss of Conveyance

In the case of a simple floodway encroachment, as discussed previously, a "no-rise" determination can usually be made based on consideration of conveyance only. In such a situation, it is the difference in the conveyance before and after encroachment, or the aforementioned loss of conveyance, that must be addressed if the effects of development are to be compared against the "no-rise" criterion. The computation of loss of conveyance is most appropriately accomplished on a micro scale by isolating a portion of a cross section, separate from the backwater computer model, and performing hand computations. Examples of typical hand computations for proposed fill and bridge pier construction are attached for reference. However, it is appropriate to incorporate one or more new cross sections at the site of the proposed construction that reflect existing conditions into the unencroached and encroached backwater computer models. This is done to establish the base flood conditions at that location, which are to be used in the hand computations. The formula used to determine conveyance (K) is

$$K = 1.49/n\ AR^{2/3}$$

where n = Manning's roughness coefficient
A = Flow area
R = Hydraulic radius

The loss of conveyance is computed using the "n" value and hydraulic radius at the site of the encroachment as applied in the computer model.

Compensation

Once a determination has been made as to the amount of conveyance lost as a result of the proposed development, the designer or engineer is required to adequately compensate for this loss. This compensation is accomplished by including some means or measures within the proposed floodway development for providing an increase in effective conveyance, at some point on the cross section, equal to or greater than that lost. Equal area exchanges are only valid if the "n" value and hydraulic radius remain unchanged between the encroachment site and the compensation site. It is also important that the flow area provided be truly effective; that is, open to inflow and outflow and not just an isolated low spot or depression. This increased effective conveyance could be computed by hand in a manner similar to that used to compute the loss of conveyance.

The means or measures used to provide this effective conveyance (e.g., excavation, roughness coefficient reduction) would be at the discretion of the designer or engineer but must be approved by the community. Where these means and measures require some form of maintenance, the community must assume ultimate responsibility for their maintenance.

Data Requirements

The items identified below are necessary to document and demonstrate compliance with the "no-rise" criterion for simple floodway encroachments.

1. Hydraulic backwater models of the 100-year flood and floodway water-surface profiles for the following:

   a. Duplicate of the effective Flood Insurance Study (FIS) model.

   b. Existing-conditions (effective FIS) model modified to include cross sections through the project site. Cross sections must reflect conditions prior to construction of the project.

   c. Post-project conditions model. This model must include cross sections through the project site reflecting floodplain conditions after construction of the project. The 100-year flood (without floodway) and floodway elevations for this model must not be greater than those in the existing-conditions model described at letter "b" above. This hydraulic backwater model is necessary to ensure that any changes in transition losses, which are based on velocity heads rather than conveyance, do not cause increases in water-surface elevations. Also, a hydraulic backwater model provides a means of evaluating effective flow areas upstream and downstream of the encroachment and compensation sites.

2. A copy of the appropriate NFIP map showing the existing floodway and indicating the project area

3. Topographic mapping of the entire project area indicating the locations of all cross sections used in the modified hydraulic model and a plan view of all project elements

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4. Construction plans, certified by a registered professional engineer, for all project elements, including those measures employed to provide additional effective conveyance

5. The following information, to be obtained by hand computation using the cross section and 100-year encroached hydraulic data in the modified existing-conditions computer model output provided under Item 1b:

   a. Calculation of the reduction in conveyance (K) caused by the proposed obstruction, assuming no change in floodway water-surface elevation, and using the "n" value appropriate for the site of the proposed obstruction

   b. Calculation of the increase in conveyance (K) obtained by the proposed offsetting measure, using the "n" value appropriate for the site of this measure

   c. Comparison showing that the conveyance increase computed in 5b equals or exceeds the loss computed in 5a

6. Evidence that the increase in effective conveyance provided for in Item 5b will be maintained perpetually. This should be in the form of a self-maintaining measure or certified maintenance plans for the measure provided.

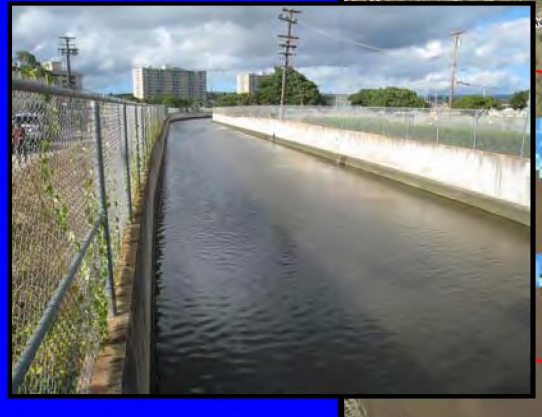
7. An executed copy of the attached certification statement signed and sealed by a registered professional engineer

A copy of the hydraulic computer model for the effective FIS for the specified stream and community may be obtained by written request to the following address:

Federal Emergency Management Agency
Federal Insurance Administration
Risk Studies Division
500 C Street, SW
Washington, D.C. 20472

AR00041341

ADVANCED FLOODPLAIN EVALUATIONS FOR WAIPAHU TRANSIT CENTER STATION - WAIKELE, KAPAKAHI AND WAILANI STREAMS



DRAFT

Prepared for:

Parsons Brinckerhoff Inc.
1001 Bishop Street, Suite 2400
Honolulu, HI 96813
808-531-7094

Prepared by:

Northwest Hydraulic Consultants Inc.
3950 Industrial Boulevard, Suite 100C
West Sacramento, CA 95691
916-371-7400

April 30, 2010

List of Abbreviations

1-d	one-dimensional
2-d	two-dimensional
ARF	area reduction factor in FLO-2D model
BFE	Base Flood Elevation
ControlPoint	ControlPoint Surveying, Inc.
DPP	City and County of Honolulu Department of Planning and Permitting
DTM	digital terrain model
FEMA	Federal Emergency Management Agency
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
FLO-2D	2-d hydraulic model
GDS	grid develop system in FLO-2D model
GEC	General Engineering Consultant
GIS	geographic information system
HEC-RAS	Hydrologic Engineering Center's River Analysis System, 1-d hydraulic model
HHCTCP	Honolulu High-Capacity Transit Corridor Project
JCLS	John Chance Land Surveys, Inc.
LiDAR	Light Detection and Ranging
LAI	Lyon Associates, Inc.
MSE	mean square error
NFIP	National Flood Insurance Program
NHC	Northwest Hydraulic Consultants, Inc.
Park	Park Engineering
PB	Parsons Brinckerhoff
PE/EIS	Preliminary Engineering and Environmental Impact Statement
TIN	triangulated irregular network
USACE	U.S. Army Corps of Engineers
USGS	U.S. Geological Survey
WRF	width reduction factor in FLO-2D model
WY	water year

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EXECUTIVE SUMMARY

The Honolulu High-Capacity Transit Corridor Project (HHCTCP) is an elevated rail line that will provide rapid public transportation service in the City and County of Honolulu on the island of Oahu, Hawaii. The Federal Emergency Management Agency (FEMA) and the City and County of Honolulu Department of Planning and Permitting (DPP) require that all segments of the HHCTCP comply with their floodplain regulations and "no rise" policies, i.e. project features will not cause any increase in regulatory flood elevations within floodway zones. According to the existing FEMA Flood Insurance Study (2004) and flood maps (FIRMS), the proposed HHCTCP Waipahu Transit Centre Station and guideway support columns are located within the mapped floodway of the Waikele-Kapakahi-Wailani stream system. However, existing FEMA effective models of the streams do not allow accurate assessment of the potential impacts of the proposed station and guideway support columns on water levels within the Waipahu Town area. The FEMA models are outdated, oversimplified, one-dimensional (1-d) models that do not account for the effects of densely spaced residential and commercial buildings and roads on flow conveyance and water levels. Therefore, the present effective models do not adequately depict the spatially complex floodplain hydraulic conditions within the heavily urbanized Waipahu Town area. Also, the present FEMA models and mapping assume steady-state flows which over-simplify the flashy characteristics of floods (short duration with volume-limited hydrographs) observed on the Waikele, Kapakahi, and Wailani Streams. Therefore, it was decided to re-evaluate present hydraulic conditions and future with-project conditions with more up-to-date sophisticated models in order to (1) assess complex flow characteristics within the Waipahu study area, (2) to determine how flows may affect HHCTCP features located on the densely urbanized floodplain, and (3) to develop alternative project designs that minimize potential impacts during a 100-year flood. Results from these models were used to better understand local flow conditions (depths and velocities) that may affect the project and to help Parsons Brinckerhoff's (PB's) design team to develop project designs that are safe and comply with DPP and FEMA floodplain development regulations. The models incorporate up-to-date channel and floodplain geometry, existing structures and road networks in the area, and real flow hydrographs rather than assuming steady flows.

Parsons Brinckerhoff (PB) is the General Engineering Consultant (GEC) providing planning and engineering services for the Preliminary Engineering and Environmental Impact Statement (PE/EIS) phase of the project. PB contracted with Lyon Associates, Inc. (LAI), who in turn sub-contracted with Northwest Hydraulic Consultants, Inc. (NHC) to address flood related design considerations and floodplain regulation compliance for the project. The main objectives of this study undertaken by NHC were to evaluate existing flow conditions in the Waipahu floodplain area, assess potential impacts caused by proposed HHCTCP structures on 100-year flood levels within the Waipahu Town area, and develop a project alternative that has no impact on flood elevations or floodplain storage. NHC's tasks included (1) field inspection of the project area, (2) collection of the pertinent topographic and hydraulic data needed for computer simulation of the flooding in the vicinity of the project station, (3) hydrologic analysis, (4) one-dimensional (1-d) hydraulic modeling of in-stream flows (using HEC-RAS model), (5) two-dimensional (2-d) modeling of overland flooding (using FLO-2D model), and (6) evaluation of several project alternatives.

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The Waipahu study area encompasses approximately 1.6 square miles of heavily urbanized Waipahu Town and open lands between the West and Middle Lochs of Pearl Harbor. During heavy winter rainfalls, the Waipahu Town area is subject to frequent flooding from local streams. The stream network in the study area is comprised of Waikele Stream, Kapakahi Stream, and Wailani Canal, as well as a few small drainage canals. All of the streams flow into Pearl Harbor and are tidally influenced in the lower reaches. The major source of overland flooding in Waipahu comes from Waikele Stream. During large flood events, flow from Waikele Stream combines with flows from the other smaller flooding sources which include local runoff to Kapakahi Stream and Wailani Canal. The existing earthen levee between Waikele Stream and the Waipahu Cultural Garden Park as well as flood walls along Wailani Canal are not "certified" structures and as such are not recognized by FEMA as providing flood protection to the Waipahu area.

A field inspection of the study area was jointly conducted by PB, LAI and NHC on January 15, 2009 and involved documentation of channel and floodplain conditions along the streams in the study area, pertinent hydraulic features, locations where ponding and ineffective flow are likely, and high water mark locations left during the most recent significant flood event that occurred on December 11, 2008. Following the site inspection, NHC developed a detailed program for collecting channel survey data needed to support development of 1-d and 2-d models and the hydraulic analyses. The surveys were conducted by ControlPoint Surveying, Inc. (ControlPoint) in the spring of 2009 and included measurement of cross-section profiles between high banks (levees) at various locations along the study streams. Also surveyed were details of the USGS weir on Waikele Stream upstream of Farrington Highway, geometry of bridges and culverts, flood walls along Wailani Canal, and the Waikele Stream levee adjacent to the Waipahu Cultural Garden Park. Additional topographic data used in this study included 1-ft contour interval Light Detection and Ranging Data (LiDAR)-based topographic data within the HHCTCP corridor obtained by John Chance Land Surveys, Inc. (JCLS) in 2007 (official topographic data prepared for the project corridor) and 5-ft by 5-ft grid FEMA LiDAR data developed in 2006 for the FEMA Hurricane Study that covers the entire Oahu coastline including Waipahu Town area. For consistency with the project drawings, the official project topography data were used for the HHCTCP corridor, while the FEMA LiDAR data were used for the areas outside of the project corridor. For use in this study, NHC developed a modified topographic data set for the entire study area in which the 2007 project corridor topography data was inserted into the 2006 FEMA LiDAR data to create one continuous topographic data set that covers the entire Waipahu study area.

Up-to-date hydrologic data were developed to support hydraulic analysis of potential impacts of the proposed HHCTCP facilities on 100-year flood levels in the Waipahu Town area. Based on the most recent USGS streamflow data, NHC updated the 100-year peak flow values and developed 100-year flood hydrographs for Waikele Stream, Kapakahi Stream, and Wailani Canal for use in the 1-d and 2-d models.

Using present day topographic data, channel cross sections, and hydrologic information, NHC developed up-to-date 1-d HEC-RAS models of Waikele Stream, Kapakahi Stream, and Wailani Canal. The Waikele Stream HEC-RAS model includes a 1.1 mile long reach of the stream that

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extends from just upstream of the Waipahu Cultural Garden Park to the outlet into West Loch in Pearl Harbor. The Kapakahi Stream model includes the entire 1.2 mile long channel of the stream from the Waipahu Cultural Garden Park to West Loch. The Wailani Canal model contains about 1.2 mile long reach of the canal between Paiwa Street and its mouth, including west- and east-side drainage canals running along the Energy Corridor. The main intent of these 1-d HEC-RAS models was to support the development and application of the more sophisticated 2-d FLO-2D model of the entire Waipahu floodplain area.

The HEC-RAS model of Waikele Stream was used to simulate flow characteristics for with-levee conditions (with the existing levee between Waikele Stream and the Waipahu Cultural Garden Park in place) and for no-levee conditions (to comply with FEMA's regulations which do not recognize this non-certified levee as providing flood protection to the area). Hydrographs simulated from the 1-d model for Waikele Stream in-channel flows below Farrington Highway and breakout flows from Waikele Stream to the Waipahu area upstream of Farrington Highway were used to define upstream inflow boundary conditions for the 2-d model developed for the Waipahu floodplain area. HEC-RAS depth-discharge results for hydraulic structures on Waikele Stream, Kapakahi Stream, and Wailani Canal were used to specify rating tables at these structures in the 2-d floodplain model.

A set of 2-d unsteady fixed-bed FLO-2D computer models was developed for the Waipahu floodplain system to simulate 100-year flooding conditions in the vicinity of the proposed Waipahu Transit Station and Guideway features. FLO-2D provides a well tested tool for quantifying spatially variable flow hydraulics in complex riverine and floodplain environments similar to the Waipahu study area (particularly shallow urban flooding) and is accepted by FEMA for floodplain assessments and mapping. Two models with different spatial coverage and grid resolution were developed: (1) a primary area-wide coarse grid model (with 50 ft grid cell size) and (2) a supporting smaller-area fine gird model (with 10 ft grid cell size). The coarse grid model covers approximately 1.6 square miles of the study area and includes the lower reaches of Waikele Stream, Kapakahi Stream, and Wailani Canal. The fine grid model focuses on a 0.4 square mile area along Farrington Highway between Waikele Stream and Wailani Canal. The coarse grid model was used to evaluate area-wide flooding characteristics with and without proposed project features included. The fine grid model was used to simulate and assess more local flow conditions and understand more about flooding details in the immediate vicinity of the proposed project structures. It was also used to check the reasonableness of the coarse grid model predictions. Results from these models were used to help PB's design team to develop alternative project designs that minimize potential impacts during a 100-year flood and to develop project designs that are safe and comply with DPP and FEMA floodplain development regulations.

The FLO-2D models of the Waipahu area were developed using a topographic data set that combined the 2007 LiDAR-based topographic data along the project corridor and the 2006 FEMA LiDAR data for the rest of the floodplain area. Stream channels were developed using the 2009 cross-section data. Inflow boundary conditions were developed from up-dated hydrology and results from the 1-d model simulations.

The NHC-developed 2-d models were used to simulate 100-year flooding in the Waipahu Town area under existing and with-project conditions and to develop and evaluate project alternatives

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that reduce or eliminate impacts to water levels in the floodway caused by the project. Two flooding scenarios were evaluated for both existing and with-project conditions: (1) with the existing east bank levee in place on Waikele Stream at the Waipahu Cultural Garden Park and with existing flood walls along Wailani Canal (to evaluate the most likely flooding scenario), and (2) without the levee or flood walls (to comply with FEMA's regulations that require removal of all non-certified flood control structures from model simulations). Results from these simulations indicate that the 100-year flood event will cause widespread flooding in the Waipahu Town area for both with-levee and no-levee conditions. The overall flooding pattern is generally similar for both of these flooding scenarios; however, the FEMA "no-levee" scenario is more severe. Flood waters break out of Waikele Stream upstream of Farrington Highway and flow in the easterly direction north of Farrington Highway, overtop Farrington Highway, and then travel southerly over a wide portion of the floodplain between Kapakahi Stream and Wailani Canal. Capacities of Kapakahi Stream and Wailani Canal are insufficient to accommodate overland inflows from Waikele Stream which results in significant overland (floodplain) flooding along both these streams.

According to the FLO-2D model results, construction of project station features as they were originally designed will reduce local floodplain storage capacity, confine flow along Farrington Highway, create significant obstruction to the swift overland flow (especially adjacent to the south entrance building), and thereby will increase water levels along Farrington Highway by approximately 0.1-0.4 ft for with-levee conditions and by 0.2-0.5 ft for no-levee conditions. According to the 2-d model results, placement of the 6-foot diameter guideway support columns along the Farrington Highway median will not affect flood depths due to their relatively small size, shallow flow depths and wide spacing between columns.

Because the original project designs for the north and south entrance buildings caused a rise in water levels, three additional project alternatives were evaluated using the FLO-2D models. The three additional alternatives include: (Alt 1) north building only and complete removal of the south entrance building, (Alt 2) north building and reduced south entrance building footprint (reduced to approximately half its original size), and (Alt 3) the preferred alternative which includes an elevated plaza (above the 100-year flood level) and new north entrance building that fully fills the gap between the existing adjacent buildings and a new south entrance building to be located in the same approximate location with the same maximum external ground floor dimensions (approximately 20 by 40 feet) as the existing building that it replaces.

The FLO-2D model results indicate that Project Alternative 1 (north building only, no south entrance building) will produce no detectable impact on flood levels along Farrington Highway. However, as presently proposed this alternative allows transit flow through a narrow gap (presently blocked by existing buildings) along the north entrance building thereby increasing flow depths in this gap by 0.1-0.5 ft which violates the "no rise" policy. The rise caused by flow between the buildings can be eliminated by including an elevated plaza (above the 100-year flood level) all around the north entrance building.

According to the FLO-2D model results, Project Alternative 2 will still create a significant rise in water levels. Even though the size of the south entrance building in Alternative 2 is approximately half of its original design, it is still located in an extremely unfavorable flow area

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with significant flow depths and high velocities. Therefore, the south entrance building in Alternative 2 will produce a significant obstruction to flows that spill over Farrington Highway at this location. For Project Alternative 2, maximum inundation depths will increase at Farrington Highway between the station entrance buildings by approximately 0.1-0.3 ft for with-levee conditions and by 0.1-0.5 ft for no-levee conditions.

FLO-2D model results show that Project Alternative 3 (the preferred alternative) will have no detectable impact on 100-year flood elevations (within the inherent accuracy of the model) in the Waipahu Town area. The model shows no change in flooding pattern, inundation extent, and maximum inundation depths with the preferred alternative (Alt 3).

Results from these analyses provide PB's design team with a much clearer understanding of local flow conditions at the project site and sufficient information to prepare a safe project alternative that should comply with FEMA's and DPP's "no-rise" policy. Therefore, Alternative 3 is the preferred alternative and it will have no detectable impact on 100-year flood elevations in the Waipahu Town area. Alternative 3 calls for a new north entrance building to be constructed at the originally selected station location, but the plaza and ground floor for the north entrance building should be raised above the 100-year flood elevation (above approximately 13.5 ft HILOCAL). The pedestrian plaza adjacent to the north building should also be raised above the 100-year water level and occupy the entire width between the existing structures to match the present effects of the existing building that blocks the entire north-south gap between the existing adjacent structures. The south entrance building should be constructed in approximately the same location and be reduced in size to match the same maximum external ground floor dimensions (approximately 40 ft east to west by 20 ft north to south) of the existing building footprint which it replaces. However, if it is decided not to build the south entrance building at all, the existing small building at this location should be left in place or replaced by a structure with similar maximum external ground floor dimensions to preserve present day flow conditions. A supplemental report that explicitly addresses the no-rise and no net loss of floodplain storage requirements for DPP and FEMA was also prepared by NHC, entitled "No Rise Analysis for Waipahu Transit Center Station and HHCTCP Guideway Columns – Waikele, Kapakahi and Wailani Streams" (NHC April 30, 2010).

Peak overbank inflow from Waikele Stream to the Waipahu Town area used in this study is slightly greater than that used by FEMA (2004) in their flood inundation study. Therefore, the estimated project impacts reported herein are conservative, which adds a factor-of- safety to the results of this study.

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Advanced Floodplain Evaluations for Waipahu Transit Center Station - Waikele, Kapakahi and Wailani Streams

1. INTRODUCTION

The Honolulu High-Capacity Transit Corridor Project (HHCTCP) is an elevated rail line that will provide rapid public transportation service in the City and County of Honolulu on the island of Oahu, Hawaii. The Federal Emergency Management Agency (FEMA) and the City and County of Honolulu Department of Planning and Permitting (DPP) require that all segments of the HHCTCP comply with their "no-rise" policy, i.e. project features will have no impact on water levels within floodway zones. FEMA manages the National Flood Insurance Program (NFIP). NFIP floodplain management building requirements are described in Vol. 44 Code of Federal Regulations (44 CFR), Sections 59 through 65. DPP is the lead agency responsible for Flood Hazard District compliance in the City and County of Honolulu. DPP floodplain requirements are described in Section 21-9.10 - Flood Hazard Districts, in their Land Use Ordinance.

According to the existing FEMA flood maps, the proposed HHCTCP Waipahu Transit Centre Station and guideway support columns are located within the mapped floodway of the Waikele-Kapakahi-Wailani stream system. However, existing FEMA effective models of the streams do not allow accurate assessment of the potential impacts of the proposed station and guideway support columns on water levels within the Waipahu Town area. The FEMA models are outdated, oversimplified, one-dimensional (1-d) models that do not account for the effects of densely spaced residential and commercial buildings and roads on flow conveyance and water levels. Therefore, the present effective models do not adequately depict complex floodplain hydraulics within the heavily urbanized Waipahu Town area. Also, the FEMA models and mapping assume steady-state flows, which over-simplifies the flashy characteristics of floods (short duration, volume limited hydrographs) observed on the Waikele, Kapakahi, and Wailani Streams. Therefore, it was decided to re-evaluate present hydraulic conditions and future with-project conditions with a more sophisticated model which incorporates up-to-date channel and floodplain geometry, existing structures and road networks in the area, roughness information, and actual flow hydrographs rather than assuming steady flows.

Parsons Brinckerhoff (PB) is the General Engineering Consultant (GEC) providing planning and engineering services for the Preliminary Engineering and Environmental Impact Statement (PE/EIS) phase of the project. PB contracted Lyon Associates, Inc. (LAI), who in turn sub-contracted with Northwest Hydraulic Consultants, Inc. (NHC) to address the flood related design considerations and floodplain regulation compliance for the project. The main objective of this study is to evaluate existing flow conditions in the Waipahu floodplain area, assess potential impacts that HHCTCP project features could have on water levels during a 100-year flood event the Waipahu area, and develop a "no-rise" project design alternative. NHC's work involved field inspection of the project area, collection of the data required to prepare new flood simulation models for the project area, hydrologic analysis, one-dimensional (1-d) hydraulic modeling of in-

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stream flows (using HEC-RAS model), two-dimensional (2-d) modeling of overland flooding (using FLO-2D model), and evaluation of various project alternatives. Revision of the effective FEMA flood maps is outside the scope of this study. Following is a description of work and services performed by NHC.

2. STUDY AREA

The Waipahu study area encompasses approximately 1.6 square miles. It includes the heavily urbanized Waipahu Town area and the open lands between the West and Middle Lochs of Pearl Harbor (Figure 2.1). The stream network in the study area is comprised of Waikele Stream, Kapakahi Stream, and Wailani Canal, as well as a few small drainage canals. All the streams eventually discharge into Pearl Harbor and they are tidally influenced in the lower reaches. The floodplain area is relatively flat and is bordered by high ground on the north side of Waipahu Street between Waikele Stream and Wailani Canal. An historic map of the major streams in the Pearl Harbor area is shown in Figure 2.2. Historic aerial photographs of the lower Waikele Stream are shown in Figures 2.3-2.6.

The climate on the island of Oahu is generally mild, with fairly uniform temperatures. The wet season extends from October through April, the dry season from May through September. During heavy winter rainfalls, the Waipahu Town area is subject to frequent flooding from the local streams. The major source of overland flooding is Waikele Stream. During large flood events, flow from Waikele Stream combines with urban runoff from other small catchments and drainage sources, including local runoff in Kapakahi Stream and Wailani Canal. According to USACE (1982), primary causes of overland flooding in the Waipahu area include inadequate channel sections, inadequate bridge openings and culvert sections, and the cumulative effects of on-going urban development. More detailed information on individual stream characteristics is given in Chapter 3.

A portion of the official FEMA Flood Insurance Rate Map (FIRM) is shown for the study area in Figure 2.7. This map was digitized and revised by FEMA in September 30, 2004 and can be downloaded from FEMA's website (www.fema.gov). According to this map, a significant portion of the Waipahu Town area is located within the regulatory floodway of the Waikele-Kapakahi-Wailani stream system and is subject to flooding during the 100-year and higher flood events.

The proposed elevated guideway will be aligned along the center median of Farrington Highway, with the proposed transit station and station entrance buildings located approximately in the middle of the floodplain between Kapakahi Stream and Wailani Canal. Footprints of the originally proposed design for the project buildings and guideway support columns are shown in Figure 2.1. A three-dimensional (3-d) architectural rendering of the original design for the Waipahu Transit Station is shown in Figure 2.8.

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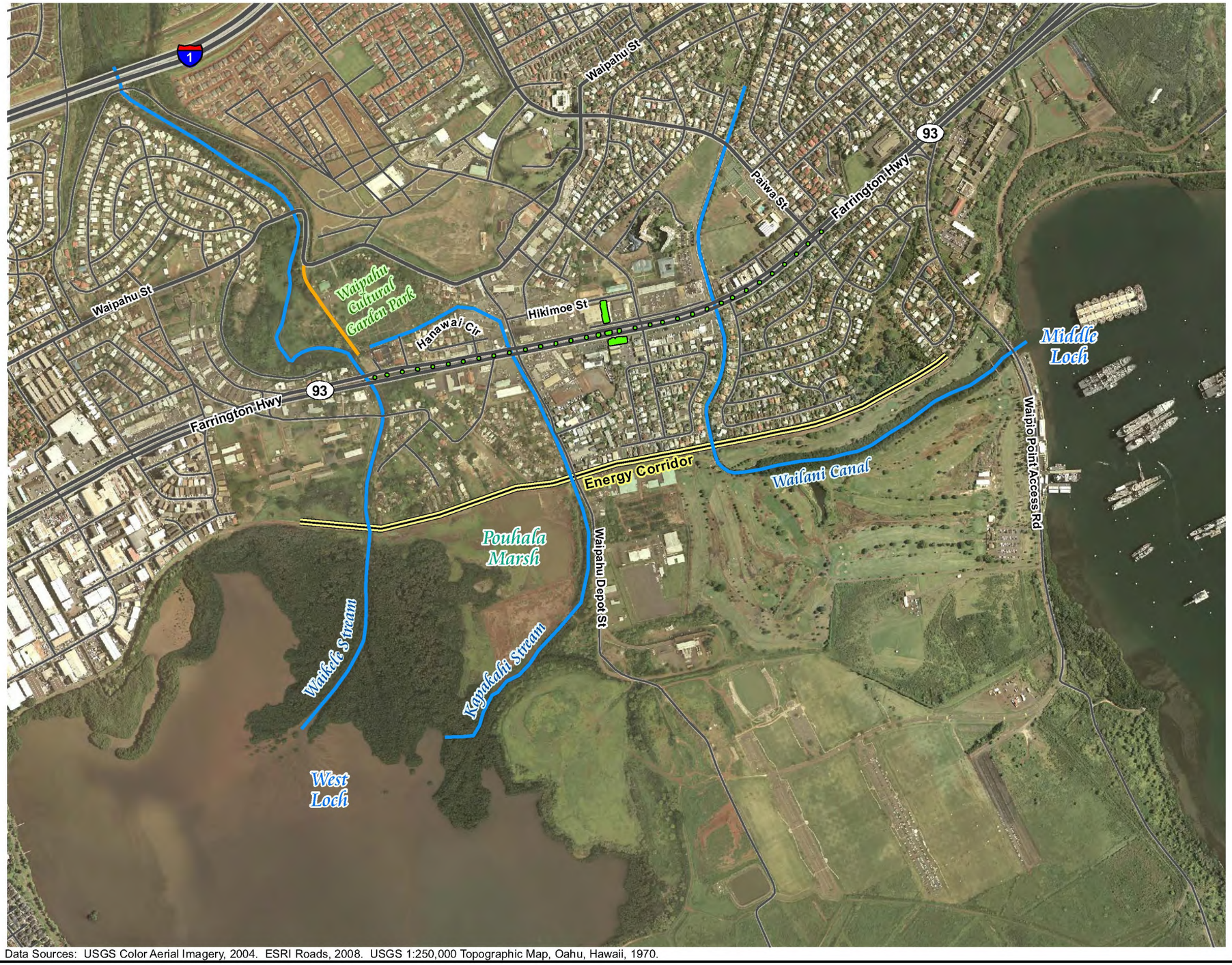


Data Sources: USGS Color Aerial Imagery, 2004. ESRI Roads, 2008. USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

Study Area

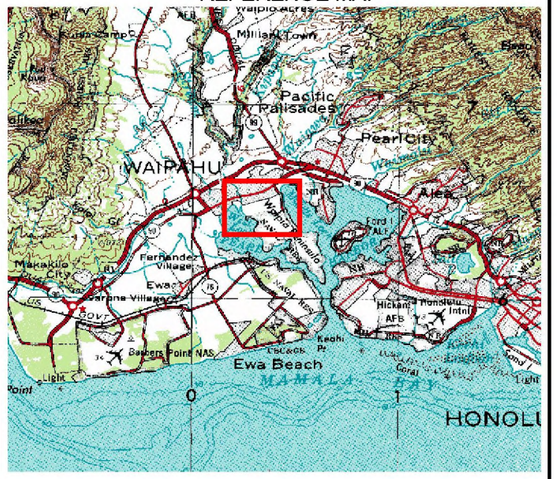
Scale - 1:12,000 1 inch = 1,000 feet

0 500 1,000 2,000 Feet

N W E S

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



LEGEND

- Project Buildings and Piers/Columns
- Roads
- Streams & Canals
- Levee
- Energy Corridor

Figure 2.1 - Study Area

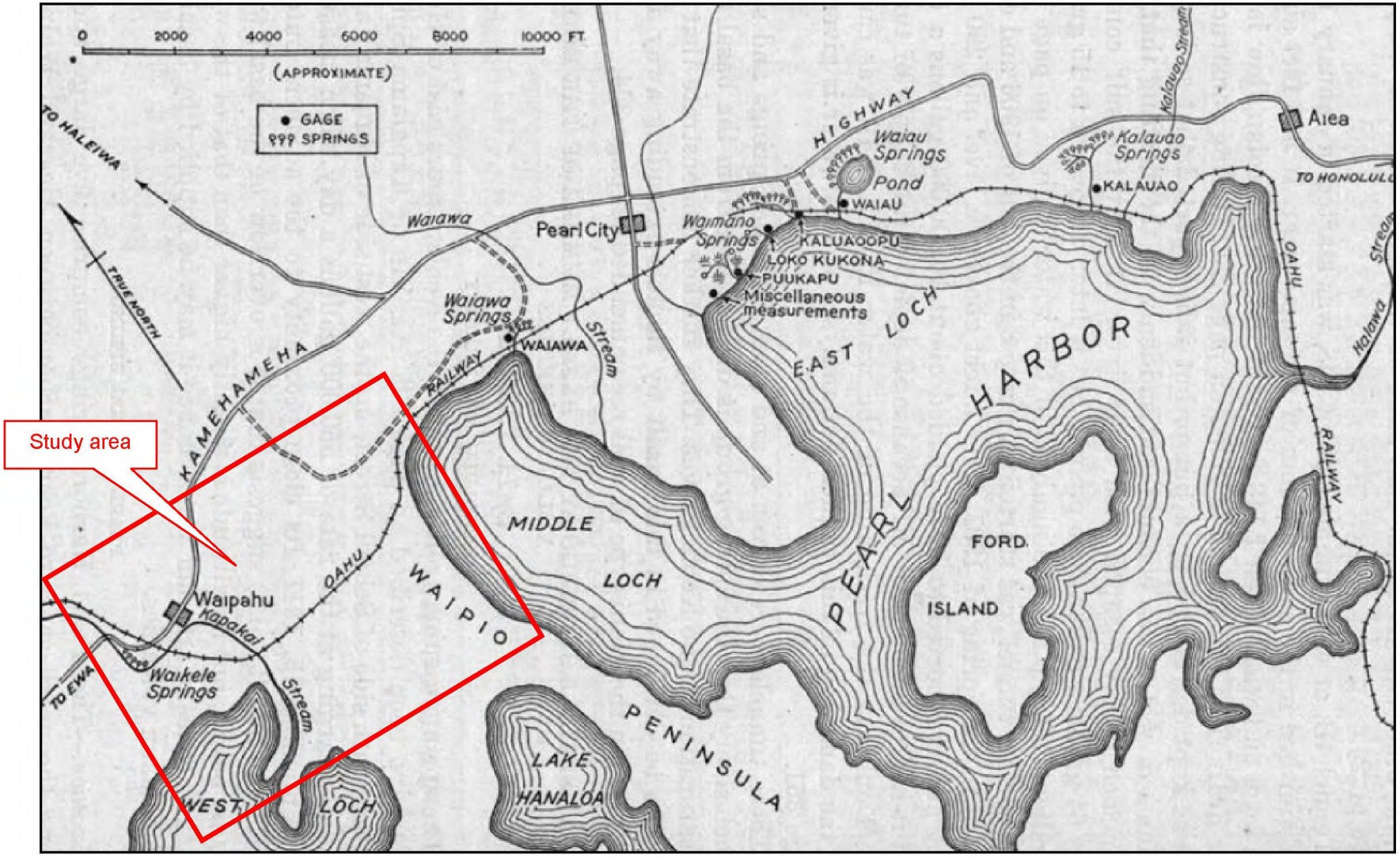


Figure 2.2. Historic map of major streams in Pearl Harbor area in 1935 (image from Englund et al 2000). Note absence of present day channel of Waikele Stream.

Figure 2.3. Lower Waikele Stream in 1951 (image from Englund et al 2000).

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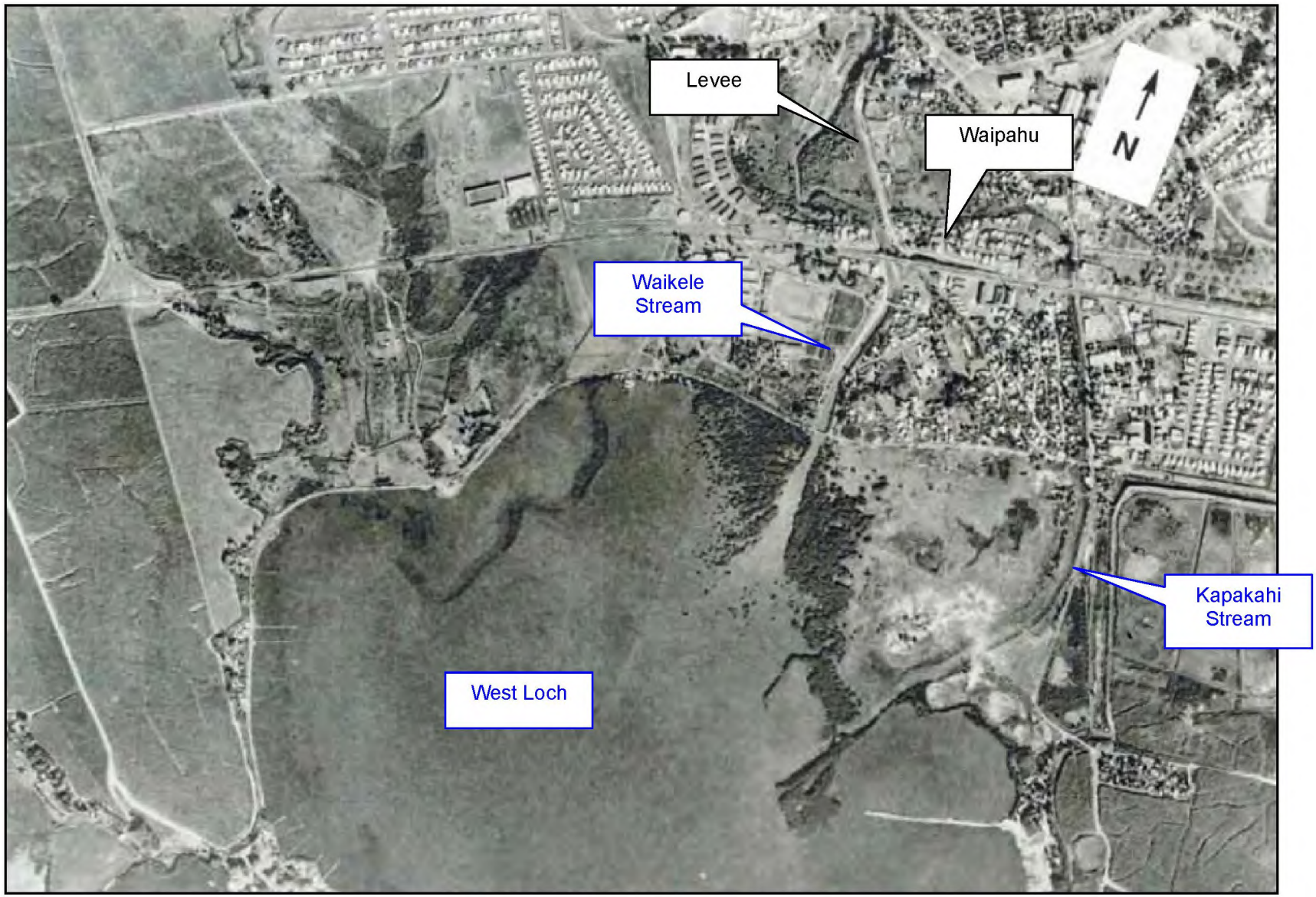

Figure 2.4. Lower Waikele Stream in 1965 (image from Englund et al 2000).

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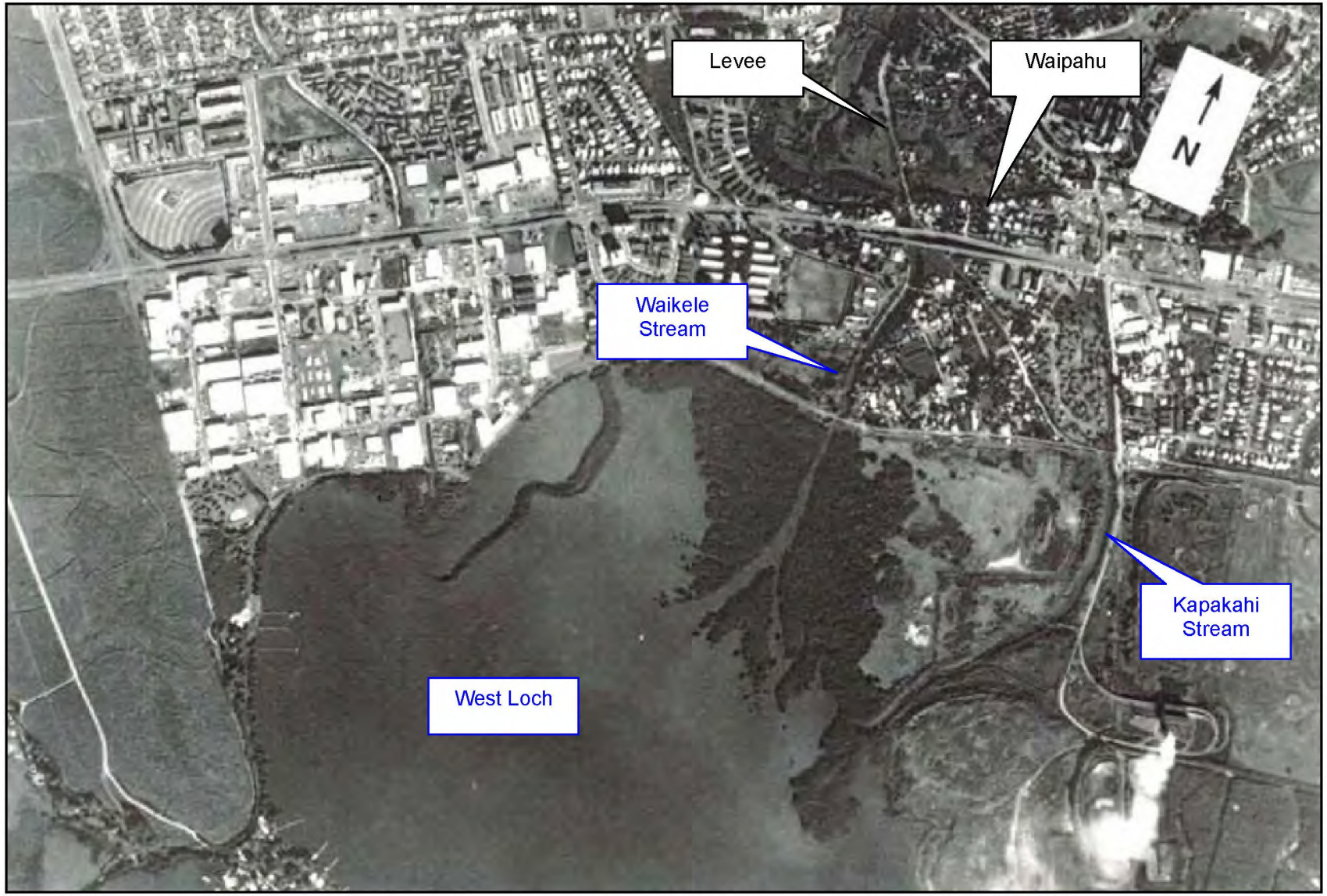

Figure 2.5. Lower Waikele Stream in 1975 (image from Englund et al 2000).

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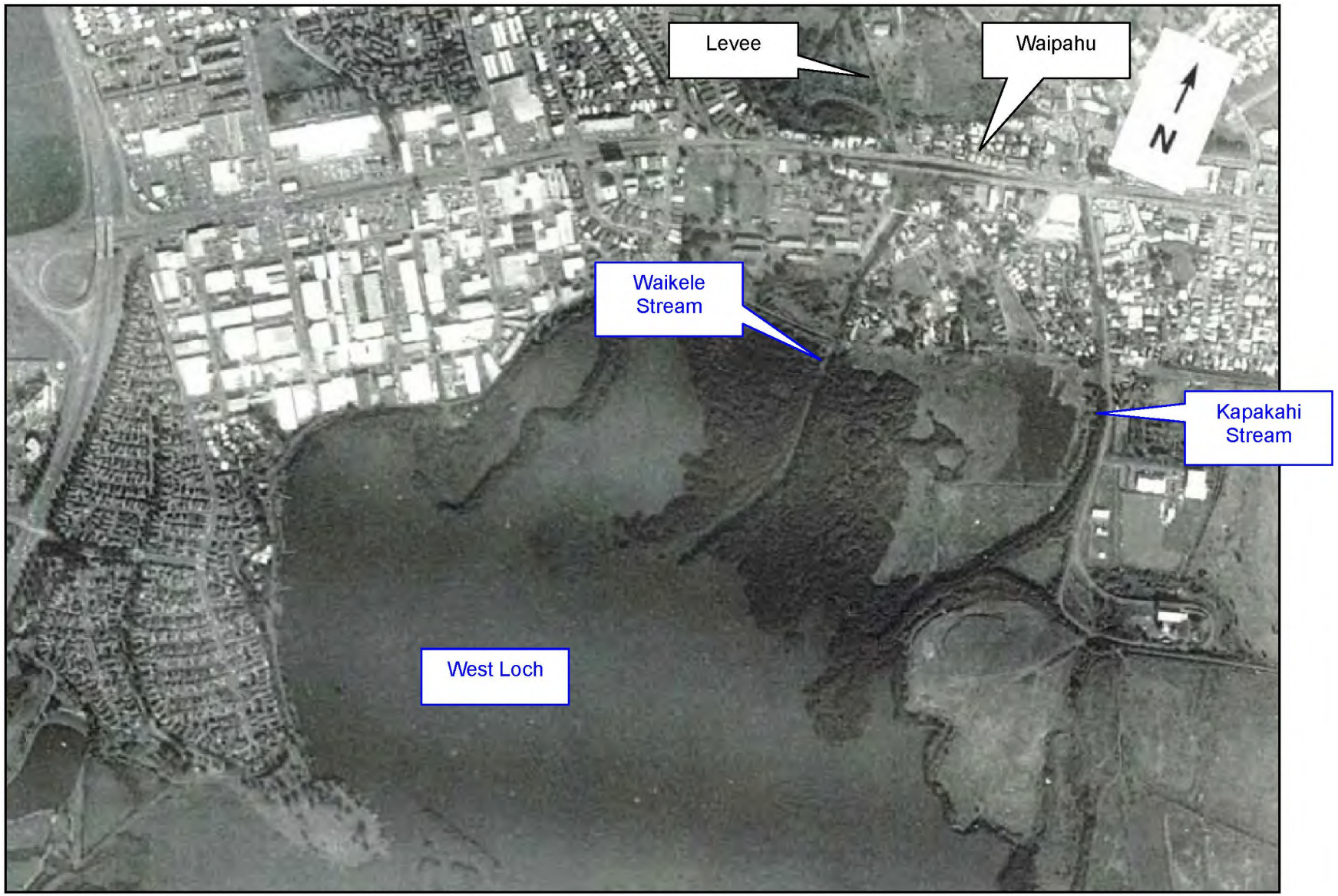

Figure 2.6. Lower Waikele Stream in 1995 (image from Englund et al 2000).

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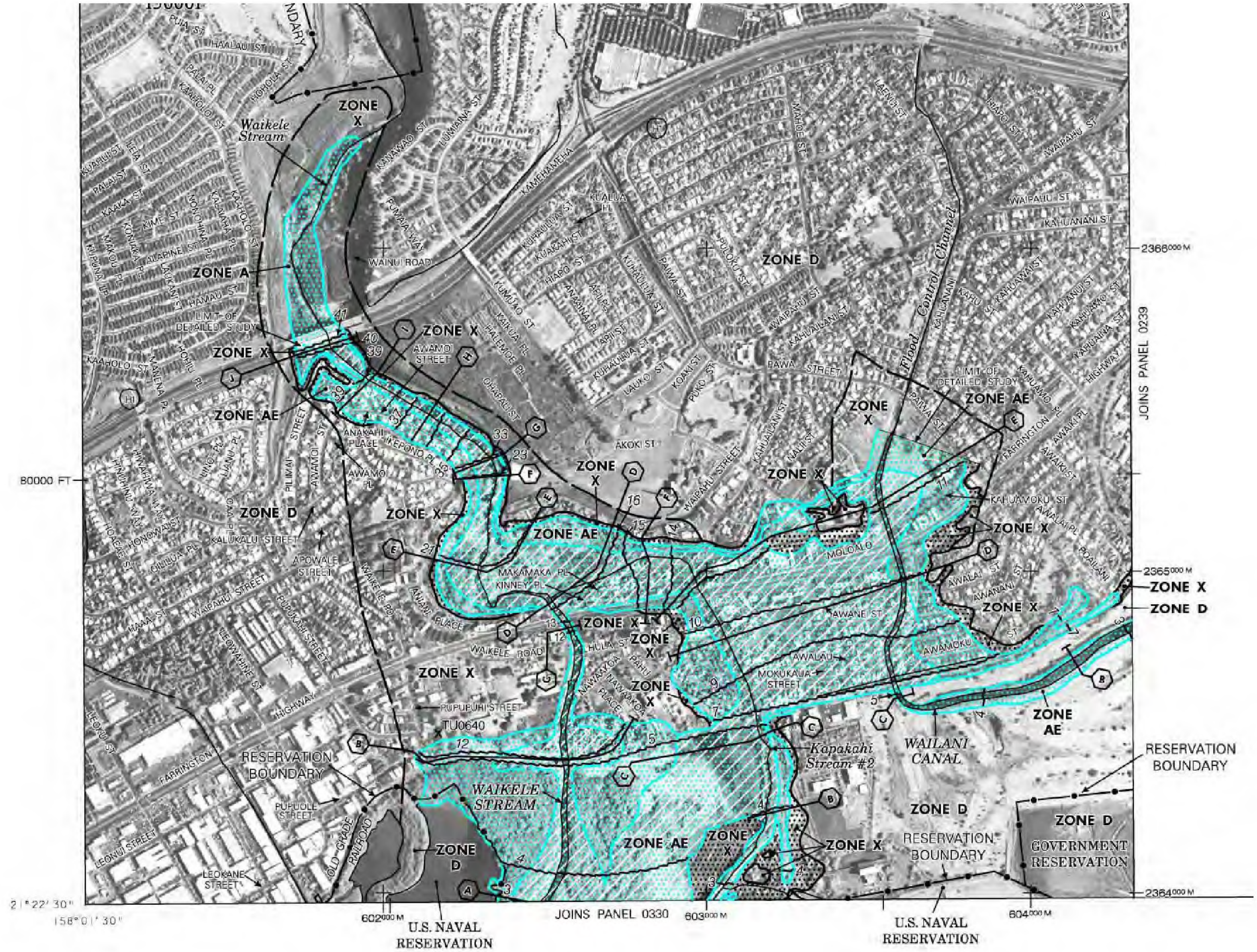

Figure 2.7. FEMA Flood Insurance Rate Map (FIRM, 2004).

Mauka (north) station

Makai (south) station

Figure 2.8. HHCTCP Waipahu Transit Station (original design concept).

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3. FIELD INVESTIGATIONS AND DATA COLLECTION

This chapter presents results from a field inspection of the study area and data collection activities undertaken to support development of 1-d models for the three study streams and 2-d models for the greater Waipahu floodplain area.

3.1. Site Inspection

A field inspection of the study area was conducted jointly by PB, LAI, and NHC on January 15, 2009. The purpose of the site inspection was to document channel and floodplain characteristics of the study streams, pertinent hydraulic features, locations where ponding and ineffective flow are likely, and high water mark locations left during the December 11, 2008 flood event. Results of the field inspection are discussed below.

3.1.1. Waikele Stream

Waikele Stream is the main watercourse that contributes to flooding in the Waipahu Town area. The Hawaiian meaning of the stream's name is "muddy water". The town's name means "bursting or exploding water", which indicates that Waipahu has experienced a long history of flooding problems. The reach of Waikele Stream considered in this study extends approximately one mile from the Waipahu Street Bridge to the mouth at the West Loch, Pearl Harbor. Ground photographs of the river collected during the field inspection are shown Figures 3.1-3.18. Waikele Stream flows southerly through a meandering two-stage channel between Waipahu Street and Farrington Highway (Figures 3.2, 3.9, 3.12); then it flows through a 1,100-ft-long, concrete-lined flood control channel downstream of Farrington Highway (Figures 3.14 and 3.15), and finally through a mangrove forest downstream of the Energy Corridor (Figure 3.18). The natural low flow channel ranges from about 40 ft to 70 ft wide. The floodplain width in the meandering reach is up to 500 ft on both sides of the channel. The stream in this reach is bordered by high banks and a man-made east bank levee located upstream of Farrington Highway and adjacent to the Waipahu Cultural Garden Park. The bottom width of the trapezoidal concrete flood control channel downstream from Farrington Highway is about 50 ft and the top width ranges from 90 to 110 ft.

The river in the study reach is crossed by five bridges (listed in the downstream direction): (1) Waipahu Street Bridge, (2) westbound Farrington Highway Bridge, (3) eastbound Farrington Highway Bridge, (4) a footbridge located approximately 330 ft downstream from Farrington Highway, and (5) an abandoned railroad bridge at the Energy Corridor. The bridges are shown in Figures 3.1, 3.13, 3.15, and 3.16. The U.S. Geological Survey (USGS) gage 16213000 is located on the left bank about 300 ft upstream from Farrington Highway Bridge (Figure 3.10 and 3.12).

According to the USACE (1982) report, "prior to 1950 the Waikele Stream flowed toward the town of Waipahu … and caused severe flood damages to the sector around Arakawa Store and Waipahu Depot Road. The State of Hawaii in the early 1950's diverted flows toward its present

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day course by blocking the portion of Waikele Stream that previously flowed through the town of Waipahu. Following the 1954 flood disaster the City and County of Honolulu purchased a portion of the Waikele Stream floodplain area and declared the land to be a flood hazard area, thereby, incorporating floodplain zoning regulations and utilizing the area as a "park site". In 1960, the City and County of Honolulu constructed a flood control channel which consists of 2,600 feet of bank protection and a concrete-lined and earth trapezoidal channel from the stream's outlet to the vicinity of Waipahu Street. The Oahu Resource Conservation and Development Council (2006) report states that according to Tetra Tech (2005) "a berm was built in the 1960's along the railroad grade that provided flood protection from the Waikele Stream to the Waipahu town center." However, this berm (or levee) is not a certified structure and as such is not recognized by FEMA as providing flood protection to the Waipahu area. An historic map and historic air photographs of Waikele Stream and Kapakahi Stream are presented in Figures 2.2-2.6 in Section 2.

Prior to completion of the present flood control project in the 1960's, Waikele Stream frequently overflowed its banks causing considerable damage to the Waipahu Cultural Garden Park area, where trees were uprooted, plants destroyed, and a thick layer (2 to 6 ft) of fine sediment from Waikele Stream was deposited (USACE 1982). The most recent flooding along Waikele Stream occurred during a record high flood of December 11, 2008. During this event, the stream overtopped the east bank levee upstream from Farrington Highway and spilled into the Waipahu Cultural Garden Park. However, significant overland flows did not propagate beyond the garden and into the Waipahu downtown area. The USGS prepared a provisional peak flow estimate of 23,700 cfs for Waikele Stream upstream of the levied section for the December 11, 2008 flood of record (personal communication, Ron Rickman, USGS Honolulu, February, 2009). According to the USGS, this estimate represents a total flow including a smaller discharge of approximately 1,100 cfs that overtopped the levee for a short period of time. According to NHC estimates (see hydrology analysis in Chapter 4), this flood had a recurrence interval between 75 and 100 years. During the site inspection on January 15, 2009, high water marks (debris deposits and drift lines) were identified along the levee and in the vicinity of the USGS gage (Figures 3.6-3.8). Overtopping also occurred during this flood event at the Waipahu Street Bridge and the former railroad bridge located at the Energy Corridor. The December 11, 2008 flood damaged the USGS gage (Figure 3.10), eroded the east bank upstream from Farrington Highway (Figure 3.11), outflanked and damaged the former railroad bridge (Figure 3.16), and deposited large amounts of woody debris at many locations in the channel (Figure 3.2), on the floodplain (Figures 3.3 and 3.4), at bridge piers (Figure 3.14), and on the former railroad bridge (Figure 3.17). Thick layers of fine sediment were also deposited on the floodplain of Waikele Stream during the December 11, 2008 flood (Figure 3.5).

3.1.2. Kapakahi Stream

The entire length of Kapakahi Stream (approximately 1.2 miles) is located within a mapped FEMA floodplain in the study area. Kapakahi Stream originates in the vicinity of the Waipahu Cultural Garden Park north of Farrington Highway, just to the east of Waikele Stream, and flows into the West Loch, Pearl Harbor. Ground photographs of Kapakahi Stream are presented in Figures 3.19-3.28.

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Kapakahi Stream is fed by multiple local springs, a high water table and local runoff during storm events (Oahu Resource Conservation and Development Council, 2006). Kapakahi Stream initially flows east for about 0.2 miles parallel to Farrington Highway through a partially developed area (Figure 3.19). Once the stream reaches a shopping area next to Waipahu Depot Street, it flows into three 48-inch culverts (the fourth culvert is blocked) beneath the parking lot (Figure 3.20). It then reemerges at a single 54-inch culvert at the upstream face of Farrington Highway Bridge (Figure 3.21). The exact location of the transition from three culverts to one under the parking lot is not known. Downstream from the culvert, the stream flows south under the Farrington Highway Bridge (Figures 3.22 and 3.23) and through the straightened earthen channel along the west side of Waipahu Depot Street (Figure 3.25). It then flows under a former railroad bridge at the Energy Corridor (Figure 3.26), past 70-acre Pouhala Marsh (Figures 3.27 and 3.28), under a small footbridge, through thick stands of mangrove, and eventually discharges into the West Loch of Pearl Harbor. The width of the stream channel ranges from about 20 to 50 ft. In the area of Farrington Highway the channel of Kapakahi Stream is shallow and very silty. The floodplain area between the culverts and the railroad bridge is occupied by densely spaced residential and commercial buildings, which reduce the flow conveyance and potential storage capacity of the area. This section is comprised mostly of impervious surfaces with only a few grassed open fields, private yards, and undeveloped land. Downstream of the railroad bridge, the floodplain is very flat and undeveloped.

Significant changes have occurred to Kapakahi Stream area since the 1950's. According to Oahu Resource Conservation and Development Council (2006), who refer to the study by Tetra Tech (2005), "the original stream channel of Kapakahi Stream was either the main channel of the Waikele Stream or at least a secondary channel that conveyed high flows from Waikele Stream. Historically, during high flow events, the urban area of Waipahu would get flooded". The construction of the flood control project and the levee in the 1960's (discussed in Section 3.1.1) "reduced flows (high water) through the Kapakahi stream channel and prevented the Waikele from flooding Waipahu". The field inspection on January 15, 2009 did not reveal any signs of significant overland flooding along Kapakahi Stream during the December 11, 2008 flood event, however, NHC was unable to obtain any photographs of the Kapakahi channel area that were taken immediately following the December 2008 flood event.

3.1.3. Wailani Canal

The study reach of Wailani Canal is about 1.2 miles long and extends from Paiwa Street to its mouth at Middle Loch in Pearl Harbor. The canal system also includes the west-side and east-side drainage canals that run parallel to the Energy Corridor and connect to Wailani Canal. Ground photographs of Wailani Canal and the drainage canals in the Energy Corridor are shown in Figures 3.29-3.40.

Wailani Canal conveys flow from urban runoff in a southerly direction through the Waipahu Town area under the Farrington Highway Bridge (Figures 3.33 and 3.34) and under two footbridges at the Energy Corridor (Figures 3.37 and 3.38). The canal then turns sharply east and flows toward the Middle Loch. A footbridge and Waipio Point Access Road Bridge are located

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at the canal outlet to the loch. From Paiwa Street to about 400 ft downstream of the Energy Corridor, Wailani Canal is a concrete-lined rectangular channel with flood walls that are elevated up to 2-4 ft above the adjacent floodplain area (Figures 3.31, 3.32, 3.35, and 3.36). The canal converts to a shallow earthen channel in the downstream reach near its mouth. The width of the canal is between 40-50 ft in the Waipahu Town area and increases up to 100-150 ft below the Energy Corridor. The width of the side drainage canals in the Energy Corridor is generally within 30-40 ft. The floodplain area within the town of Waipahu consists of residential and commercial lands, with a few grassy areas and undeveloped land. Below the Energy Corridor, the floodplain is occupied by the Ted Makalena Golf Course. Dense vegetation grows along the canal banks in the reach near the mouth.

3.2. Channel Survey

Present day channel survey information along Waikele Stream, Kapakahi Stream, Wailani Canal, and drainage canals along the Energy Corridor was needed to support model development and hydraulic analyses for the HHCTCP. Following the site inspection of the study area, NHC developed a detailed channel survey program. The survey was conducted by ControlPoint Surveying, Inc. (ControlPoint) in the spring of 2009. The survey data included channel cross-section profiles between high banks (levees) at various locations upstream from Farrington Highway for Waikele Stream and along the entire study reaches of Kapakahi Stream and Wailani Canal. Also surveyed were details of the USGS weir on Waikele Stream upstream of Farrington Highway, geometry of bridges and culverts, flood walls along Wailani Canal, and the east bank levee profile along Waikele Stream upstream of Farrington Highway. Altogether, 8 cross-sections were surveyed along Waikele Stream upstream of Farrington Highway, 20 cross-sections along Kapakahi Stream, and 23 cross-sections along Wailani Canal (including the west- and east-side drainage canals in the Energy Corridor). The distance between the surveyed cross-sections depended on the complexity of the channel topography and ranged from approximately 10 to130 ft in the vicinity of hydraulic structures (bridges, USGS weir on Waikele Stream, and Kapakahi Stream culvert) to approximately 500 to 1,500 ft in relatively uniform channel reaches. Shorter intervals between cross-sections were specified at the hydraulic structures for the computation of energy losses due to the structures. Bridge and culvert surveys included measurement of dimensions and elevations of the culvert and 3 bridges on Kapakahi Stream and 3 bridges on Wailani Canal. Dimensions of the other bridges were available from existing FEMA effective models of the streams.

3.3. Topographic Data

Bare ground surface topography was needed to support development of the 2-d model of the study area. Topographic data used in this study included 1-ft contour interval Light Detection and Ranging Data (LiDAR)-based topography data within the HHCTCP corridor obtained by John Chance Land Surveys, Inc. (JCLS) in 2007 (official data for the project) and 5 ft by 5 ft grid FEMA LiDAR data developed in 2006 for the FEMA Hurricane Study and covering the entire Oahu coastline including Waipahu Town area. These topographic data sets were provided to the project team by the City and County of Honolulu.

A considerable effort was made by NHC working with ControlPoint to ensure consistency and compatibility (vertical and horizontal) of the different topographic datasets used in this study. The 2007 project corridor topography data and 2006 FEMA LiDAR data were compared in ArcGIS (geographic information system). A triangulated irregular network (TIN) was generated from the 2007 1-ft contours. The TIN was then converted to a 5 ft by 5 ft raster grid, where each grid cell was snapped and aligned to the 2006 FEMA LiDAR grid cells. A direct comparison of the two topographic data sets was made for the overlapping area by subtracting the 2006 grid elevations from the 2007 grid elevations. The comparison indicated a close agreement (within one foot) of ground elevations between these topographic data sets for 92% of the overlapping area (the mean difference in elevations was 0.4 ft). The elevation differences higher than one foot were detected along the elevated middle portion of Farrington Highway and along some stream channels, which is likely due to errors in detection of spatial locations and elevations of vertical or near-vertical topographic features/structures. A comparison was also made in ArcGIS between the 2006 FEMA LiDAR data and point topographic data surveyed along Waikele Stream, Kapakahi Stream, and Wailani Canal by ControlPoint in the spring of 2009. The difference in the ground elevations between these data sets was within one foot, which indicated the overall reasonableness and reliability of the topography data used in this study.

For consistency with the project drawings, the official project topography data were used for the HHCTCP corridor, while the FEMA LiDAR data were used for the areas outside of the project corridor. For use in this study, NHC developed a modified topographic data set in which the 2006 FEMA LiDAR data that is located within the project corridor were replaced with the 2007 project topography data. The original FEMA LiDAR data were screened to remove artificial "walls" (linear spikes in elevation) and "holes" (missing grid cells) in some areas along edges of LiDAR tiles. Ground elevations for these misrepresented areas were determined by interpolation from the surrounding topography. Since the LiDAR data only represent dry land surfaces, NHC generated a set of generalized bathymetric contours for the bay area and inserted them into the modified topographic data set for subsequent use in the 2-d model.

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Figure 3.1. Waipahu Street Bridge on Waikele Stream. View upstream. Photo of January 15, 2009.

Figure 3.2. Waikele Stream below Waipahu Street Bridge. View downstream. Note channel scour and debris deposited after December 11, 2008 flood. Photo of January 15, 2009.

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Figure 3.3. Debris deposited on west floodplain of Waikele Stream below Waipahu Street Bridge after December 11, 2008 flood. Photo of January 15, 2009.

Figure 3.4. East floodplain of Waikele Stream above Farrington Highway with debris from December 11, 2008 flood. Flow right to left. Photo of January 15, 2009.

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Figure 3.5. Sediment deposited on east floodplain of Waikele Stream upstream of Farrington Highway after December 11, 2008 flood. Photo of January 15, 2009.

Figure 3.6. Debris deposited along east bank levee of Waikele Stream upstream of Farrington Highway after December 11, 2008 flood. Engineer is pointing to approximate depth of flow that overtopped the east levee into the Waipahu Cultural Garden Park during this event. Photo of January 15, 2009.

Figure 3.7. High Water drift line deposited along east bank levee of Waikele Stream upstream of Farrington Highway after December 11, 2008 flood. Photo of January 15, 2009.

Figure 3.8. Debris line deposited along east bank levee of Waikele Stream upstream of Farrington Highway after December 11, 2008 flood. Photo of January 15, 2009.

Figure 3.9. Waikele Stream upstream of USGS gage. View upstream. Photo of January 15, 2009.

Figure 3.10. USGS gage 16213000 on Waikele Stream upstream of Farrington Highway. Flow right to left. Note damage caused by December 11, 2008 flood. Photo of January 15, 2009.

Figure 3.11. Eroded east bank of Waikele Stream at USGS gage. View upstream. Photo of January 15, 2009.

Figure 3.12. USGS weir on Waikele Stream upstream of Farrington Highway. Bank erosion on west bank. View upstream. Photo of January 15, 2009.

Figure 3.13. Farrington Highway Bridges on Waikele Stream. View downstream. Photo of January 15, 2009.

Figure 3.14. Debris from December 11, 2008 flood deposited on eastbound Farrington Highway Bridge piers in Waikele Stream. View downstream. Photo of January 15, 2009.

Figure 3.15. Down stream portion of concrete-lined Waikele Stream flood control channel with footbridge. View upstream. Photo of January 15, 2009.

Figure 3.16. Abandoned railroad bridge on Waikele Stream at Energy Corridor. View downstream. Photo of January 15, 2009. Note damage caused by December 11, 2008 flood.

Figure 3.17. Debris from December 11, 2008 flood deposited on abandoned railroad bridge on Waikele Stream at Energy Corridor. Flow left to right. Photo of January 15, 2009.

Figure 3.18. Mangrove forest along Waikele Stream below Energy Corridor. View downstream. Photo of January 15, 2009.

Figure 3.19. Upstream reach of Kapakahi Stream north of Farrington Highway. View upstream toward Waipahu Cultural Garden Park. Photo of January 15, 2009.

Figure 3.20. Inlet culverts on Kapakahi Stream upstream of Farrington Highway. View downstream. Photo of January 15, 2009.

Figure 3.21. Outlet culvert on Kapakahi Stream just upstream of Farrington Highway. View upstream. Photo of January 15, 2009.

Figure 3.22. Farrington Highway Bridge on Kapakahi Stream. View downstream. Photo of January 15, 2009.

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Figure 3.23. Kapakahi Stream under Farrington Highway Bridge. View downstream. Photo of January 15, 2009.

Figure 3.24. Farrington Highway at Waipahu Depot Street. View east from the Farrington Highway Bridge on Kapakahi Stream. Photo of January 15, 2009.

Figure 3.25. Kapakahi Stream downstream of Farrington Highway. View upstream. Photo of January 15, 2009.

Figure 3.26. Abandoned railroad bridge on Kapakahi Stream at Energy Corridor. Flow right to left. Photo of January 15, 2009.

Figure 3.27. Kapakahi Stream downstream of Energy Corridor. View downstream. Note Pouhala Marsh on right floodplain. Photo of January 15, 2009.

Figure 3.28. Pouhala Marsh on right floodplain of Kapakahi Stream south of Energy Corridor. Photo of January 15, 2009.

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Figure 3.29. Wailani Canal upstream of Paiwa Street. View upstream toward unlined section. Photo of January 15, 2009.

Figure 3.30. Paiwa Street Bridge on Wailani Canal. View downstream. Photo of January 15, 2009.

Figure 3.31. Wailani Canal downstream of Paiwa Street Bridge. View downstream. Photo of January 15, 2009.

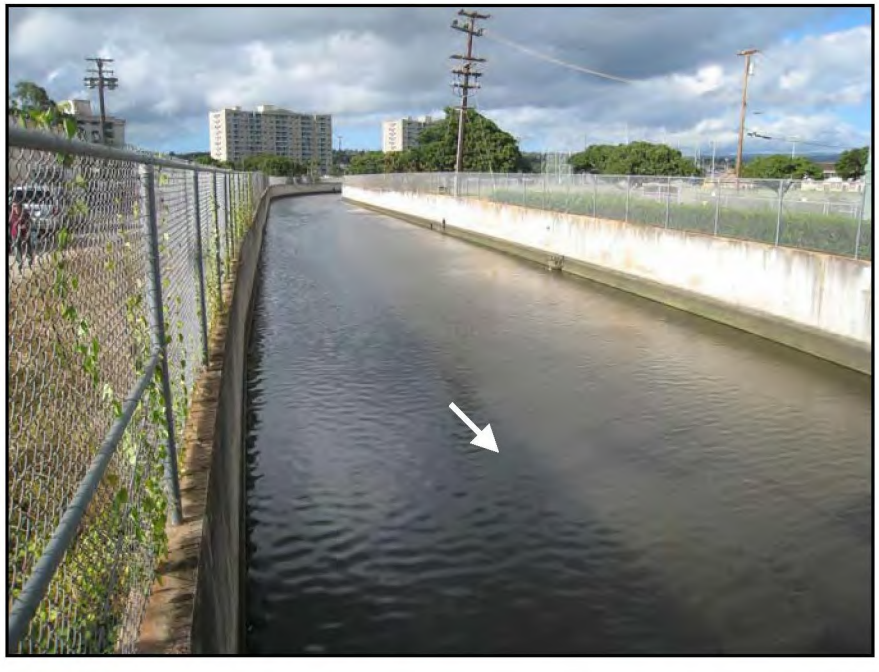

Figure 3.32. Wailani Canal upstream of Farrington Highway. View upstream. Photo of January 15, 2009.

Figure 3.33. Farrington Highway Bridge on Wailani Canal. View downstream. Photo of January 15, 2009.

Figure 3.34. Wailani Canal below Farrington Highway Bridge. View upstream. Photo of January 15, 2009.

Figure 3.35. Wailani Canal downstream of Farrington Highway. View downstream from Farrington Highway Bridge. Photo of January 15, 2009.

Figure 3.36. Wailani Canal above Energy Corridor. View upstream from footbridge. Note dark high tide line on canal walls. Photo of January 15, 2009.

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Figure 3.37. Wailani Canal at footbridge in Energy Corridor. Flow left to right. View toward east-side drainage canal. Photo of January 15, 2009.

Figure 3.38. Wailani Canal below Energy Corridor. View downstream from footbridge. Photo of January 15, 2009.

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Figure 3.39. Drainage canal in Energy Corridor (west-side tributary to Wailani Canal). View east toward Wailani Canal. Photo of January 15, 2009.

Figure 3.40. Middle portion of west-side drainage canal in Energy Corridor. View east toward Wailani Canal. Photo of January 15, 2009.

4. HYDROLOGY

NHC developed up-to-date hydrologic data to support hydraulic analysis of potential impacts of the proposed HHCTCP facilities on 100-year flood levels in the Waipahu Town area. The effective Flood Insurance Study (FIS) for the City and County of Honolulu (FEMA 2004) was originally published on November 20, 2000 and subsequently revised and re-issued on September 30, 2004. The FIS documents hydrologic and hydraulic analyses and provides maps for dozens of streams on the island of Oahu. These analyses utilized a variety of methods and data sources to compute flood quantiles and map Base Flood Elevations (BFEs). The following sections document a review and updating of the FIS peak flow quantiles as well as the development of flood hydrographs necessary to support detailed, unsteady hydraulic modeling that quantifies the effects of proposed HHCTCP facilities in the study area.

4.1. Review of FEMA Effective FIS Hydrologic Analysis

The proposed HHCTCP alignment is immediately adjacent to Farrington Highway in the vicinity of the Waikele Stream crossing. The planned Waipahu Transit Station is located within the existing, mapped FEMA floodplain in this area where flow from Waikele Stream commingles with the flow from smaller flooding sources including Kapakahi Stream and Wailani Canal during large flood events.

The effective FIS for the City and County of Honolulu (FEMA 2004), and all other previous hydraulic studies of Waikele Stream, applied 1-d, steady state hydraulic modeling methods to determine flood profiles and map 100-year inundation limits. Typically, the hydrologic input to this type of hydraulic analysis requires the estimation of 10-year, 50-year, 100-year, and 500-year flood quantiles (peak flows) derived from a frequency curve that has been fit to instantaneous peak annual discharge data, or from regional regression equations.

The effective FIS (FEMA 2004) cites the application of methods specified by Bulletin 17B (WRC 1981) to fit a Log-Pearson III distribution to 31 years of USGS data at gage 16213000 from WY (water year) 1952 through WY 1983. An attempt was made to replicate the peak flow quantiles reported in the effective FIS using Bulletin 17B procedures coded in the HEC-SSP Version 1.0 computer program. The replication applied a weighted skew, a regional skew of -0.05, and a mean square error (MSE) of 0.302 on the regional skew. While the results shown in Table 4.1 do not match the effective FIS exactly, they are within 5% except for the 500-year which is 6.2% lower in NHC's analysis using the HEC-SSP program. The origin of these small discrepancies is not known; however, they are not considered significant.

4.2. Re-Analysis of Waikele Stream Frequency Curve

NHC utilized the additional, post-WY 1983 data available at gage 16213000 to re-fit a Log-Pearson III frequency curve according to Bulletin 17B procedures using the HEC-SSP program. This analysis includes a very recent flood of record which occurred on December 11, 2008. The

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USGS has provided a provisional peak flow estimate of 23,700 cfs (personal communication, Ron Rickman, USGS Honolulu, February, 2009) for this flood of record. According to the USGS personnel, this estimate represents a total "at latitude" flow including a smaller discharge of approximately 1,100 cfs that overtopped the left bank levee upstream of the USGS gage near the peak of the event. Figure 4.1 shows an estimated hydrograph for the December 11, 2008 flood of record.

There is a possibility that a few of the other large peaks in the gage record may be biased low due to overtopping of this levee. Unfortunately, details of the history of the left bank levee profile are somewhat uncertain. Apparently, the current channel alignment was engineered to direct flood flows away from Waipahu Town in the 1950's and the current concrete channel and levee generally date from 1960 (USACE 1982). Additional improvements or repairs may have been made to the levee since this time, but details are lacking. The extent of the bias on the USGS peak discharge record is not precisely known, but is considered to be small given that less than 5% of the peak was estimated to have "escaped" the gage during the recent flood of record. Therefore, the re-computed flood frequency curve (shown in Table 4.2) is judged to be a good approximation of the annual exceedance probability of "at latitude" peak discharges.

As shown in Table 4.1, the re-computed flood frequency for Waikele Stream is remarkably similar to the frequency curve reported in the effective FIS, in spite of the fact that the re-computed curve is based on nearly twice as long a discharge record. While stability in the flood frequency curve at a site is what is hoped for over time, it rarely works out with this degree of precision in practice. The close match between the two curves is probably due in part to the presence of the recent December 11, 2008 peak of record in the re-analysis. This extreme peak compensates for the effect of increased record length. If WY 2009 were excluded, the recomputed 100-year peak would have been 18% less than the effective FIS value. Figure 4.2 shows the fit of the current full record to the Log-Pearson III curve.

4.3. Use of Hydrology for Hydraulic Modeling of Waikele Stream Floodplain

As discussed in Chapter 1, the current FEMA effective models are outdated, very limited, and are not capable of addressing spatially complex floodplain hydraulics in the Waipahu study area. Therefore, an unsteady, 2-d hydraulic modeling approach was proposed to analyze water levels and inundation extents during the 100-year flood event in the Waipahu area because such an approach can resolve the complex flow patterns and the time variant overflows and interactions of storage areas which affect flood levels. This hydraulic modeling approach requires the input of hydrographs rather than static peak discharges to the hydraulic model. This modeling approach is far more sophisticated than previous floodplain modeling which relied on 1-d, steady flow modeling. The purpose of the 2-d approach is to more accurately represent and evaluate flow patterns and flood depths resulting from channel overflows and multiple flow pathways in the floodplain both upstream and downstream of Farrington Highway. In this area, there is a considerable volume of floodplain storage that has the capacity to store flood water and reduce (attenuate) flood levels. A steady flow modeling approach effectively ignores the possibility that floodplain storage may not be fully filled to capacity by a passing flood. Effectively, steady flow analysis assumes that the instantaneous peak discharge persists for a sufficient time to equilibrate

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and fully occupy all hydraulically connected areas with ground levels below the maximum water surface in the main channel. In reality, the 100-yr peak flow may not occur for a long enough period to overflow and fill all of these storage areas to that level. Steady flow analysis is conservative because it usually predicts lateral flooding extents (assuming the same level of topographic detail in the models) that are at least as widespread as those predicted by an unsteady model. Where floodplain storage is relatively small, there is little advantage to a more arduous unsteady flow modeling approach; however, where the flood volume is limited and floodplain storage is potentially large, an unsteady modeling approach is justified and may be necessary to avoid computing and mapping an unrealistically large inundation area or flow depths. Waikele Stream near Waipahu Town appears to be such a case.

In addition to Waikele Stream, there are two additional, smaller sources of flood water – local urban runoff that drains into Kapakahi Stream and Wailani Canal. Estimated 100-year discharges for these sources are shown in Table 4.3. Inflow hydrographs to the hydraulic model for each of these sources were scaled based on these discharges.

4.3.1. Hydrograph Shape

A good estimation of realistic and appropriate hydrographs is a significant determinant in the accuracy and utility of the 2-d, unsteady hydraulic analysis. Water levels predicted by the model depend not only on the instantaneous peaks which are derived from flood frequency analysis, but also the volume of water contained in the flood hydrograph and how that water is distributed onto the floodplain with time during a flood event. Proposed hydrographs for this study were estimated by examining the entire USGS record of flood hydrographs which extends back to the mid-1980's. The December 11, 2008 event (Figure 4.1) represents a prime example of such a flood hydrograph. Other hydrographs recorded by the USGS at 30-minute time steps were also examined to determine hydrograph shapes and whether it was necessary to test the sensitivity of hydraulic modeling results to alternative hydrograph shapes.

4.3.2. Multiple Hydrograph Timing

As a conservative approximation, it is assumed that peaks from all three flood water sources will be concurrent. This is considered conservative, because it is likely that flood peaks from the two smaller source areas would occur prior to the peak from the larger Waikele watershed.

4.3.3. 100-Year Design Hydrograph for Waikele Stream

A significant challenge to the development of an unsteady hydraulic model is the estimation of unsteady flow (hydrograph) inputs to the study reach that must be both realistic and suitably conservative in representing conditions for a given annual exceedance probability. The peak of the hydrograph is determined through flood frequency analysis similar to the steady flow approach; however, the remainder of the hydrograph must also be suitable to reflect 100-year flood event time-variant characteristics. This means that high flows must not persist for too short

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a time period resulting in underestimation of flooding extents in overbank storage areas, or too long a time resulting in flooding extents that are unrealistically large.

There are various approaches to constructing inflow hydrographs for unsteady hydraulic models including application of flow duration analysis. In this approach, 100-year instantaneous peak flows are assumed to occur concurrently with 100-year hourly, 2-hourly, 4-hourly, 8-hourly, daily, 2-day, etc., mean flows. This is a so-called "balanced hydrograph" approach. The shape of the hydrograph constructed with the durational statistics is often influenced by available flood hydrograph data. Typical shapes are "advanced" in which the flow rises quickly and recedes more slowly, "centered" in which the hydrograph rises and falls at approximately the same rate, and "retarded" or "delayed" in which the rise to the peak takes longer than the recession.

USGS records for Waikele Stream include a considerable amount of detailed hydrograph data. These are typically discharges recorded at 15-minute or 30-minute intervals. The record of such detailed data for USGS gage 16213000 (Waikele Stream at Waipahu) extends back to WY 1987. For purposes of determining a suitably conservative 100-year hydrograph, all flood hydrographs in this record with a peak flow greater than 5,000 cfs (between a 2-year and 5-year flood) were examined. Figure 4.3 presents the 8 flood records meeting this criterion in non-dimensional form. All discharges within each hydrograph with values greater than the 10% of the hydrograph peak were normalized by the peak and plotted along a non-dimensional time axis. The time axis was normalized by the period over which the 10% threshold discharge was exceeded. The legend includes the normalizing time base and peak flow values for each hydrograph.

From the hydrographs shown in Figure 4.3, it can be seen that the larger floods in Waikele Stream typically rise and fall fairly rapidly and are "advanced" in shape. As shown on the legend, all floods recede to less than 10% of their peak value in considerably less than a day. Further, among all of the 8 floods, the longest durations of flows exceeding 50%, 75%, and 90% of the peak are exhibited by the December 11, 2008 event at 7.0, 3.2, and 1.8 hours respectively. Therefore, for this study, a conservative 100-year hydrograph was developed by a simple scaling of the December 11, 2008 flood hydrograph using the estimated 100-year peak quantile (flow) for each flooding source as shown in Table 4.3.

Figure 4.4 shows all three inflow hydrographs that were developed for use in the 2-d, unsteady hydraulic model for Waipahu. In the absence of measured flow data for Kapakahi Stream and Wailani Canal, the hydrographs for these streams are assumed to have the same temporal pattern (shape) as the main flow coming from Waikele Stream.

4.3.4. Design Hydrograph Ordinates

Table 4.4 provides dimensionless flow values for the proposed 100-year design hydrographs. In this table, time is in physical units of hours. Available records indicate that all floods on Waikele Stream of any consequence for flood analysis will rise and fall in much less than the 15 hour period provided in the table. These are very flashy, short duration events. To obtain flow hydrographs, the dimensionless values in the table must be multiplied by the instantaneous 100-year peak flow associated with each flooding source as shown in the Table 4.2.

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Table 4.1. Waikele Stream flood quantile estimates.

Study	Location	Drainage area (square miles)	10-yr (cfs)	50-yr (cfs)	100-yr (cfs)	500-yr (cfs)	Method
Effective FIS	"Below H-1 Freeway"	44.9	10,450	20,700	26,000	40,800	Bulletin 17B, 31 years of record, one missing year.
HHCTCP (NHC), Effective FIS verification	USGS 16213000, Waikele Stream at Waipahu (upstream of Farrington Highway)	45.7	10,537	20,030	24,921	38,282	Bulletin 17B, 31 years of record. Weighted Skew, WYs 1952-1983 (one missing year), HEC-SSP V1.0 software
Change from Effective FIS			+0.8%	-3.2%	-4.2%	-6.2%	
HHCTCP (NHC), revised using additional data	"At latitude" (upstream of Farrington Highway)	45.7	10,304	20,054	25,306	40,367	Bulletin 17B, 56 years of record. Weighted Skew, WYs 1952-2009 (two missing years), HEC-SSP V1.0 software
Change from Effective FIS			-1.4%	-3.1%	-2.7%	-1.1%	

Table 4.2. Re-computed frequency curve for Waikele Stream (total "at latitude flow").

Computed flow (cfs)	Annual chance of exceedance (%)	Return period (years)
40,367	0.2	500
31,275	0.5	200
25,306	1	100
20,054	2	50
14,113	5	20
10,304	10	10
7,018	20	5
3,334	50	2
1,564	80	1.25
1,048	90	1.11
751	95	1.05
399	99	1.01

Table 4.3. 100-year peak flows for three flooding sources in study area.

Basin	Drainage area (square miles)	100-yr (cfs)	Method
Waikele Stream (total "at latitude" flow)	45.7	25,306	Bulletin 17B, 57 years of record
Kapakahi (local drainage)	0.329	847	USGS Regression (Wong 1994)
Wailani Canal	1.57	2,681	Effective FIS (FEMA 2004)

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Table 4.4. 100-year design hydrograph pattern.

Time (hrs)	Q_i/Q_p	Time (hrs)	Q_i/Q_p	Time (hrs)	Q_i/Q_p
0.00	0.001	5.95	0.536	11.25	0.073
0.25	0.003	6.00	0.506	11.50	0.070
0.50	0.015	6.25	0.419	11.75	0.062
0.75	0.015	6.50	0.377	12.00	0.056
1.00	0.068	6.75	0.363	12.25	0.054
1.25	0.131	7.00	0.347	12.50	0.050
1.50	0.186	7.25	0.323	12.75	0.046
1.75	0.198	7.50	0.314	13.00	0.043
2.00	0.216	7.75	0.276	13.25	0.041
2.25	0.226	8.00	0.255	13.25	0.041
2.50	0.281	8.25	0.239	13.50	0.038
2.75	0.393	8.50	0.192	13.75	0.036
3.00	0.565	8.75	0.184	14.00	0.035
3.25	0.823	9.00	0.157	14.25	0.033
3.55	0.966	9.25	0.152	14.50	0.031
3.85	1.000	9.50	0.141	14.75	0.029
4.15	0.970	9.75	0.121	15.00	0.030
4.45	0.916	10.00	0.114	15.25	0.030
4.75	0.844	10.25	0.100	15.5	0.028
5.05	0.781	10.50	0.091	15.75	0.027
5.35	0.705	10.75	0.085	16.00	0.026
5.65	0.620	11.00	0.078		

Q_i = flow (cfs), Q_p = peak flow (cfs)

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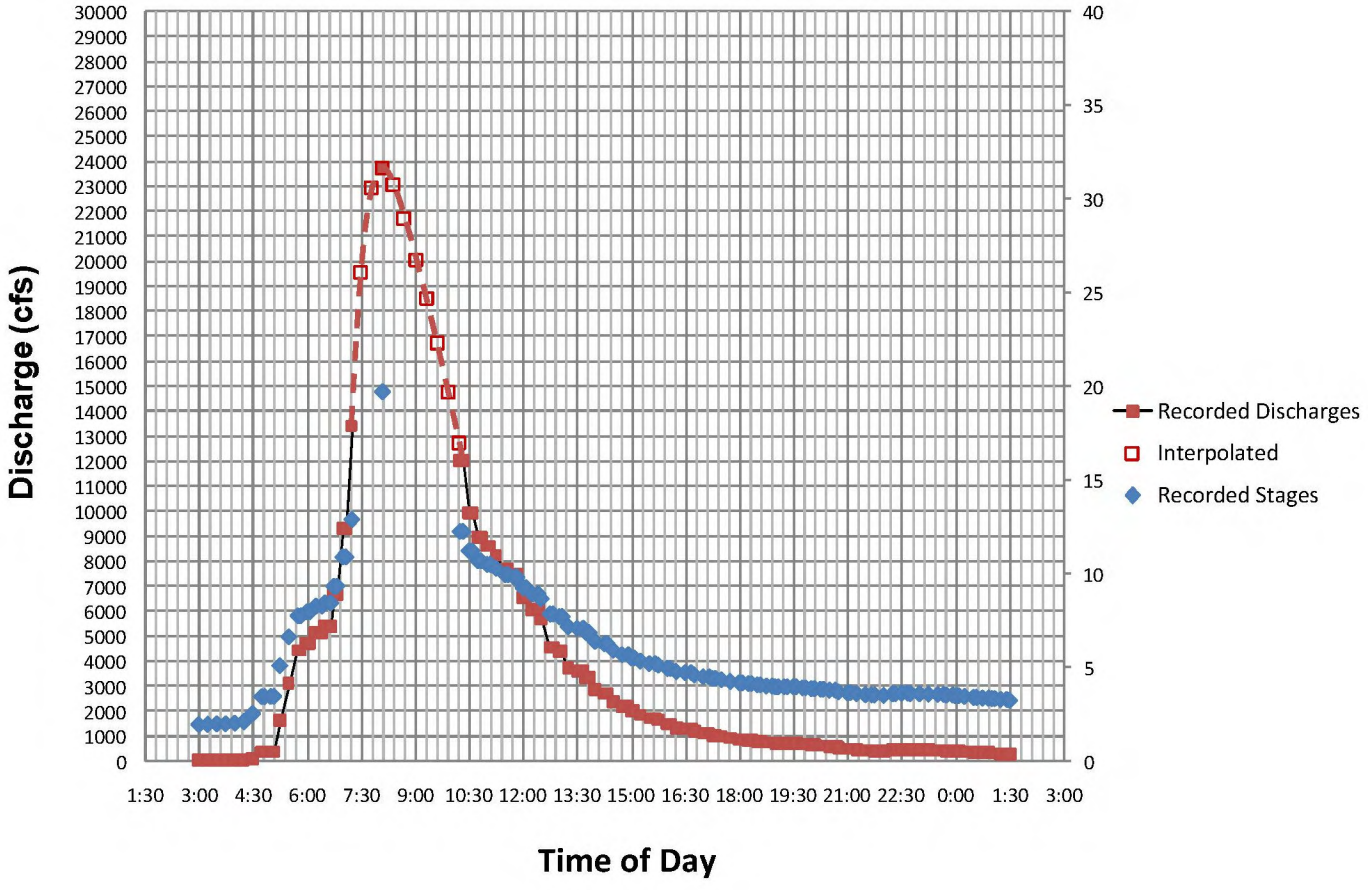

Figure 4.1. Estimated "at latitude" flood hydrograph for event of December 11, 2008, Waikele Stream at Waipahu.

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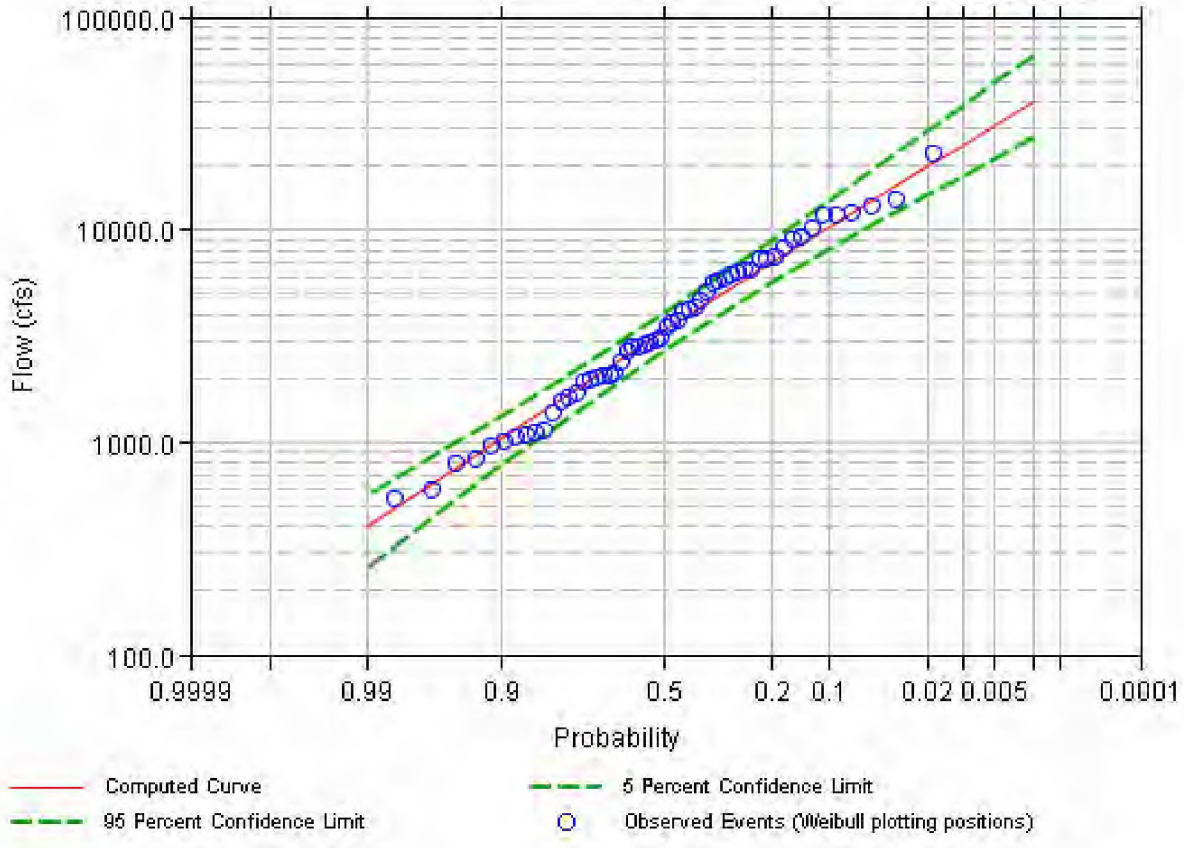

Figure 4.2. "At latitude" Log-Peason III frequency curve, Waikele Stream at Waipahu (USGS gage 16213000)

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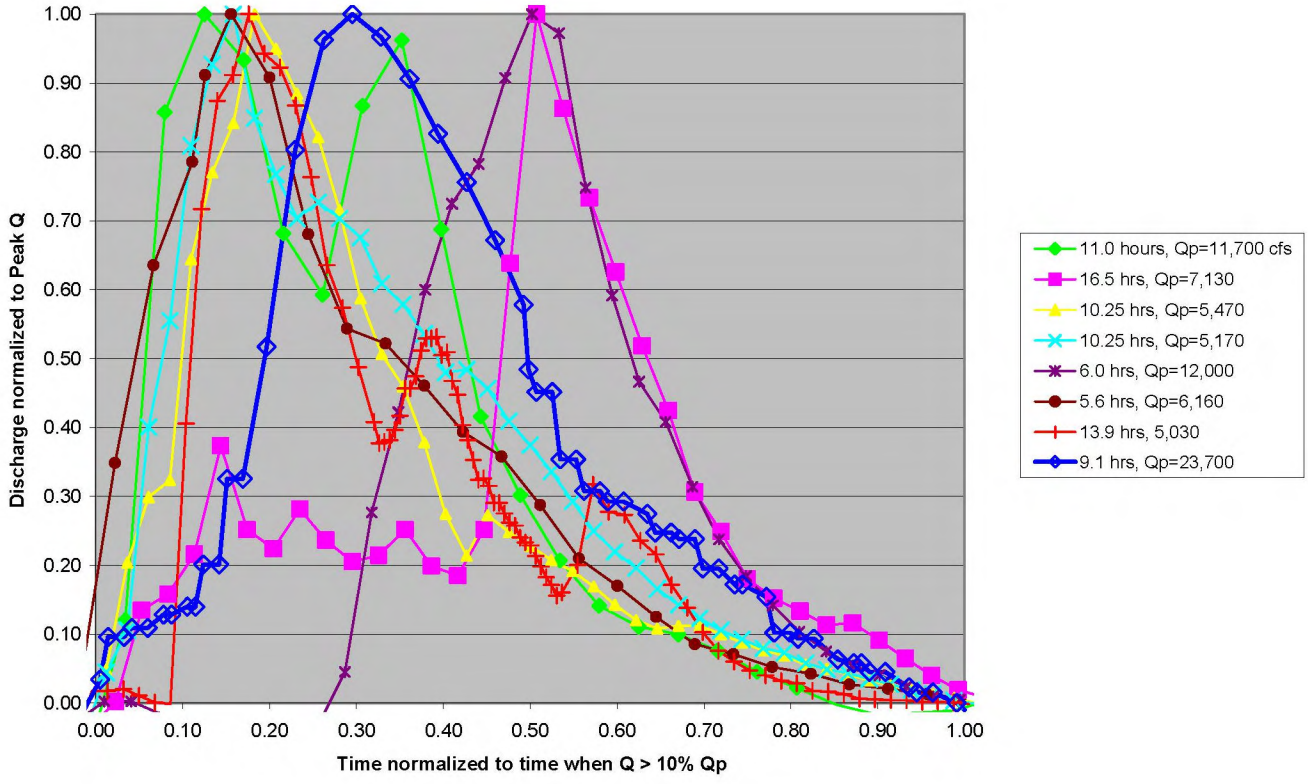

Figure 4.3. Dimensionless recorded flood hydrographs, Waikele Stream at Waipahu (USGS 16213000)

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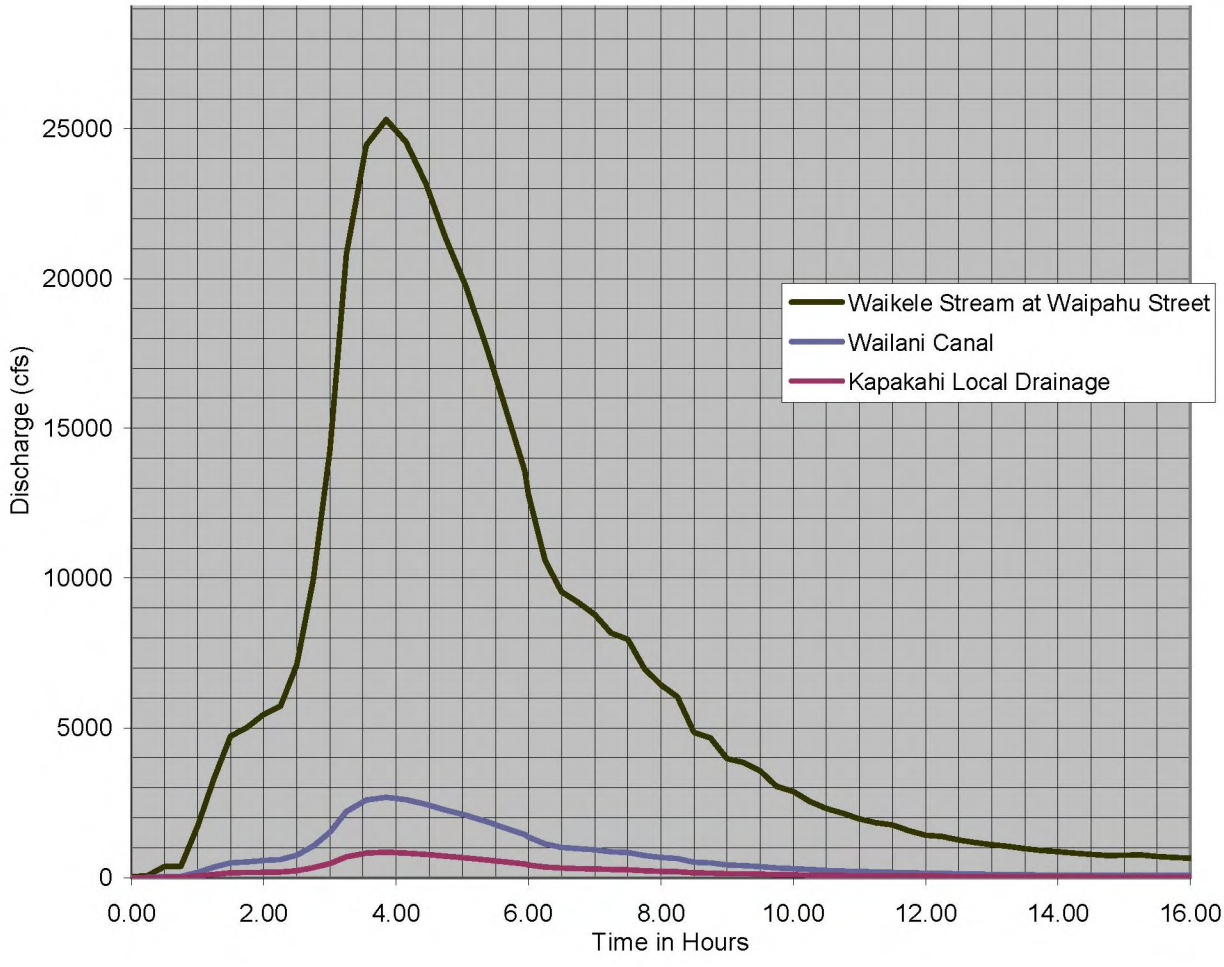

Figure 4.4. 100-year inflow hydrographs for 2-d, unsteady hydraulic model.

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5. ONE-DIMENSIONAL HYDRAULIC MODELING

NHC developed up-to-date one-dimensional (1-d) HEC-RAS models for Waikele Stream, Kapakahi Stream, and Wailani Canal. The HEC-RAS models were needed to support the development and application of a two-dimensional (2-d) model of the Waipahu floodplain area. The HEC-RAS models were developed using channel survey data collected along the study streams by ControlPoint Surveying, Inc. (ControlPoint) in the spring of 2009. NHC used the HEC-RAS models to determine water levels during the 100-year flood event and to determine breakout flows that leave Waikele Stream over the east bank levee at the Waipahu Cultural Garden Park upstream from Farrington Highway. The models were also used to determine flow characteristics (depth-flow rating tables) through culverts and bridge openings on all three streams. Two flooding scenarios were evaluated: (1) with the existing east bank levee in place on Waikele Stream at the Waipahu Cultural Garden Park upstream from Farrington Highway (to evaluate the most likely flooding scenario), and (2) without the levee (to comply with FEMA's regulations). The levee along Waikele Stream as well as the flood walls along Wailani Canal are not "certified" structures and as such they are not recognized by FEMA as providing flood protection and were therefore omitted from FEMA's effective models. Results from the 1-d models were used to determine inflow hydrographs and rating tables for culverts and bridges in the 2-d model of the study area.

The following sections describe the development of the HEC-RAS models, derivation of input data used in the model, key assumptions, model parameters, modeling scenarios, and results from the computer simulations. All elevations in the report are given in feet, HILOCAL.

5.1. Description of Computer Model HEC-RAS

The Hydrologic Engineering Center's River Analysis System (HEC-RAS) is a computer program designed to perform one-dimensional (1-d) hydraulic computations for a network of open channels (USACE 2009). HEC-RAS has the capability of performing simulations under either steady or unsteady flow conditions.

The steady flow component of HEC-RAS is intended for computing steady gradually varied flow and is capable of modeling subcritical, supercritical, and mixed flow regime water surface profiles. The basic computational procedure is based on the solution of the 1-d energy equation for gradually varied sub- and supercritical flows. Water surface profiles are determined from one cross section to the next by solving the energy equation (Manning's equation) with an iterative procedure referred to as the standard step method. The momentum equation is utilized in situations where the water surface profile is rapidly varied. These situations include mixed flow regime computations (i.e., hydraulic jumps), hydraulics of bridges, and evaluating profiles at river confluences (stream junctions). The effects of various obstructions such as bridges, culverts, weirs, and structures in the floodplain may be considered in the computations. The steady flow solver is designed for and frequently applied to floodplain assessments and flood insurance studies. The model can be used to evaluate water surface elevation impacts of

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floodway encroachments and for assessing the change in water surface profiles due to channel improvements and levees.

The unsteady flow component of HEC-RAS is intended for simulating time variant flows (i.e. flow and stage hydrographs). The unsteady hydraulic model is based on the solution of the Saint-Venant continuity and momentum equations for 1-d flow. The hydraulic calculations for cross-sections, bridges, culverts, weirs, and other hydraulic structures that were developed for the steady flow component are incorporated into the unsteady flow module. The unsteady component of HEC-RAS also provides routines for modeling floodplain storage areas (in which water can enter or leave storage areas depending on local hydraulic conditions) and routing hydraulic linkages between main channel conveyance and floodplain storage.

In HEC-RAS, a river system is comprised by one or more stream reaches. Each reach is defined by a series of cross-sections. Floodplain storage areas are defined by an elevation-volume relationship. Storage areas can be connected to reaches and other storage areas by weirs, culverts, etc. Lateral weirs can be used to model levees along stream reaches. A HEC-RAS simulation represents a hydrologic event that is defined by upstream and downstream hydraulic boundary conditions.

Data input requirements for the HEC-RAS model include: (1) river system geometric data, including channel cross-sections, reach lengths, specification of ineffective flow areas, channel and floodplain roughness, contraction and expansion losses due to changes in cross-sections, storage areas information, and hydraulic structures data; and (2) hydraulic information, including flow rate (steady or unsteady), flow regime (subcritical, supercritical, or mixed), boundary conditions (steady or time variant flows and/or water surface elevations), and initial flow and stage conditions.

The HEC-RAS model is based on the following assumptions: (1) the channel is sufficiently straight and uniform in the reach so that the flow may be physically represented by a 1-d flow model; (2) the flow is normal to each cross-section; (3) the water surface elevation and velocity vary only in the streamwise direction; (4) the water surface is horizontal in each cross-section; (5) the velocity is uniformly distributed over the cross-section; (6) transverse effects are not explicitly considered; (7) the pressure distribution is hydrostatic; (8) the river channel slope is small (less than 10 %); and (9) there is always water in each cross-section.

5.2. Limitations of FEMA's Effective Model

The FEMA flood insurance rate map (FIRM) for the Waipahu Town area (see Figure 2.7 in Chapter 2) is based on results from FEMA's effective models developed for Waikele Stream (separate models upstream and downstream from Farrington Highway), Kapakahi Stream (includes Wailani Canal upstream from the Energy Corridor), and Wailani Canal (downstream from the Energy Corridor). FEMA's effective models used a limited number of cross-sections to depict Kapakahi Stream, Wailani Canal, and their interaction with the greater Waipahu floodplain. Cross sections in FEMA's effective models are quite simplified and do not include planimetric details for buildings, homes, roads or other urban features located in the floodplain.

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Peak flows used in the FEMA (2004) study are listed in Table 5.1. Methods used by FEMA to estimate split flows on the Waikele Stream upstream from Farrington Highway are not documented in their reporting and are therefore unknown.

NHC ran the existing FEMA's effective models and attempted to assess potential impacts of proposed HHCTCP features on water levels. However, because of the limitations of the FEMA effective models, they do not allow accurate assessment of the potential impacts of the proposed station and guideway support columns on water levels within the Waipahu Town area. Therefore, NHC determined that more detailed hydraulic models were needed to understand local flow conditions and determine how proposed project features may affect water levels in the Waipahu floodplain. These new models would also be used to help PB develop reliable project design alternatives that comply with FEMA and DPP floodplain requirements.

5.3. HEC-RAS Model Development

Steady and unsteady HEC-RAS models (version 4.0) of Waikele Stream, Kapakahi Stream, and Wailani Canal were developed to simulate in-channel flood flow and breakout flow characteristics. The following sections describe in detail the methods, approximations, and assumptions used to develop the HEC-RAS models for the study streams.

5.3.1. Channel Geometry

The Waikele Stream model includes a 1.1 mile long reach of the stream that extends from just upstream of the Waipahu Cultural Garden Park to the outlet into West Loch in Pearl Harbor. The Kapakahi Stream model includes the entire 1.2 mile long channel of the stream from the Waipahu Cultural Garden Park to West Loch. The Wailani Canal model includes a 1.2 mile long reach of the canal between Paiwa Street and its mouth, including west- and east-side drainage canals running along the Energy Corridor. The model layouts are shown in Figures 5.1-5.3.

Most model geometry data were obtained from the channel surveys conducted in the study area by ControlPoint during the spring of 2009. For Waikele Stream downstream of the USGS gage, channel and floodplain geometry were obtained from the HEC-RAS model developed by Park Engineering (Park) in 2004. Park's model together with Park's accompanying report were provided to the project team by the City and County of Honolulu. According to the Park Engineering (2004) report, their model was developed using their own channel survey data collected between Farrington Highway and the H-1 Freeway in June 2002. Cross-section data along the lower reach of Waikele Stream in Park's model were obtained from the FEMA (2004) flood insurance study.

To increase computational stability of the HEC-RAS models, additional cross-sections were added to complex stream reaches using an interpolation tool available in HEC-RAS. The minimum distance between the interpolated cross-sections was set to 100 ft for the steep reach of Kapakahi Stream upstream of Farrington Highway and to 300 ft for the other stream reaches. To account for channel curvature in meandering reaches, left and right overbank distances between

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cross-sections were measured from topographic maps along the anticipated path of the center of mass of the overbank flow.

A contraction coefficient of 0.1 and expansion coefficient of 0.3 was specified in the models for gradually varying cross-sections in accordance with the HEC-RAS Hydraulic Reference Manual recommendations. Contraction and expansion coefficients of 0.2 and 0.4, respectively, were set for the cross-sections in the meandering reach of Waikele Stream upstream of Farrington Highway where significant changes in channel geometry occur. Manning's roughness coefficient was estimated from field observations, aerial photographs, available HEC-RAS models (FEMA's effective models of Waikele and Kapakahi Streams and Park's model of Waikele Stream), and technical references (Chow 1959, Barnes 1967). The channel roughness coefficient was set to 0.015 for concrete-lined sections and 0.03 for natural channels. Overbank roughness coefficients were set to 0.03-0.04 for bare earth and grassed areas, 0.05-0.10 for brush (depending on visual estimate of brush density), and 0.15 for mangrove forests in the downstream marsh lands and channel mouth reaches. The roughness coefficients could not be calibrated due to the absence of reliable measured flow and water level data (field data reliability issues are discussed in greater details in subsequent sections below).

5.3.2. Hydraulic Structures

The modeled hydraulic structures on Waikele Stream include the USGS weir upstream of Farrington Highway, two Farrington Highway Bridges (westbound and eastbound), the abandoned railroad bridge in the Energy Corridor, and east bank levee upstream from Farrington Highway. Structures on Kapakahi Stream include the culvert upstream of Farrington Highway, Farrington Highway Bridge, abandoned railroad bridge in the Energy Corridor, a small bridge near the channel mouth, and berms running along the channel. Structures on Wailani Canal include Farrington Highway Bridge, two footbridges in the vicinity of the Energy Corridor, a footbridge at the stream mouth, and berms running along the drainage canals in the Energy Corridor. The footbridge on Waikele Stream downstream of Farrington Highway and Waipio Point Access Road Bridge at the outlet of Wailani Canal are located high above the streams, do not have piers, have no effect on flooding conditions in the study area, and therefore were not included in the models for simplicity. Dimensions and elevations of most of the structures were determined from the survey data collected by ControlPoint during the spring of 2009. Dimensions and elevations of the bridges located on Waikele Stream downstream of the USGS gage were obtained from Park's HEC-RAS model.

Contraction and expansion coefficients at the structures depend on the degree of their encroachment into the channel. Contraction and expansion coefficients of 0.1 and 0.3, respectively, were set for the USGS weir and Farrington Highway Bridges on Waikele Stream and for all the bridges on Wailani Canal. Contraction and expansion coefficients of 0.3 and 0.5, respectively, were specified for the abandoned railroad bridge in the Energy Corridor on Waikele Stream and for the culvert and all the bridges on Kapakahi Stream. The energy-based method was used for computing water surface profiles through the bridge openings. Ineffective flow areas were specified for all the cross-sections affected by the hydraulic structures.

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A number of lateral weirs were specified to simulate the east bank levee on Waikele Stream upstream of Farrington Highway, as well as berms running along Kapakahi Stream and along drainage canals in the Energy Corridor. Under free flow conditions, the weir coefficient can vary between 2.5 and 3.1 depending on levee cross-sectional characteristics. The broad-crested weir coefficient was set at 2.6 for all the modeled weir sections, in accordance with the recommendations of the HEC-RAS Hydraulic Reference Manual.

One of the modeling scenarios included the removal of the levee running along the east bank of Waikele Stream upstream from Farrington Highway. This was achieved by lowering the lateral weir in the HEC-RAS model to the elevation of the adjacent floodplain located immediately to the east of the levee. The floodplain elevations adjacent to the levee were determined using ground topography data within the HHCTCP corridor obtained in 2004 and FEMA LiDAR data developed in 2006 for the FEMA Hurricane Study that happens to cover the study area. These topographic data sets were provided to the project team by the City and County of Honolulu and were also used to develop the 2-d model of the study area.

5.3.3. Model Testing

The model developed for Waikele Stream was tested against streamflow data measured by the USGS at the gage 16213000 located on Waikele Stream upstream of Farrington Highway. Test runs were conducted in a steady state mode using constant peak flows estimated by the USGS for extremely high flow events of February 10, 1979, January 7, 1982, and December 11, 2008. Peak flows estimated for these events are 8,800 cfs, 12,600 cfs, and 22,600 cfs, respectively (USGS 2009). Water surface elevations at the downstream mouth during these events are not known. However, a reasonable tide elevation of 1.9 ft (as was used in the FEMA effective models and Park's model for the 100-year flood event) was used as the downstream controlling stage for all the tested flows. The downstream boundary in the Waikele Stream model is located far enough (about 0.7 miles) from the USGS gage, so it has no effect on streamflow conditions simulated upstream from Farrington Highway.

The computed longitudinal water surface profile for the December 11, 2008 event and maximum water stage observed at the USGS gage during this event are shown in Figure 5.4. Comparison of simulated and observed water surface elevations for all three floods is shown graphically in Figure 5.5 and is summarized in Table 5.2. Results simulated by Park's model for these events are also shown in these figures and the table for comparison. It is seen that both models (NHC's and Park's) predict similar water surface elevations, which are 2-4 ft higher than the stage reported by the USGS for these flood events. The reason for such a consistent difference between the observed and simulated data is uncertain. The deviation of the simulated water surface elevations from the stage reported by the USGS is significant. However, the two different HEC-RAS models (which use independent cross-section data upstream of Farrington Highway) show consistent results. This suggests the overall reasonableness of the model developed in this study.

An attempt was made to match the USGS values by reducing channel and floodplain roughness coefficients, including an artificially small value of 0.01 (corresponding to metal and glass surface roughness). However, this did not change computed water stages significantly. This

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indicates that streamflow hydraulics at the USGS gage is mainly controlled by the significant channel constriction at this location rather then bed surface roughness. Given the extremely complex channel planform, topography, and effects of dense overbank vegetation in the vicinity of the gage, it is possible that the USGS's flow estimates that correspond to their measured water levels or high water marks were overestimated.

No measured streamflow data are available for Kapakahi Stream and Wailani Canal, so the models developed for these streams could not be tested against observed data.

5.3.4. Model Boundary Conditions

The NHC-developed HEC-RAS models were used to simulate the 100-year flood events and to determine breakout flows that leave over the left bank of Waikele Stream at the Waipahu Cultural Garden Park upstream from Farrington Highway, as well as flow characteristics through culverts and bridge openings on Waikele Stream, Kapakahi Stream, and Wailani Canal. The 100-year design flood hydrographs developed by NHC for these streams (see hydrology analysis in Chapter 4) were used as upstream inflows to the HEC-RAS models. The 100-year inflow hydrographs are reproduced in Figure 5.6. The estimated 100-year peak flows are 25,306 cfs for Waikele Stream upstream of the Waipahu Cultural Garden Park, 847 cfs for Kapakahi Stream, and 2,681 cfs for Wailani Canal. In the Waikele Stream model and Wailani Canal model, the inflow hydrographs were applied at the most upstream cross-sections. In the model of Kapakahi Stream (which has no direct inflow at the upstream end and which is fed by multiple local springs in the vicinity of the Waipahu Cultural Garden Park), the inflow hydrograph was uniformly distributed along the upper Kapakahi reach upstream of the culvert. A constant tide elevation of 1.9 ft (as was used in the FEMA effective models and Park's model) was used as the downstream controlling stage. The inflow hydrographs for Kapakahi Stream and Wailani Canal shown in Figure 5.6 do not include overtopping flows from Wailani Canal.

To develop depth-discharge rating tables for hydraulic structures (for subsequent use in the 2-d model), the HEC-RAS models were run in a steady state mode for a range of constant upstream inflows and a constant water surface elevation of 1.9 ft at the downstream boundary.

5.3.5. Computational Parameters

To perform unsteady hydraulic computations, HEC-RAS requires the user to specify certain numerical parameters, which include computation interval, hydrograph and profile output intervals, mixed flow regime option, implicit weighting factor, water surface calculation tolerance, maximum number of iterations, flow stability factors for lateral and inline structures, and weir flow submergence decay exponent. For this study, the computational time step was set to 5 seconds (s) for all the models. This time step was sufficiently short to satisfy the Courant stability condition (Courant numbers less than one) and, at the same time, provided manageable run times. The output intervals for computed stage/flow hydrographs and profiles were set to 10 minutes to provide detailed resolution of the simulated hydraulic data. The simulation period was set to 16 hrs and covered the main phase of the flood events. The mixed flow regime mode

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allowing subcritical and supercritical flows in the models was used in the simulations. Implicit weighting factor was set to 1.0 (fully implicit solution) to provide greater numerical stability. The water surface calculation tolerance was 0.02 ft (default value in HEC-RAS). The maximum number of iterations for solving the unsteady flow equations was 20 (default value). The lateral structure flow stability factor was 2.0, inline structure flow stability factor was 1.0, and weir flow submergence decay exponent was 1.0 (default values). The HEC-RAS model parameters are summarized in Table 5.3.

5.4. HEC-RAS Model Results

5.4.1. Waikele Stream

The 100-year flood maximum water surface profiles simulated for Waikele Stream with- and without the levee at the Waipahu Cultural Garden Park are shown in Figure 5.7. Hydrographs simulated for Waikele Stream at the USGS gage and for breakout flows at the Waipahu Cultural Garden Park for with-levee and without-levee conditions are shown in Figure 5.8.

For the with-levee scenario, most of the flood waters are conveyed down Waikele Stream past Farrington Highway, through the concrete-lined flood control channel, and then the flow spreads out away from the channel onto low-lying floodplain areas downstream of the flood control channel and the Energy Corridor. The computed 100-year peak flow at Farrington Highway is 20,100 cfs. The east-side levee at the Waipahu Cultural Garden Park upstream from Farrington Highway is overtopped during the 100-year event by up to 2-4 ft depending on the location along the levee. Overtopping flows spill eastward into the Waipahu Town area. Maximum breakout flow over the levee is computed to be 4,990 cfs. Upstream of Farrington Highway, the maximum in-channel flow velocities are approximately 5-12 ft/s and overbank velocities (on the floodplain terraces at the Waipahu Cultural Garden Park) are approximately 2-8 ft/s. In the concrete-lined channel portion of Waikele Stream, the flow becomes supercritical with maximum velocities reaching 19-23 ft/s. Downstream of the concrete channel, maximum in-channel flow velocities are generally 5-10 ft/s and overbank flow velocities are approximately 1-3 ft/s. The peak stage in Waikele Stream is below the elevation of both Farrington Highway Bridges and the foot bridge, but exceeds the elevation of the low chord of the abandoned railroad bridge in the Energy Corridor.

For the no-levee scenario more flow leaves Waikele Stream onto the Waipahu floodplain than continues downstream past the USGS gage. For this scenario, the peak in-channel flow below Farrington Highway reduces to 11,500 cfs while peak overbank outflow at the Waipahu Cultural Garden Park increases to 13,900 cfs. For comparison, FEMA (2004) estimated the 100-year flood peak flow of 26,400 cfs below H-1, with approximately 15,300 cfs being conveyed in the Waikele Stream channel below Farrington Highway and the remaining 11,100 cfs breaking out of the channel through the Waipahu Cultural Garden Park upstream of Farrington Highway and flowing eastward to the Waipahu Town area. According to the NHC's HEC-RAS model results, the channel capacity downstream from the concrete-lined section of Waikele Stream is still insufficient to accommodate all the estimated flood flows, which results in flood water leaving Waikele Stream and inundating vast floodplain areas in the reaches adjacent to and downstream

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of the Energy Corridor. Maximum flow velocities in the Waikele Stream channel for the without-levee scenario are within 10-20 ft/s upstream of Farrington Highway, 10-18 ft/s in the concrete-lined channel section downstream of Farrington Highway, and 4-9 ft/s in the mouth reach of the stream. Maximum overbank flow velocities are approximately 2-5 ft/s upstream of Farrington Highway and 1-2 ft/s in the mouth reach below the concrete channel section.

Peak flows simulated by the NHC's HEC-RAS model for Waikele Stream are compared with the FEMA (2004) and Park Engineering (2004) results in Table 5.1. It is seen in this table that for the no-levee conditions NHC's peak flow in Waikele Stream below Farrington Highway is lower than the FEMA's peak flow, while the NHC's overflow from Waikele Stream to the Waipahu Town area is higher than the FEMA's overflow. The NHC model uses the updated hydrology and present day channel, levee and floodplain topography, while the FEMA's results are possibly based on the outdated information and FEMA's method for estimating split flows in the Waikele Stream upstream of Farrington Highway is unclear. Thus, the NHC's overbank flows are conservative relative to the FEMA data and will provide a more conservative estimate of the effects of the project on flooding characteristics in the Waipahu Town area (as simulated by the 2-d model, see Chapter 6).

It should be noted that the 1-d HEC-RAS model of Waikele Stream cannot accurately simulate spatially complex flows in floodplain areas and does not account for return flows back into the stream channel from the Waipahu Town area. Therefore, a 1-d model is likely to underestimate water levels and flows in the reach below the concrete-lined channel where there is active interaction between in-channel and overbank flows. The HEC-RAS results were used to define inflow boundary conditions for the 2-d model and to develop depth-discharge rating tables for the bridge crossings to simulate their effect in the 2-d model (which does not model hydraulic structures explicitly).

5.4.2. Kapakahi Stream

The Kapakahi Stream model was used to simulate local runoff and did not include overbank inflows from Waikele Stream. The computed maximum 100-year flood water surface profile for Kapakahi Stream is shown in Figure 5.9. The HEC-RAS simulation shows that the flow capacity of Kapakahi Stream upstream of Farrington Highway is insufficient to accommodate all the flood waters entering the stream. The channel capacity is restricted by its small size and the limited capacity of the culvert located immediately upstream of Farrington Highway. The 100-year peak inflow to Kapakahi Stream from local rainfall and runoff only is approximately 847 cfs (this does not include overbank inflow from Waikele Stream). Of this peak flow, only 192 cfs flows through the culvert and continues downstream of Farrington Highway. Therefore, the present day Kapakahi Stream culverts that run beneath the parking lots north of Farrington Highway have insufficient capacity to carry flow from just the local 100-year runoff. The water in excess of the channel and culvert capacity spill over the stream banks into adjacent urban floodplain areas. According to the HEC-RAS model, in-channel maximum flow velocities in Kapakahi Stream are up to approximately 2-4 ft/s upstream of the culvert, 2-5 ft/s between Farrington Highway and the Energy Corridor, and up to 1-3 ft/s in the reach below the Energy Corridor.

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The 1-d HEC-RAS model of Kapakahi Stream does not include any floodplain areas and does not account for overbank (breakout) waters returning from the inundated floodplain back to the channel. It also does not include overtopping flows from Waikele Stream that periodically flow over the east levee upstream of Farrington Highway. Therefore, the 1-d model significantly underestimates flows in Kapakahi Stream below the culvert that occur during a 100-year flood if you were to include the likely overtopping flows from Waikele Stream. The HEC-RAS model of Kapakahi Stream was mainly used to determine depth-discharge rating tables for the culvert and bridge crossings for subsequent use in the 2-d model.

5.4.3. Wailani Canal

The Wailani Canal model was used to simulate in-channel flow from local runoff and did not include overbank inflows from Waikele and Kapakahi Streams. The computed maximum 100-year flood water surface profile along Wailani Canal is shown in Figure 5.10. According to the HEC-RAS model results, the canal has sufficient capacity to convey the 100-year flood discharge generated from local rainfall and runoff upstream from Paiwa Street. However, this assumes that no additional lateral inflow comes from the Waipahu floodplain. The only overtopped areas are located north of the east-side drainage canal running parallel to the Energy Corridor. Maximum flow velocity in Wailani Canal is approximately 8-10 ft/s upstream of Farrington Highway, 6-11 ft/s between Farrington Highway and the Energy Corridor, and reduces to approximately 3-7 ft/s in the reach downstream of the Energy Corridor. None of the bridges that cross the canal are overtopped, although the peak water level in the canal is only about 0.4 ft below the low chord of the Farrington Highway Bridge. The 1-d model of Wailani Canal, however, does not account for possible inflow of overbank flows from the Waipahu Town area and therefore may underestimate flows and water levels along the study reach of the canal during a basin-wide 100-year flood event. The NHC HEC-RAS model of the canal was used to develop depth-discharge rating tables for the bridge crossings for subsequent use in the 2-d model.

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Table 5.1. 100-year peak flows used in previous and present studies.

Stream	100-year flood peak flow (cfs)		
	FEMA (2004)	Park Engineering (2004)	NHC
Waikele Stream below H-1 Freeway	26,000	26,400**	25,306
Waikele Stream below Farrington Hwy (with levee)	N/A	26,400**	20,100***
Waikele Stream below Farrington Hwy (no levee)	15,300	N/A	11,500***
Overflow from Waikele Stream (with levee)	N/A	N/A	4,990***
Overflow from Waikele Stream (no levee)	11,971*	N/A	13,900***
Kapakahi Stream (local drainage)	N/A	N/A	847
Wailani Canal	N/A	N/A	2,681

* Includes Waikele Stream overflow and Kapakahi Stream local drainage;
** Drainage area at ocean; *** NHC's HEC-RAS model.

Table 5.2. Comparison of observed and simulated water surface elevations.

Date	Peak flow (cfs)	Water surface elevation (ft HILOCAL)			Difference (ft) (model minus observed)	
		Observed	NHC model	Park model	NHC model	Park model
Waikele Stream at USGS gage 16213000						
12/11/2008	22,600*	21.07	23.61	24.03	2.54	2.96
1/7/1982	12,600*	13.90	17.77	17.87	3.87	3.97
2/10/1979	8,800*	12.17	15.11	14.98	2.94	2.81

* Estimated by USGS.

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Table 5.3. HEC-RAS model parameters for unsteady computations.

Parameter	Value
Computational interval	5 sec
Hydrograph and profile output intervals	10 min
Simulation period	16 hrs
Mixed flow regime option	Yes
Implicit weighting factor	1.0 (default value)
Water surface calculation tolerance	0.02 ft (default value)
Maximum number of iterations	20 (default value)
Lateral structure flow stability factor	2.0 (default value)
Inline structure flow stability factor	1.0 (default value)
Weir flow submergence decay exponent	1.0 (default value)

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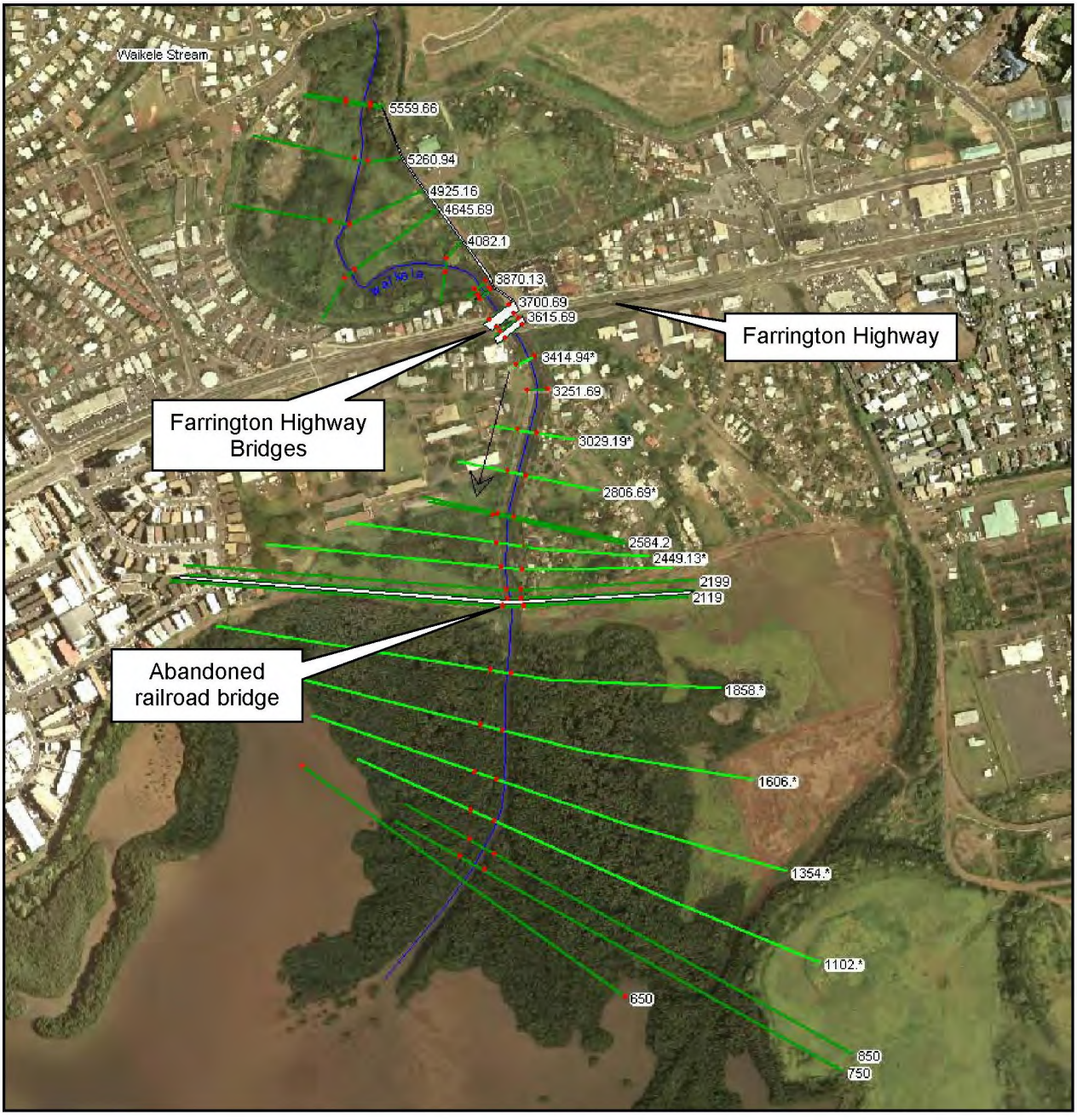

Figure 5.1. NHC's HEC-RAS model layout for Waikele Stream. Downstream of Farrington Highway, NHC's model was extended using cross-section and bridge data from HEC-RAS model developed by Park Engineering (2004).

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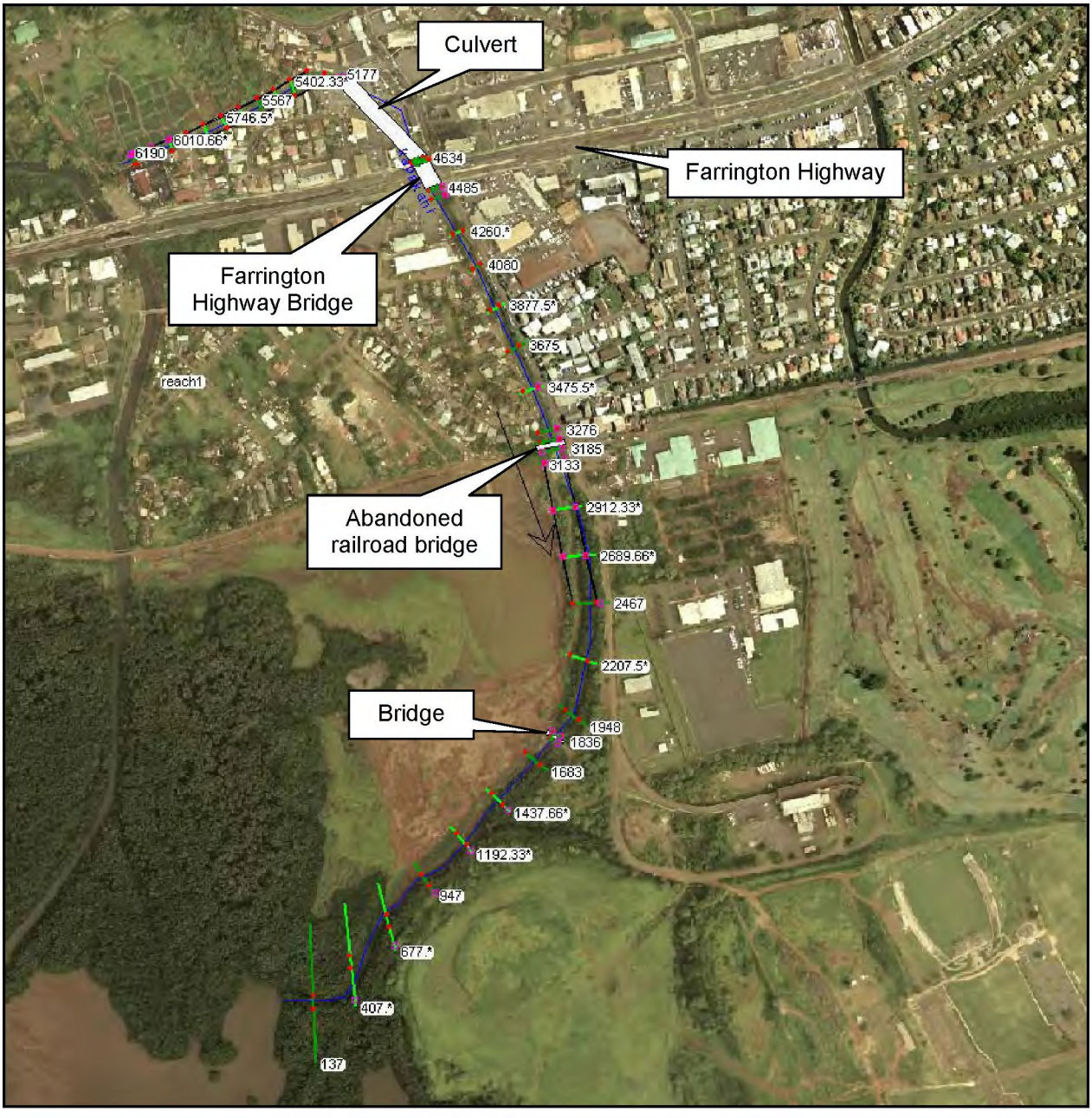

Figure 5.2. NHC's HEC-RAS model layout for Kapakahi Stream.

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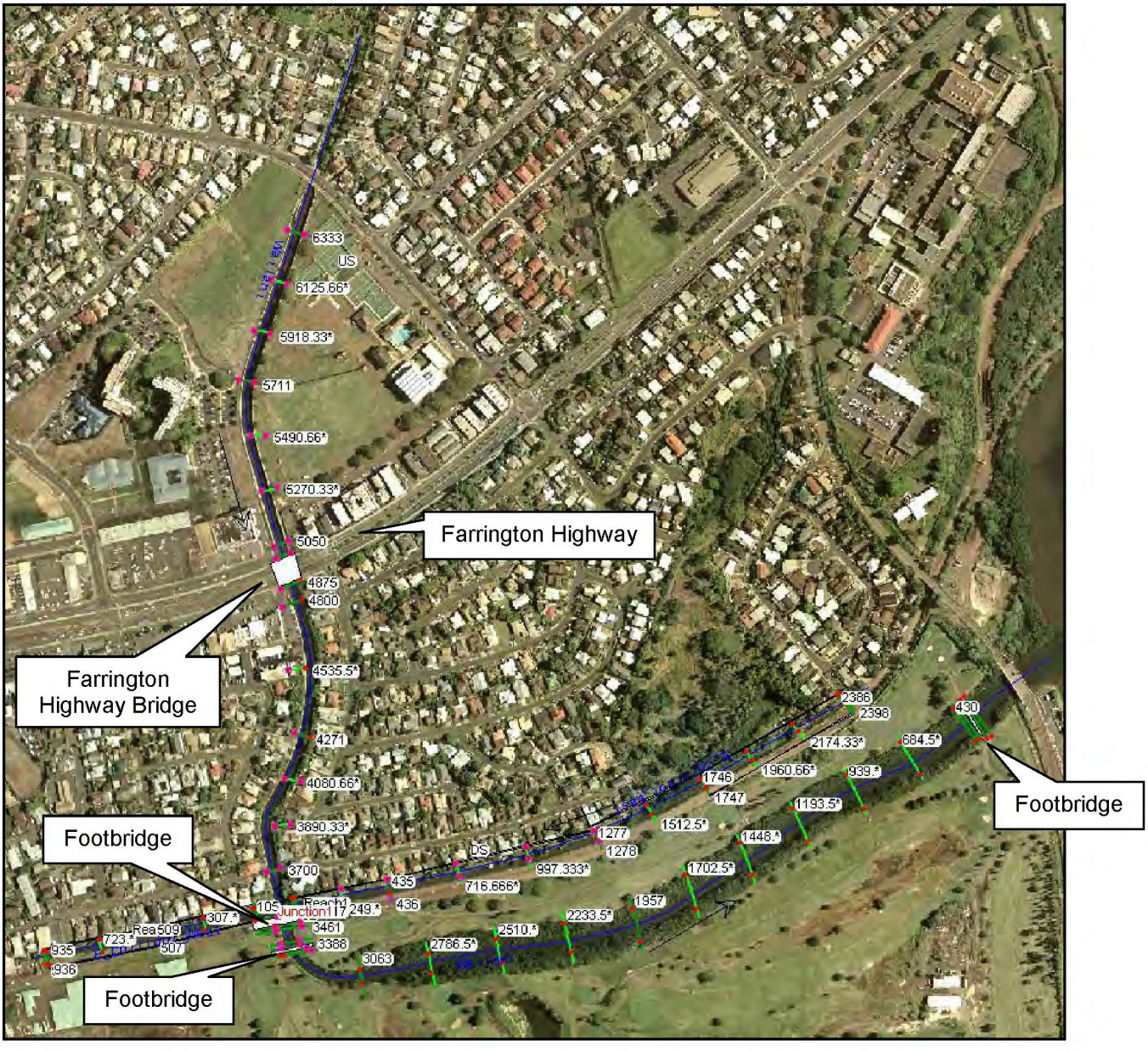

Figure 5.3. NHC's HEC-RAS model layout for Wailani Canal.

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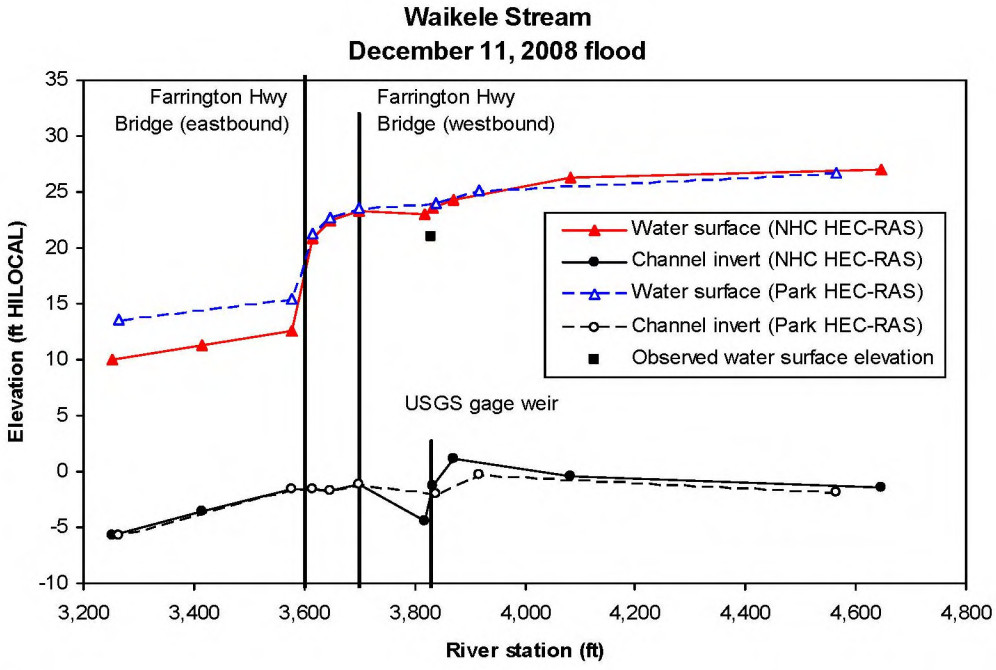

Figure 5.4. Longitudinal water surface profiles simulated for Waikele Stream at USGS gage 16213000 for December 11, 2008 flood peak flow by NHC's and Park's HEC-RAS models.

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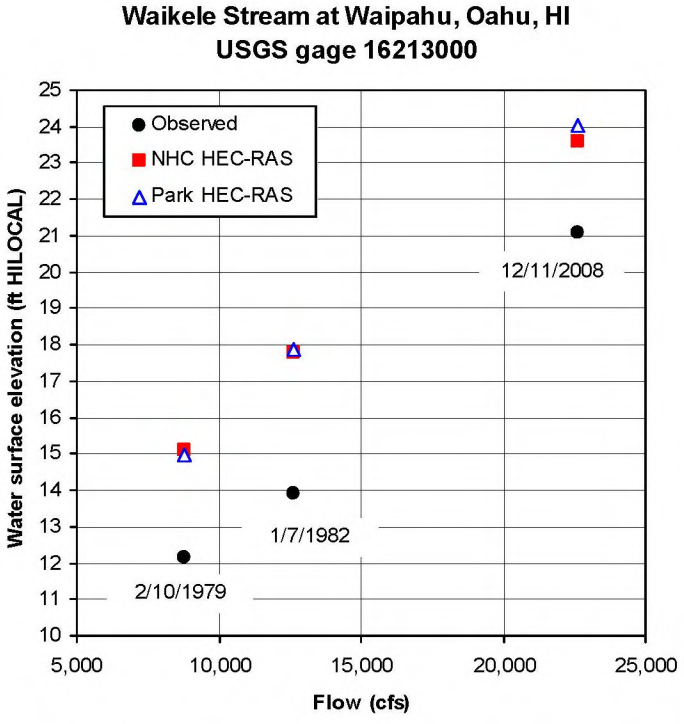

Figure 5.5. Comparison of observed (by USGS) and simulated (by NHC' and Park's HEC-RAS models) water surface elevations in Waikele Stream at USGS gage 16213000.

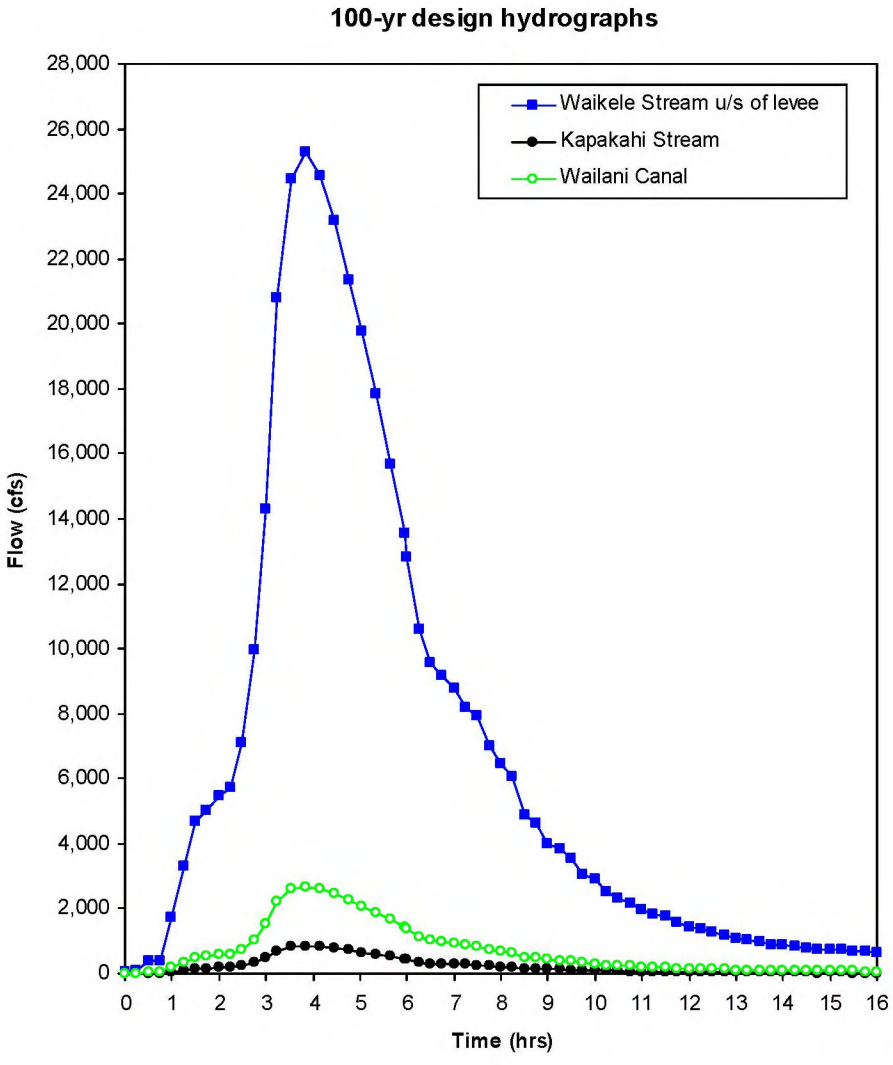

Figure 5.6. Inflow 100-year design hydrographs.

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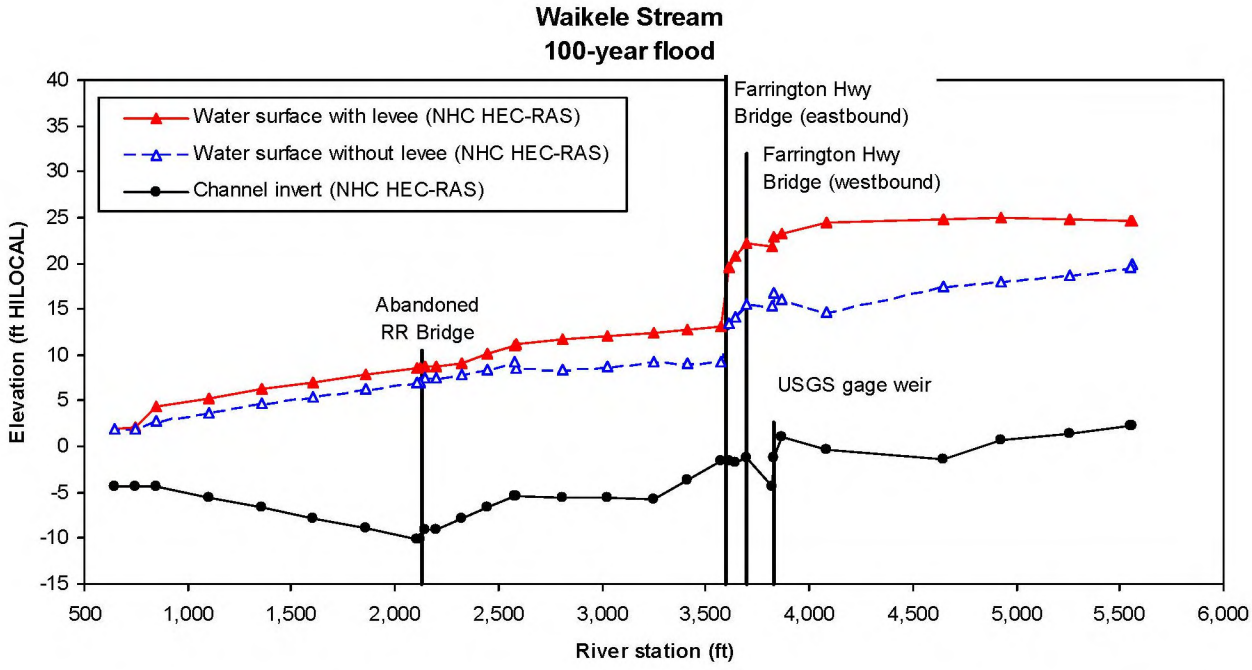

Figure 5.7. Maximum water surface profiles simulated for Waikele Stream for 100-year flood with and without levee at Waipahu Cultural Garden Park by NHC's HEC-RAS model.

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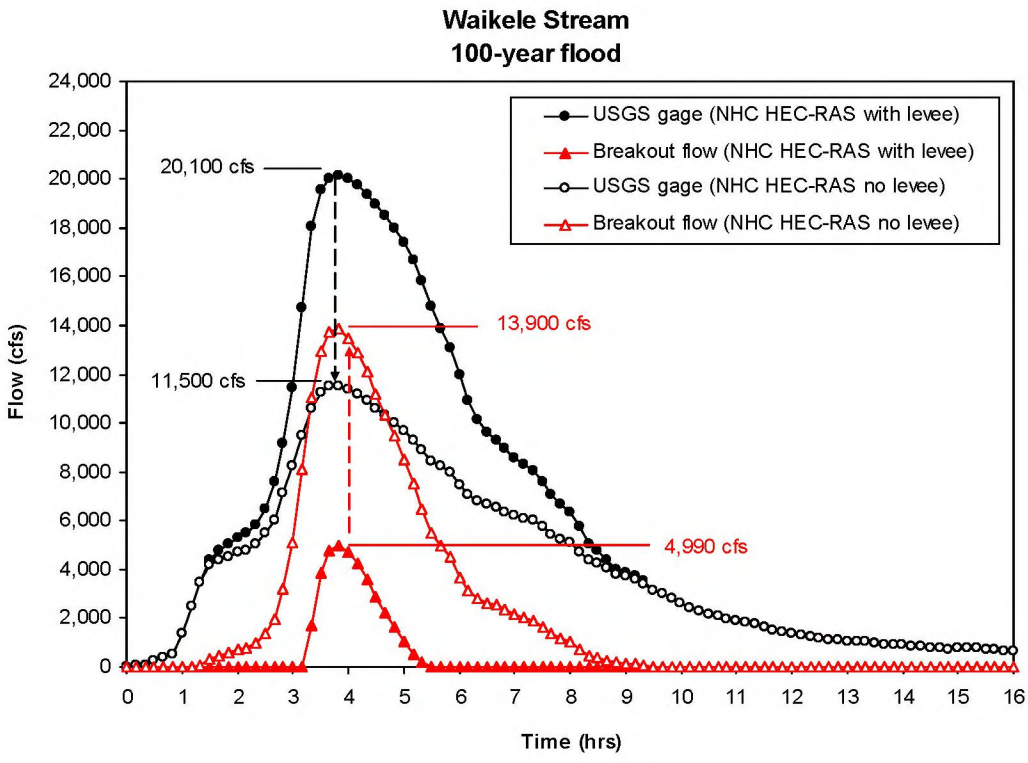

Figure 5.8. Hydrographs simulated for Waikele Stream at USGS gage and for breakout flow at Waipahu Cultural Garden Park for with-levee and without-levee conditions by NHC's HEC-RAS model.

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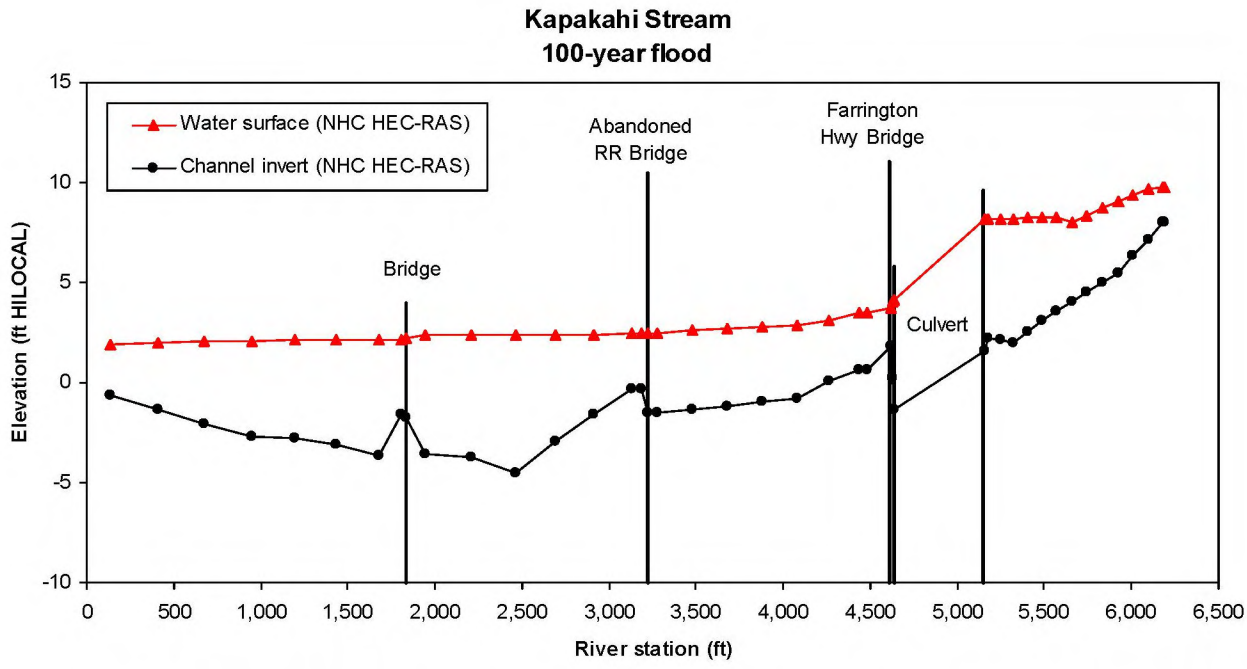

Figure 5.9. Maximum water surface profile simulated for Kapakahi Stream for 100-year flood by NHC's HEC-RAS model. Note that results shown here are obtained for local runoff from upper Kapakahi watershed and do not include overtopping inflows from Waikele Stream and return flows from floodplain areas back into Kapakahi Stream below Farrington Highway.

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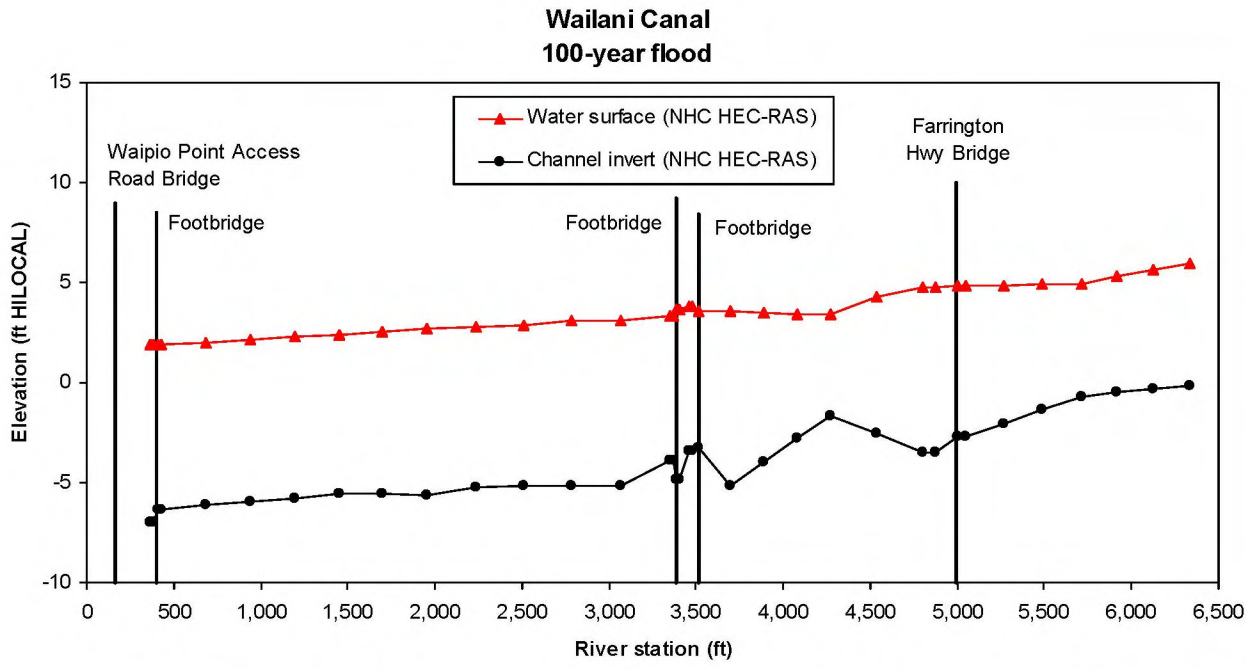

Figure 5.10. Maximum water surface profile simulated for Wailani Canal for 100-year flood by NHC's HEC-RAS model. Note that results shown here are obtained for local runoff from upper Wailani watershed and do not include possible overbank inflows from Waikele and Kapakahi Streams.

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6. TWO-DIMENSIONAL HYDRAULIC MODELING

NHC developed a set of two-dimensional (2-d) unsteady fixed-bed FLO-2D computer models for the Waipahu floodplain system in the vicinity of the proposed Waipahu Transit Station. FLO-2D provides a well tested tool for quantifying spatial flow hydraulics in complex riverine and floodplain environments similar to the Waipahu study area (particularly shallow urban flooding) and is accepted by FEMA for floodplain assessments and mapping. Two models with different spatial coverage and grid resolution were developed: (1) a primary area-wide coarse grid model (with 50 ft grid cell size) and (2) a supporting smaller-area fine gird model (with 10 ft grid cell size). The coarse grid model covers approximately 1.6 square miles of the study area and includes the lower reaches of Waikele Stream, Kapakahi Stream, and Wailani Canal. The fine grid model focuses on a 0.4 square mile area along Farrington Highway between Waikele Stream and Wailani Canal. The coarse grid model was developed to evaluate area-wide flooding characteristics with and without proposed project features included. The fine grid model was developed to simulate flooding details in the immediate vicinity of the proposed project structures and to check the reasonableness of the coarse grid model predictions.

The Waipahu area FLO-2D models were developed using LiDAR-based topography data within the HHCTCP corridor obtained by John Chance Land Surveys, Inc. (JCLS) in 2007, FEMA LiDAR data developed in 2006 for the FEMA Hurricane Study and covering the study area, and cross-section data collected along the study streams by ControlPoint Surveying, Inc. (ControlPoint) in the spring 2009. Inflow hydrographs for the streams and overbank flows and a rating table for the Kapakahi Stream culvert were determined from NHC's up-to-date one-dimensional (1-d) HEC-RAS models developed for the study streams (see Chapter 5).

The two NHC 2-d models were used to simulate 100-year flooding in the Waipahu Town area under existing (no project) and with-project conditions and to evaluate project alternatives that may be needed to reduce or eliminate impacts to water levels in the floodway caused by the project. Two flooding scenarios were evaluated for both existing and with-project conditions: (1) the first scenario includes the existing east bank levee on Waikele Stream at the Waipahu Cultural Garden Park upstream of Farrington Highway and includes the existing flood walls along Wailani Canal (to evaluate the most likely flooding scenario), and (2) this scenario excludes the levee and flood walls (to comply with FEMA's regulations that require removal of all non-certified flood control structures from model simulations). The levee along Waikele Stream at the Waipahu Cultural Garden Park and the flood walls along Wailani Canal are not "certified" structures and as such are not recognized by FEMA as providing flood protection (therefore, the effective FEMA Flood Insurance Study (FIS) assumes that the levee and flood walls do not exist). Results from the 2-d simulations were used to assess impacts of the project features (station buildings and piers supporting elevated railroad) on flooding extent and maximum inundation depths in the Waipahu area.

This chapter describes the development of the FLO-2D models, derivation of input data used in the models, key assumptions, model parameters, modeling scenarios, and preliminary results from the computer simulations. All elevations in the model and in the report are in feet, HILOCAL.

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6.1. Description of Computer Model FLO-2D

Overland flooding in the study area was simulated using computer model FLO-2D (version 2007.06). FLO-2D is a quasi 2-d volume conservation model that simulates channel flows and overland flows including unconfined flows over complex topography and roughness, split channel flows, and urban flooding (FLO-2D Software Inc 2009). The model numerically routes a flood hydrograph while predicting the area of inundation and floodwave attenuation. FLO-2D uses the full dynamic wave momentum equation and a central finite difference routing scheme with eight potential flow directions within each computational cell to predict the progression of a flood hydrograph over a system of uniform square grid elements.

FLO-2D can be applied to simulate a variety of complex flood problems including:

- River overbank flooding
- Unconfined alluvial fan and floodplain flows
- Urban flooding
- Flood insurance studies
- Flood mitigation design

FLO-2D is a hydraulic model approved by the Federal Emergency Management Agency (FEMA) for riverine and unconfined floodplain studies. The following descriptions provide a brief overview of the main features of the model pertinent to river and overland flooding.

6.1.1. Surface Topography

FLO-2D requires two sets of data for any flood simulation, a digital terrain model (DTM) and an inflow hydrograph. The potential flow surface topography is represented by a uniform square grid system. The grid elements (or grid cells) are assigned elevations from an interpolation of the DTM points. A processor program GDS (grid developer system) generates the grid system and assigns the elevations to each grid element. The GDS superimposes a grid system over the DTM points and interpolates grid cell elevations using DTM point filters. It automatically generates the FLO-2D floodplain and other data files to start an overland flood simulation. Images can be imported into the GDS to assist graphical editing. Any size grid cell can be used in the model, but the computational time step is governed by wave celerity and small grid cells will require smaller time steps which require longer model run times. A typical square grid cell size will range from 10 ft to 500 ft on a side. The number of grid cells is unlimited. However, if the number of grid cells exceeds 100,000, the model simulation may be very slow and may take days or weeks to run.

6.1.2. Surface Roughness

For flows less than 0.2 ft deep (where the flow depth is on the order of the roughness elements), a default value for a shallow flow Manning's roughness coefficient of 0.2 is used in FLO-2D. For flow depths between 0.2 and 0.5 ft, the shallow flow roughness coefficient is reduced by

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half. To improve the timing of the floodwave progression through the grid system, a depth-dependent roughness is assigned by the model for flow depths ranging from 0.5 ft to 3 ft. For depths in excess of 3 ft, a user-defined Manning's roughness value is used.

6.1.3. Inflow Hydrographs

Inflow hydrographs can be designated for either channel or floodplain nodes. The number of inflow hydrograph nodes is unlimited.

6.1.4. Routing Algorithm Stability and Volume Conservation

Computational time steps are automatically determined by FLO-2D and typically range from 1 to 30 seconds. The time step is increased or decreased automatically according to strict flood routing numerical stability criteria. Numerical stability is linked to volume conservation. The key to any successful flood routing model is volume conservation. When the model accurately conserves volume the model runs faster and is likely to produce reliable results. Volume conservation is tracked and is reported both during the simulation and in summary output files when the simulation is complete.

6.1.5. Channel and Overland Flow Simulation

Channel flow is simulated as 1-d flow in FLO-2D, with the channel geometry represented by natural, rectangular or trapezoidal cross-sections. The GDS can convert HEC-RAS cross-sections into a data file formatted for FLO-2D. Overland flow is modeled as 2-d flow as either sheet flow or flow in multiple channels (rills and gullies). Unconfined overland flow is simulated in eight directions (4 compass directions and 4 diagonal directions). Channel overbank flow is computed when the channel capacity is exceeded. An interface routine calculates the channel to floodplain flow exchange including return flow to the channel. Once the flow overtops the channel, it will disperse to other overland grid cells based on topography, roughness, and obstructions. For urban flooding, the model can simulate channel overbank flow through residential areas with street conveyance and flow around obstructions and then have the flow return to the channel downstream, depending on floodplain topography.

6.1.6. Hydraulic Structures, Levees, Buildings and Flow Obstructions

Hydraulic structures (such as bridges, culverts, weirs) can be simulated by user specified discharge rating curves or rating tables assigned to either channel or floodplain elements. Levees, berms, road embankments, small dams, and flood walls can be approximated by specifying crest elevations for any combination of the eight grid cell flow directions. Streets can be modeled as shallow rectangular channels by specifying a width and curb height. Floodplain storage loss due to buildings and other flow obstructions (such as poles, piers, and large trees) on a grid cell basis can be modeled using area reduction factors (ARF). A portion of a grid cell or the entire cell can

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be removed from potential inundation during a flood simulation. Reducing the amount of flood storage forces more flow downstream. The flow exchange between grid cells can be partially or entirely obstructed with a flow width reduction factor (WRF) for any or all of the eight flow directions.

6.1.7. Model Output, Results and Mapping

The floodwave progression over the flow surface can be viewed along with a plot of the inflow hydrograph while the model is running. The main output results from a flood simulation include maximum water surface elevations within the computational domain, maximum flow depths, velocities, and velocity vector. The simulation results can be viewed graphically in the MAPPER post-processor program. MAPPER automatically generates and saves shape files of flood plots for viewing in ArcGIS.

6.2. FLO-2D Model Development

The model development components included development of the model geometry and hydraulic boundary conditions. The following sections describe in detail the methods, approximations, and assumptions used in developing the FLO-2D models of the study area.

6.2.1. Model Geometry

The coarse grid model covers approximately 1.6 square miles (1,010 acres) of the Waipahu study area extending approximately from Waipahu Street to the West and Middle Locks and including the lower Waikele Stream downstream of the Waipahu Cultural Garden Park, the entire Kapakahi Stream, and Wailani Canal downstream of Paiwa Street. The fine grid model focuses on a 0.4 square mile (240 acres) area extending from Waipahu Street to the Energy Corridor and including the Waipahu Cultural Garden Park and residential areas along and between Kapakahi Stream and Wailani Canal. The extent (floodplain coverage) of the coarse gird and fine grid model domains are shown in Figures 6.1 and 6.2, respectively.

Ground surface topography in the FLO-2D models was developed using a topographic data set developed by NHC. This topographic data set combines 1 ft contour topography data within the HHCTCP corridor obtained by JCLS in 2007 (official data for the project) with 5 ft by 5 ft grid FEMA LiDAR data developed in 2006 for the FEMA Hurricane Study and covering the Waipahu Town area and generalized bathymetry for the bay areas in the West and Middle Lochs.

The model grid cell size was selected according to the FLO-2D user's manual recommendations and was set to 50 ft for the coarse grid models and to 10 ft for the more focused fine grid model. These grid cell sizes were sufficiently small to describe relatively small-scale topographic details (such as streets, highways, large buildings, HHCTCP project stations) and at the same time provided manageable model run times. There are 17,613 grid cells in the coarse grid model and

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106,652 grid cells in the fine grid model. An average ground elevation within each grid cell was computed from the topographic data.

In the coarse grid model, 1-d stream channels were specified within the 2-d computational domain using channel cross-sections (bathymetry) data collected by ControlPoint in the spring of 2009 (the same cross-section data were used to develop the HEC-RAS models for the Waikele Stream, Kapakahi Stream, and Wailani Canal). In the fine grid model, the channels of Kapakahi Stream and Wailani Canal were modeled as part of the 2-d floodplain model rather then 1-d channels because the grid cell size (10 ft by 10 ft) is sufficiently small to adequately depict the channel conditions.

Model roughness (Manning's roughness coefficient) was estimated from field observations, aerial imagery, available HEC-RAS models [FEMA's effective models of Waikele and Kapakahi Streams and Park's model of Waikele Stream, see Chapter 5], and technical references (FLO-2D user's manual, Chow 1959, Barnes 1967). A roughness coefficient of 0.04 was assigned to all grid cells within urban and open floodplain areas. This value was within the 0.02-0.05 range suggested in the FLO-2D user's manual for asphalt and concrete and within the 0.04-0.10 range suggested for grassland and open floodplain areas. The value of 0.04 was believed to be generally representative of the urbanized areas modeled. A roughness coefficient of 0.07 was assigned to areas covered with thick brush and 0.15 for mangrove forests. The roughness of the bay bottom was set to 0.02. The channel roughness coefficient was set to 0.015 for concrete-lined sections and 0.03 for natural channels (same as specified in the supporting 1-d HEC-RAS models).

In order to simulate loss of storage capacity due to numerous buildings and other structures located in the floodplain, appropriate Area Reduction Factors (ARF) were assigned to all grid cells within developed areas in the coarse grid model. ARF values were calculated using the following procedure. Within the model boundaries, sub-regions with similar types of development (residential, commercial, or industrial) were outlined based on visual assessment of aerial photographs. ARF sub-regions were specified so as not to block major highways and streets (most highways and streets appeared in the 2-d model as topographical depressions and were important conveyors of overland flows). A single ARF value was then calculated for each sub-region as a ratio of the cumulative area occupied by buildings in a sub-region and the total area of the sub-region. The area occupied by buildings was calculated using an ArcGIS buildings shapefile (which includes all the buildings in the study area and which was provided to NHC by the City of Honolulu). Large individual buildings (including all the commercial buildings in the vicinity of the project transit station) were explicitly modeled by blocking grid cells that were occupied by large building (in this case assigning ARF values was not needed). The U.S. Geological Survey (USGS) aerial imagery collected in 2004 was used to identify the location of individual buildings in the vicinity of the project. In addition to using ARF, width reduction factors (WRF) were specified at a few locations in the vicinity of the project structures to better simulate obstructions and the effects from adjacent buildings.

In the fine grid model, all the buildings within the computational domain were modeled by blocking appropriate grid cells using the City of Honolulu buildings shapefile and USGS 2004 aerial imagery. There are indications that a few new buildings have been constructed and some

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old buildings may have been removed on the east bank of Kapakahi Stream to the south of Farrington Highway. However, in the absence of reliable official information and data, these recent changes are not included in the present models. For consistency with the coarse grid model, the fine gird model reflects the same situation in this area according to the USGS 2004 imagery.

Hydraulic structures in the study area include a few bridges on Waikele Stream, Kapakahi Stream, and Wailani Canal and an approximately 500 ft long culvert on Kapakahi Stream upstream of Farrington Highway. Hydraulic structures in FLO-2D can be modeled by specifying depth-discharge rating tables. Initially, it was intended to use the supporting HEC-RAS models (see Chapter 5) to determine these rating tables. However, preliminary FLO-2D test simulations revealed a significant mismatch between the channel conveyances at the bridges obtained from the HEC-RAS model compared to the FLO-2D model. This is likely due to the different computational methods used for 1-d channels in HEC-RAS and FLO-2D. This mismatch resulted in unrealistic backwater effects (up to a few feet of ponding) upstream of some of the bridges in the FLO-2D model, which in turn affected the magnitude of overland flooding, mainly in the vicinity of the Energy Corridor. Therefore, the effect of the existing bridges on in-channel hydraulics was simulated in the FLO-2D models by assigning a higher roughness coefficient of 0.07 to stream channels at the bridge crossings instead of using rating tables. Only the Kapakahi Stream culvert was modeled using a rating table developed from the HEC-RAS data. This culvert has a limited flow capacity and its behavior in the FLO-2D model appears to be reasonable.

Overland flow in the 2-d model was governed by the land surface topography and the effects of buildings. Small-scale features such as individual streets, drainage ditches, and small berms were not modeled explicitly due to the lack of fine resolution topographic data. The only additional features that were included in the models were the flood walls along Wailani Canal. These flood walls were simulated in the FLO-2D model by specifying crest elevations (taken from the surveyed cross-sections) for the walls within individual cells containing the wall. The existing levee between Waikele Stream and the Waipahu Cultural Garden Park was not explicitly included in the FLO-2D model geometry. The FLO-2D inflow model boundary was set on the landward side of the levee. The with-levee and no-levee inflow boundary conditions used in the FLO-2D models were obtained from the HEC-RAS overtopping simulations of Waikele Stream as discussed in Chapter 5.

Once developed and tested, the existing conditions models were used as a base for developing the project conditions models. Location and dimensions of the project features such as station entrance buildings and quideway support columns were determined from drawings provided by PB. Project buildings were incorporated by blocking grid cells within the station footprints. The total areas of the blocked grid cells in the models are approximately equal to the actual footprint areas for the buildings. Guideway support columns along the project corridor were assumed to have a 10 ft by 10 ft square shape. In the coarse grid model, the area occupied by a single column (100 square feet) is much less than the grid cell area (2,500 square feet). Therefore, the support columns in the coarse grid model were simulated by specifying an ARF value of 0.04 for all the grid cells containing project columns. In the fine grid model, the guideway support columns were simulated by blocking grid cells within column footprints.

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6.2.2. Hydraulic Boundary Conditions

The main assumption in the development of the FLO-2D model was that flooding occurs simultaneously on all the streams in the study area (Waikele Stream, Kapakahi Stream, and Wailani Canal). The 100-year hydrographs obtained from the HEC-RAS model for Waikele Stream at Farrington Highway and overbank outflows from the Waikele Stream at the Waipahu Cultural Garden Park (see Chapter 5) were used to specify upstream inflows into the FLO-2D model domain from Waikele Stream for with-levee and no-levee conditions. At the same time, the 100-year design hydrographs developed for Kapakahi Stream and Wailani Canal (see Chapter 4) were used to specify upstream inflows in these streams in the 2-d model. Peak flows used in this study are summarized in Table 5.1 in Chapter 5.

A steady tide elevation of 1.9 ft was used as the downstream controlling stage in the coarse grid models (same as in the NHC's, FEMA's, and Park's 1-d models). This tide elevation was established by specifying a row of stage-time control elements along the shoreline and at the downstream cells of the stream channels. Water level in these control elements was gradually raised (over a 4 hr simulation period ahead of the flood) from 0 ft elevation to the elevation of 1.9 ft. This provided gradual infilling of the low-lying floodplain areas in the models with tidal waters prior to initiating the flood flows. The channels were filled to the tide level by setting an initial water surface elevation in all the channels at 0.1 ft and specifying an initial flow of 1,000 cfs for Waikele Stream, 50 cfs for Kapakahi Stream, and 500 cfs for Wailani Canal. The 4 hr initial period was established from a series of preliminary runs and was sufficient to achieve initial steady state tidal conditions throughout the modeled area. Timing of the upstream inflow hydrographs was adjusted to account for this initial 4 hr period. Outflow points were assigned behind the stage-time control elements to withdraw water from the model.

In the fine grid model, outflow points were assigned along the entire downstream boundary running along the Energy Corridor. Modelers determined that simulated flow characteristics in the vicinity of the project in the fine grid model are not affected by the downstream boundary conditions which in the fine grid model do not account for backwater effects from waters outside of the modeling domain. This is supported by results from the coarse grid model which indicate that overland flow in the Waipahu Town area has a positive seaward gradient (i.e. water surface here is not horizontal) and therefore backwater effects from waters south of the Energy Corridor are insignificant and do not affect flooding in the vicinity of the project.

Infiltration and evaporation losses were assumed negligible compared to the volume of the surface runoff and therefore were not modeled.

6.2.3. Model Runs

The FLO-2D models could not be calibrated to observed events due to the absence of reliable calibration data. The models were used to route flood flows through the streams and Waipahu Town area as they progressed toward Pearl Harbor. The following modeling scenarios were analyzed:

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Existing Conditions

(1) With levee, no project – With levee means that this scenario simulated existing conditions flooding with the levee between Waikele Stream and the Waipahu Cultural Garden Park in place and the existing flood walls along Wailani Canal in place. No project means that no HHCTCP project structures were included in this scenario.
(2) No levee, no project – No levee means that this scenario simulated flooding without the Waikele Stream levee and without the Wailani Canal flood walls (to comply with FEMA's regulations that assume non-certified levees and flood walls do not exist). No project means that no HHCTCP project structures were included in this scenario.

Project Conditions (Original Design)

(3) With levee, with project – Same as Scenario (1), but with the originally proposed HHCTCP project features in place (including the north and south entrance buildings and guideway support columns).
(4) No levee, with project – Same as Scenario (3), but without the Waikele Stream levee and without the Wailani Canal flood walls.

Project Alternative 1

(5) With levee, with north entrance building – Same as Scenario (3), but only with the north entrance building in place and without the south entrance building.
(6) No levee, with north entrance building – Same as Scenario (5), but without the Waikele Stream levee and without the Wailani Canal flood walls.

Project Alternative 2

(7) With levee, with north entrance building and reduced south entrance building – Same as Scenario (3), but with the footprint of the south entrance building reduced by approximately half.
(8) No levee, with north entrance building and reduced south entrance building – Same as Scenario (7), but without the Waikele Stream levee and without the Wailani Canal flood walls.

Project Alternative 3

(9) With levee, with elevated north entrance plaza and north entrance building and minimum south entrance building footprint – This scenario includes an elevated (above the 100-year flood level) north entrance plaza that fully fills the gap between the existing adjacent buildings, an elevated (above the 100-year flood level) ground floor for the north entrance building, and a new south entrance building in the same approximate location with the same maximum external ground floor dimensions (approximately 40 ft east to west by 20 ft north to south) of the existing building it replaces.

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(10) No levee, with elevated north entrance plaza and north entrance building and minimum south building footprint – Same as Scenario (9), but without the Waikele Stream levee and without the Wailani Canal flood walls.

The simulation time step in the model runs was variable and is adjusted by FLO-2D automatically in order to provide numerically stable solutions. In the coarse grid models, the total simulation time was set to 20 hrs. This simulation time was sufficiently long so that the in-channel and overland flows reached the downstream model extent or accumulated in local depressions and no significant water flow occurred by the end of the simulations. In the fine grid models (having much smaller modeling domains), the simulation time was set to 6 hrs to compute peak flow characteristics in the vicinity of the project. The recession limb of the flood wave was not modeled in the fine grid model. Model runs generally ranged from a few hours to a few days for the coarse grid models. The fine grid models usually took a couple of weeks to finish a simulation. All the modeling scenarios described above were simulated using the coarse grid models. Due to the extremely long model run times, the fine grid model was used to simulate only the no-levee scenario for both existing and with-project conditions.

Model development included a series of preliminary runs needed to test and refine various model components, boundary conditions, and computational parameters. The final refined models used for production runs and sensitivity analyses are summarized in Table 6.1. Production runs included simulation of the 100-year flooding (for with-levee and no-levee scenarios) under existing and with-project conditions and evaluation of various project alternatives. Computed maximum inundation depths, maximum stream velocities, and changes in depths and velocities caused by the proposed project structures were mapped in GIS. The following sections discuss results obtained for different modeling scenarios.

According to the FLO-2D developer (Jim O'Brien), the accuracy of water surface elevations and depths computed by FLO-2D is limited to about 0.1 ft. Therefore, in the following analysis all depths are rounded to the nearest 0.1 ft and the differences in depths and elevations less than 0.1 ft (i.e. less than the FLO-2D accuracy) were considered insignificant and therefore were not mapped.

6.3. FLO-2D Model Results

The maximum 100-year water surface elevations, maximum inundation depths, and maximum velocities predicted for the Waipahu Town area by the 2-d models were post-processed and mapped using ArcGIS. The flood inundation maps are attached to this report in Appendices A-F. Below is a discussion of flooding patterns and flow characteristics determined for the study area with special emphasis on the area in the immediate vicinity of the proposed project features.

6.3.1. Existing Conditions

Maximum floodplain water surface elevations and maximum inundation depths simulated by the coarse grid model for the **with-levee, no-project** scenario are shown in Figures A-1 and A-2 (in

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Appendix A), respectively. Maximum depths and velocity vectors (red arrows) simulated by the coarse grid model in the vicinity of the future project station for the with-levee, no-project scenario are shown in Figure A-3 (in Appendix A). Entrance Building outlines are shown but the buildings are not included in the "no-project" scenario.

Maximum floodplain water surface elevations and maximum inundation depths determined by the coarse grid model for the **no-levee, no-project** scenario are shown in Figures B-1 and B-2 (in Appendix B), respectively. Figure B-2 compares the flood extent determined with the 2-d model with FEMA's AE flood zone (developed for the no-levee conditions). Maximum depths and velocity vectors (red arrows) simulated by the coarse grid model and fine grid model in the vicinity of the future project station for the no-levee, no-project scenario are shown in Figures B-3 and B-4 (in Appendix B), respectively. Entrance Building outlines are shown but the buildings are not included in the "no-project" scenario.

According to the 2-d modeling results, the 100-year flood event will cause widespread flooding in the Waipahu Town area for both with-levee and no-levee conditions. The overall flooding pattern is generally similar for both of these flooding scenarios. However, the area and maximum depth of inundation noticeably increases when the levee at the Waipahu Cultural Garden Park is removed from the model (because greater overland flows come from Waikele Stream upstream of Farrington Highway for the no-levee and no flood wall conditions). Total maximum inundated floodplain area is about 307 acres for the with-levee scenario and about 385 acres for the no-levee scenario. Additional areas flooded under the no-levee scenario include mostly open fields along both sides of Wailani Canal between Paiwa Street and Farrington Highway, residential areas on the east side of Wailani Canal between Farrington Highway and the Energy Corridor, and the large partially developed areas between Kapakahi Stream and Wailani Canal south of the Energy Corridor.

Overland flow paths determined for the no-levee, no-project conditions are schematically shown in Figure 6.3 at the end of this Chapter 6 as an example to demonstrate the overall water movement and flooding pattern in the study area. The major source of overland flooding is from overbank flow spilling from Waikele Stream upstream of Farrington Highway into the Waipahu Town area. The capacity of the Kapakahi Stream channel is insufficient to accommodate all local flows and becomes overwhelmed when overbank flows spill from Waikele Stream. This results in flooding depths on the order of several feet throughout the Waipahu Town area. Overland flows from Waikele Stream travel easterly through the Waipahu Cultural Garden Park and then through residential and commercial areas to Hanawai Circle. From there, some flood waters continue flowing easterly along Hikimoe Street and Farrington Highway toward Wailani Canal. Most of the overland flood waters overtop Farrington Highway and flow in a general southerly direction through commercial and mostly residential areas in a wide band between Kapakahi Street and Wailani Canal. Wailani Canal intercepts some of the overland flows. However, the capacity of Wailani Canal is insufficient to accommodate all the additional lateral inflow into the canal which results in flooding of residential areas on the east side of the canal, particularly in the vicinity of the Energy Corridor. Also partially inundated is the area on the west side of Kapakahi Stream.

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The flooding pattern in the Waipahu Town area is highly non-uniform due to complexities of the urban environment and complex interaction between in-channel and overland flows. Streets are the major conveyors of overland flow. However, the volume of overland flows is very large and the topographic relief is quite flat, which results in a widespread shallow flooding throughout the area.

After reaching the Energy Corridor, overland flows traveling through the Waipahu Town area between Kapakahi Stream and Wailani Canal fill the drainage canals running along the Energy Corridor, inundate low-lying lands on the south side of the Energy Corridor, and then are conveyed toward Pearl Harbor around elevated grounds via two major flow paths. A portion of the flood waters flows out of the Waipahu Town area through Pouhala Marsh and then meets with overland waters that spill onto the floodplain from Waikele Stream below the concrete-lined flood control channel. These flows combine and then flow south-westerly through mangrove forests and eventually spill into the West Loch of Pearl Harbor. The remaining flood waters exit the Waipahu Town area via the enlarged section of Wailani Canal and discharge into the Middle Loch of Pearl Harbor. Overland flows from Waikele Stream spread onto adjacent low-lying floodplain areas below the concrete flood control channel and inundate vast vegetated areas in the stream delta.

For the **with-levee, no-project** conditions, the area between the existing two buildings (where the proposed north entrance building for the Waipahu Transit Station will be constructed) remains mostly dry. Maximum overland flow depths in the vicinity of the proposed south entrance building (but without the entrance building in place) are within 1-3 ft. Flow depths along Farrington Highway range from less than 0.5 ft to over 3-5 ft. The deepest flow is observed in the area where overland flow crosses Farrington Highway between the proposed transit station and Kapakahi Stream.

For the **no-levee, no-project** conditions, the entire area near Farrington Highway between Kapakahi Stream and Wailani Canal is flooded. Maximum flow depths are up to 2-3 ft at the location of the future north entrance building and up to 3-5 ft at the location proposed for the construction of the south entrance building. Maximum depths along Farrington Highway range from around 5-7 ft in the vicinity of Kapakahi Stream to generally about 2-4 ft in the vicinity of Wailani Canal.

Maximum overland flow velocities simulated for existing (i.e. without project) with-levee and no-levee conditions are generally approximately 2-3 ft/s in the Waipahu Cultural Garden Park, 2-5 ft/s in the parking lot at Hanawai Circle, 2-6 ft/s along Farrington Highway, 1-3 ft/s in the built-up area south of Farrington Highway between Kapakahi Stream and Wailani Canal, 4-8 ft/s in the floodplain areas along Waikele Stream between the concrete-lined channel section and the Energy Corridor, 2-5 ft/s in the open floodplain areas to the south of the Energy Corridor, and 1-2 ft/s in vegetated floodplain areas along the shoreline. The area proposed for the construction of the north entrance building is blocked by existing buildings from the west, north, and east, and therefore represent mostly a backwater area. In the area of the proposed south building of the Waipahu Transit Station, flow spills over Farrington Highway with maximum flow velocities reaching 3-6 ft/s for the with-levee conditions and 6-9 ft/s for the no-levee conditions.

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According to the fine grid model results obtained for the **no-levee, no-project** conditions, maximum depths range from less than 0.5 ft to 1-2 ft at the location of the future north entrance building and are approximately 2-4 ft at the location of the south entrance building. Maximum stream velocities are generally less than 1-2 ft/s at the location of the south end of the north entrance building and are up to 4-9 ft/s at the location of the south entrance building. The fine grid model results are in good agreement with the results obtained from the coarse grid model. The small differences between the coarse grid and fine grid model results are due to the difference in resolution between the coarse grid and fine grid models. The fine grid model is capable of depicting topographic relief and the effects of buildings more accurately. The downstream boundary conditions are also specified differently in the fine grid versus coarse grid models. The coarse grid model provides a more accurate representation of flow boundary conditions, but has a lower resolution of land surface topography and buildings in the study area. On the contrary, the fine grid model better represents topographic features and buildings (including gaps between the buildings), but does not include details regarding tidal and backwater effects downstream of the Energy Corridor. Because of this, the coarse grid model provides a conservative estimate of the maximum depths and velocities, while the fine grid models provides more accurate details in the vicinity of the project features but likely underestimate flooding characteristics near the downstream model boundary at the Energy Corridor.

As is seen from Figure B-2 (in Appendix B), despite the significant differences in modeling approach and level of details, the flooding extent simulated for the floodplain area north of the Energy Corridor (for the no-levee conditions) by the more advanced 2-d coarse grid model is very similar to that shown on the FEMA's Flood Insurance Rate Map (FIRM). However, south of the Energy Corridor, the 2-d model shows significant flooding (to depths of up to 2-5 ft) in the area between Kapakahi Stream and Wailani Canal. The same area is shown to be dry on the FEMA's FIRM. Comparison of Figure 2.7 (in Chapter 2) and Figure B-1 (in Appendix B) shows that maximum flood elevations simulated by the 2-d model are up to about 1-3 ft higher than FEMA's Base Flood Elevations (BFEs) in the Waipahu Cultural Garden Park area and along Kapakahi Stream in the vicinity of Farrington Highway, are up to 1-2 ft higher than FEMA's BFEs between Kapakahi Stream and Wailani Canal north of the Energy Corridor, and up to 2-3 ft higher than FEMA's BFEs between Waikele Stream and Kapakahi Stream south of the Energy Corridor. These observed differences in flood elevations are due to the greater inflow from Waikele Stream to Waipahu Town in this study compared to what FEMA used (see Table 5.1 in Chapter 5), as well as a more accurate representation of the physical process of flood wave propagation over the floodplain in the 2-d model compared to the 1-d, steady state approach used by FEMA.

6.3.2. Project Conditions

Maximum inundation depths and velocity vectors simulated by the coarse grid model in the vicinity of the project station for the **with-levee, with-project** conditions are shown in Appendix C in Figure C-1 (original project design), Figure C-2 (Project Alternative 1), Figure C-3 (Project Alternative 2), and Figure C-4 (Project Alternative 3).

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Maximum inundation depths and velocity vectors computed by the coarse grid model in the vicinity of the project station for the **no-levee, with-project** conditions are shown in Appendix D in Figure D-1 (original project design), Figure D-2 (Project Alternative 1), Figure D-3 (Project Alternative 2), and Figure D-4 (Project Alternative 3). The fine grid model maximum depths and velocity vectors simulated for the no-levee, with-project conditions are shown in Appendix D in Figure D-5 (original project design), Figure D-6 (Project Alternative 1), Figure D-7 (Project Alternative 2), and Figure D-8 (Project Alternative 3).

To evaluate the effects of the original project and the three project alternatives on flooding characteristics, differential maps were developed that show differences in maximum depths and maximum velocities between the project conditions model results and corresponding existing conditions model results. Differential maps from the coarse grid models that show the project impacts on maximum inundation depths for the **with-levee** conditions are shown in Appendix E in Figure E-1 (original project design), Figure E-2 (Project Alternative 1), Figure E-3 (Project Alternative 2), and Figure E-4 (Project Alternative 3). The project effects on maximum flow velocities obtained for the with-levee conditions are shown in Appendix E in Figure E-5 (original project design), Figure E-6 (Project Alternative 1), Figure E-7 (Project Alternative 2), and Figure E-8 (Project Alternative 3).

Differential maps showing the effect of various project designs on maximum depths for the **no-levee** conditions are shown for the coarse grid models in Appendix F in Figure F-1 (original project design), Figure F-2 (Project Alternative 1), Figure F-3 (Project Alternative 2), and Figure F-4 (Project Alternative 3) and for the fine grid models in Figure F-5 (original project design), Figure F-6 (Project Alternative 1), Figure F-7 (Project Alternative 2), and Figure F-8 (Project Alternative 3). Differential maps based on the coarse grid model results and showing project effects on maximum flow velocity for the no-levee conditions are presented in Appendix F in Figure F-9 (original project design), Figure F-10 (Project Alternative 1), Figure F-11 (Project Alternative 2), and Figure F-12 (Project Alternative 3). Red colored model cells in these differential depth maps show presently open areas where project entrance buildings will be located (thus there is a negative depth and velocity difference because the buildings displaced the water).

Increases in 100-year flood maximum depths (water surface elevations) and maximum velocities caused by the project structures in the vicinity of the Waipahu Transit Station are summarized in Table 6.2. This table combines results obtained from the coarse grid and fine grid models and shows only detrimental project impacts (that cause depth and velocity increases). All the project impacts (including both increases and reductions in depths and velocities) are discussed in detail in subsequent sections of this report.

Original Project Design

Comparison of the results obtained for the with-project and no-project scenarios indicates that the proposed project (original design and the three alternatives) does not have a noticeable effect on the total inundated area. However, the original project design (which includes construction of the north and south entrance buildings on both sides of Farrington Highway) creates a significant local impact on water levels, flow depths and velocities in the immediate vicinity of the transit

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station. According to the coarse grid model results, the original project increases maximum inundation depths along Farrington Highway in the vicinity of the Waipahu Transit Station by 0.1-0.4 ft for the with-levee conditions (see Figure E-1 in Appendix E) and by 0.2-0.5 ft for the no-levee conditions (see Figure F-1 in Appendix F). At the same time, the project reduces maximum inundation depths immediately south-east of the south entrance building by 0.2-0.5 ft for both with-levee and no-levee conditions. Maximum flow velocities reduce immediately west of the project station and south-east of the south entrance building by 1-2 ft/s for both with-levee and no-levee conditions (see Figure E-5 in Appendix E and Figure F-9 in Appendix F). Along Farrington Highway between the north and south entrance buildings, maximum velocities increase by 0.5-2 ft/s for the with-levee and by 1-3 ft/s for the no-levee conditions. According to the 2-d model results, the effect on depths and velocities of the individual guideway support columns due to their relatively small size is not detectable (less than the FLO-2D model accuracy).

The results obtained for the original project design for the no-levee conditions from the coarse grid model are generally supported by the fine model results. According to the fine grid model, for the no-levee conditions maximum inundation depths increase along Farrington Highway between the station entrance buildings by 0.1-0.4 ft and reduce immediately south-east of the south entrance building by 0.1-0.3 ft (see Figure F-5 in Appendix F). The fine grid model also indicates that under project conditions there will be up to 0.3-0.5 ft increase in maximum flow depths in the narrow gap between the north entrance building and adjacent existing structure. Under the no-project conditions, this gap is blocked from the north by a small building and there is no transitional flow here. Under project conditions (original design), this small building will be removed, which will open the gap and will allow overland waters to flow from Hikimoe Street to Farrington Highway through this gap. The impact values obtained from the fine grid model are generally similar to those obtained from the coarse grid model, though the impact area is somewhat smaller.

The simulated impacts of the original project design can be explained by the following:

(1) Project features, particularly entrance buildings, occupy more surface area and thereby reduce floodplain storage capacity, which inevitably affects local flooding characteristics in the surrounding area.
(2) Project entrance buildings due to their relatively large size constrict highly dynamic overland flow along Farrington Highway laterally by approximately 25%, which increases local water levels and changes flow velocities in the vicinity of the project buildings.
(3) The south entrance building is located in a deep area with the highest flow velocities and creates a significant dynamic obstruction to flows spilling over Farrington Highway at this location. As a result, flood waters are ponded in front of the south building, which increases flow depths along the highway and at the same time reduces depths in the sheltered area behind the proposed station building.

Therefore, the original project design does not comply with FEMA's and DPP's "no-rise" criteria, so additional project alternatives were evaluated.

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Project Alternative 1

The simulation results obtained from the coarse grid models indicate that removal of the entire south entrance building will eliminate the detrimental impacts of the project on inundation depths for both with-levee and no-levee conditions (see Figure E-2 in Appendix E and Figure F-2 in Appendix F). Red colored model cells in these figures show wet areas that are presently open, but will be occupied by the north entrance building. No impact on flow velocity is detected for the with-levee conditions (see Figure E-6 in Appendix E). The only impact of this project configuration on flow velocity was detected for the no-levee conditions, where maximum velocity slightly increases by less than 0.5 ft/s immediately south of the north entrance building (see Figure F-10 in Appendix F). This velocity increase is likely caused by slightly reduced conveyance due to the north entrance building.

The fine grid model indicates that under the no-levee conditions there will be a 0.1-0.5 ft increase in maximum depths in the narrow gap between the north entrance building and adjacent structure (see Figure F-6 in Appendix F). This gap is presently blocked by a small building and will be open under the project conditions should the original design of the north entrance building be implemented. The fine grid model also shows more than 0.5 ft depth increase in one cell south of Farrington Highway, where a small building is currently located. In the present study, it was assumed that this small building will be removed under the project conditions. To completely eliminate the impact of this project configuration on 100-year flood inundation depths, the gap along the north entrance building should be blocked (to preserve the present day conditions) and the small building south of Farrington Highway should be preserved or replaced by a structure with similar maximum external ground floor dimensions measuring approximately 40 ft east to west by 20 ft north to south.

Project Alternative 2

For the project alternative which includes a reduced footprint of the south entrance building, the coarse grid models indicate that such a project design will still have noticeable impact on local depths and velocities of the overland flow, although they are less significant than the impact from the original project design. The size of the south entrance building in this alternative is approximately half of the original design, but it is located in an extremely unfavorable area of highly dynamic flow and still produces significant obstruction to flows spilling over Farrington Highway at this location. For this scenario, maximum inundation depths increase at Farrington Highway between the station entrance buildings by 0.1-0.3 ft for the with-levee conditions (see Figure E-3 in Appendix E) and by 0.1-0.4 ft for the no-levee conditions (see Figure F-3 in Appendix F). At the same time, maximum depths reduce under the project conditions immediately east of the south entrance building (in the sheltered area) by 0.1-0.4 ft for both with-levee and no-levee conditions. Maximum stream velocities reduce immediately west of the transit station and south-east of the south entrance building by 0.5-2 ft/s for both with-levee and no-levee conditions (see Figure E-7 in Appendix E and Figure F-11 in Appendix F). Along Farrington Highway between the entrance buildings, maximum velocities increase by 0.5-2 ft/s for the with-levee and by 1-3 ft/s for the no-levee conditions.

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According to the fine grid model, for the no-levee conditions and with the reduced south entrance building maximum depths increase by 0.1-0.3 ft at Farrington Highway between the entrance buildings, increase by 0.3-0.5 ft in the gap along the north entrance building, and reduce by 0.1-0.3 ft immediately west of the reduced south building (see Figure F-7 in Appendix F). The results from the fine grid model are very similar to those from the coarse model.

Project Alternative 3

This project alternative includes (1) an elevated (above the 100-year flood level) north entrance plaza that fully fills the gap between the existing buildings, (2) an elevated (above the 100-year flood level) ground floor of the north entrance building, and (3) a new south entrance building in the same approximate location with the same maximum external ground floor dimensions (approximately 40 ft east to west by 20 ft north to south) of the existing building it replaces. According to the FLO-2D model results obtained for the existing conditions, the maximum 100-year flood elevation at this location is approximately 13.5 ft (see Figure B-1 in Appendix B). The simulation results obtained from the coarse grid models indicate that this project alternative will have no detectable impacts on 100-year inundation depths or flood levels (see Figure E-4 in Appendix E and Figure F-4 in Appendix F). Red colored model cells on these figures show wet areas that are presently open, but will be occupied by the north entrance building and raised plaza. There is no impact on flow velocity for the with-levee conditions (see Figure E-8 in Appendix E) and a slight (by less than 0.5 ft/s) increase in maximum velocity only at the south end of the north entrance building for the no-levee condition (see Figure F-12 in Appendix F). The fine grid model results obtained for the no-levee conditions confirm the coarse grid model results and show that Project Alternative 3 will have no detectable (within the inherent accuracy of the model) effect on inundation depths or water levels in the Waipahu Town area (see Figure F-8 in Appendix F). Therefore, results from these analyses provide PB's design team with a better understanding of local flow conditions and sufficient information to prepare a safe project alternative that should comply with FEMA's and DPP's "no-rise" policy. A supplemental report that explicitly addresses the no-rise and no net loss of floodplain storage requirements for DPP and FEMA was also prepared by NHC, titled "No Rise Analysis for Waipahu Transit Center Station and HHCTCP Guideway Columns – Waikele, Kapakahi and Wailani Streams" (NHC April 30, 2010).

6.4. Sensitivity Analyses

FLO-2D relies on a number of user-defined input parameters to perform hydrodynamic computations. Recommended value ranges are available for these parameters, but selection of the final value is dependent on the specific application and the modeler's judgment. The validity of the selected values is usually checked by comparing model results with measured flow data. If necessary, the model parameters are then adjusted to obtain the best agreement between the modeled and measured data. This process of adjusting model input parameters is called "model calibration".

One of the most important parameters for flood simulations is surface roughness (Manning's roughness coefficient). However, no detailed flood inundation calibration data are available for

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the study area. Surface roughness was estimated from field observations, aerial imagery, available references, FLO-2D user's manual recommendations, and previous modeling experience. The roughness values used in the models ranged from 0.015 (concrete-lined channel sections) to 0.15 (mangrove forests), with the value of 0.04 assigned to all grid cells within urban and open floodplain areas.

To determine the effect of changing surface roughness coefficient on simulated flooding characteristics, sensitivity runs were performed using roughness coefficients increased by a factor of two relative to those used for the main production runs. The sensitivity runs were conducted for the no-levee, no-project and no-levee, with-project scenarios as examples. Variations in results due to the change in modeling parameters were assessed by comparing computed maximum inundated areas, inundation depths, and overall flooding patterns.

The sensitivity runs show that a two-fold increase in roughness coefficients results in a 14% increase in the maximum floodplain inundated area (from 385 acres to 439 acres), although the overall flooding pattern remains generally the same. Additional flooded areas are mostly located south of the Energy Corridor and on the east side of Wailani Canal. With the higher roughness values, maximum inundation depths increase in many areas by more than 0.5-1.5 ft, which is quite significant. However, the relative difference showing the project effect on computed flow depths remains practically unchanged. Depth differences obtained for the 100-year flood for no-levee conditions for original project design are shown at the end of this chapter in Figure 6.4. Comparison of this figure with Figure F-1 in Appendix F (showing depths differences from the main production runs for the same no-levee, with-project conditions) indicates that although varying surface roughness significantly affects absolute depth values, it appears to have no effect on the simulated relative depth differences between the project and existing conditions.

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Table 6.1. Summary of FLO-2D models used for final production runs and sensitivity runs.

Model	Grid cell size (ft)	Modeling scenarios
Existing conditions		
Test25	50	With levee, no project
Test25NL	50	No levee, no project
SD1_Test1NL	10	No levee, no project
Project conditions (original design)		
Test25_P	50	With levee, with project
Test25NL_P	50	No levee, with project
SD1_Test1NL_P	10	No levee, with project
Project alternative 1		
Test25_P_North	50	With levee, with north entrance building
Test25NL_P_North	50	No levee, with north entrance building
SD1_Test1NL_P_North	10	No levee, with north entrance building
Project alternative 2		
Test25_P_North_PB	50	With levee, with north entrance building and reduced south entrance building
Test25NL_P_North_PB	50	No levee, with north entrance building and reduced south entrance building
SD1_Test1NL_P_PB	10	No levee, with north entrance building and reduced south entrance building
Project alternative 3		
Test25_P_North_NHC	50	With levee, with elevated north entrance plaza and north entrance building and minimum south building footprint
Test25NL_P_North_NHC	50	No levee, with elevated north entrance plaza and north entrance building and minimum south building footprint
SD1_Test1NL_P_North_NHC	10	No levee, with elevated north entrance plaza and north entrance building and minimum south building footprint
Sensitivity runs		
Test25NL_n2	50	No levee, no project, double roughness
Test25NL_P_n2	50	No levee, with project (original design), double roughness

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Table 6.2. Simulated increases in 100-year flood maximum depths (water surface elevations) and maximum velocities caused by project structures (combined results from coarse grid and fine grid models).

Project alternative	Included project features	Increase in maximum flow depth (water surface elevation)		Increase in maximum flow velocity	
		With levee*	No levee**	With levee*	No levee**
Original design	North and south entrance buildings, guideway support columns	0.1-0.4 ft	0.2-0.5 ft	0.5-2 ft/s	1-3 ft/s
Project Alternative 1	North entrance building only, guideway support columns	No rise	0.1-0.5 ft (in a gap along north entrance building)	No increase	<0.5 ft/s
Project Alternative 2	North entrance building, reduced south entrance building, guideway support columns	0.1-0.3 ft	0.1-0.5 ft	0.5-2 ft/s	1-3 ft/s
Project Alternative 3	North entrance building with elevated plaza, minimum south entrance building, guideway support columns	No rise	No rise	No increase	<0.5 ft/s

* With Waikele Stream levee at Waipahu Cultural Garden Park and with flood walls along Wailani Canal;
** Without Waikele Stream levee at Waipahu Cultural Garden Park and without flood walls along Wailani Canal.

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Data Sources: USGS Color Aerial Imagery, 2004. ESRI Roads, 2008. USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

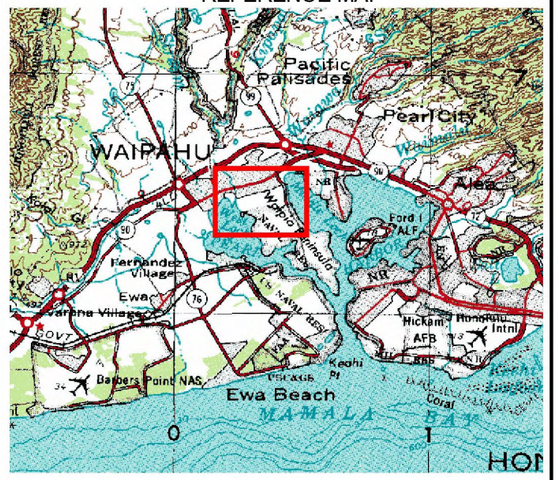
Study Area Map (Coarse Grid Model)

Scale - 1:12,000 1 inch = 1,000 feet

0 500 1,000 2,000 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Roads
- Streams & Canals
- Levee
- Energy Corridor

Figure 6.1 - Study Area Map (Coarse Grid Model)

Data Sources: USGS Color Aerial Imagery, 2004. ESRI Roads, 2008. USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

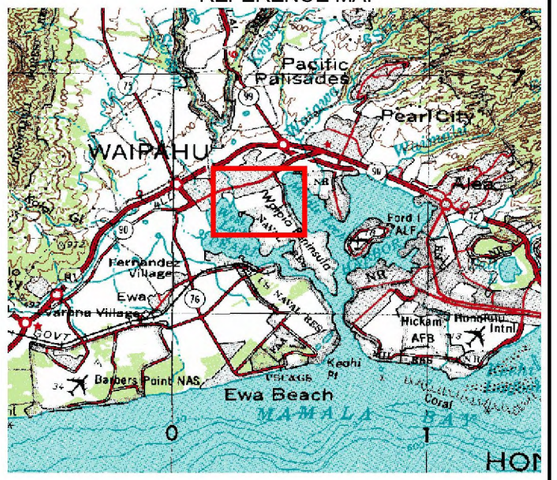
Study Area Map (Fine Grid Model)

Scale - 1:12,000 1 inch = 1,000 feet

0 500 1,000 2,000 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Roads
- Streams & Canals
- Levee
- Energy Corridor

Figure 6.2 - Study Area Map (Fine Grid Model)

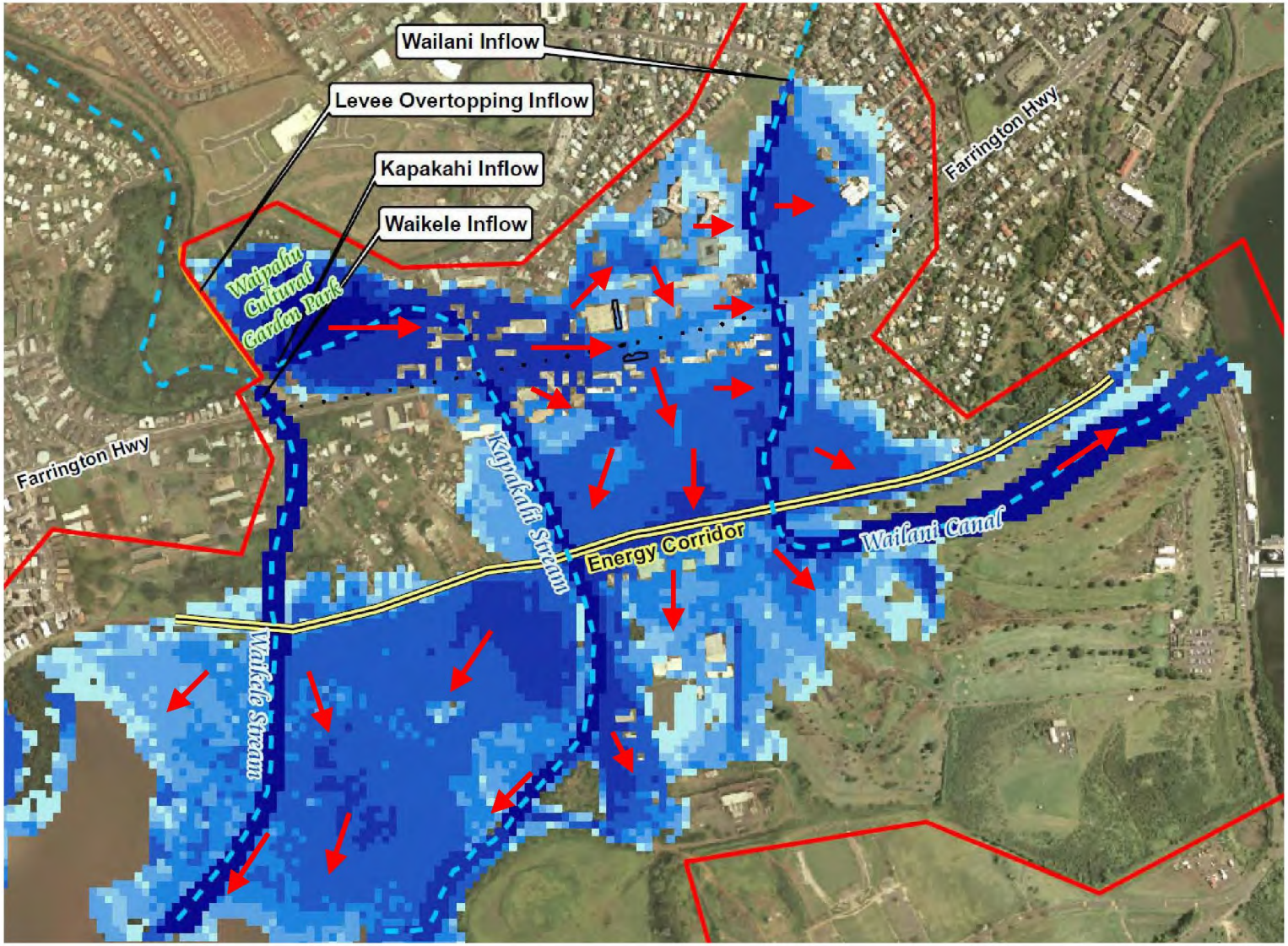


Figure 6.3. Overall flooding pattern in study area (no-levee, no-project).

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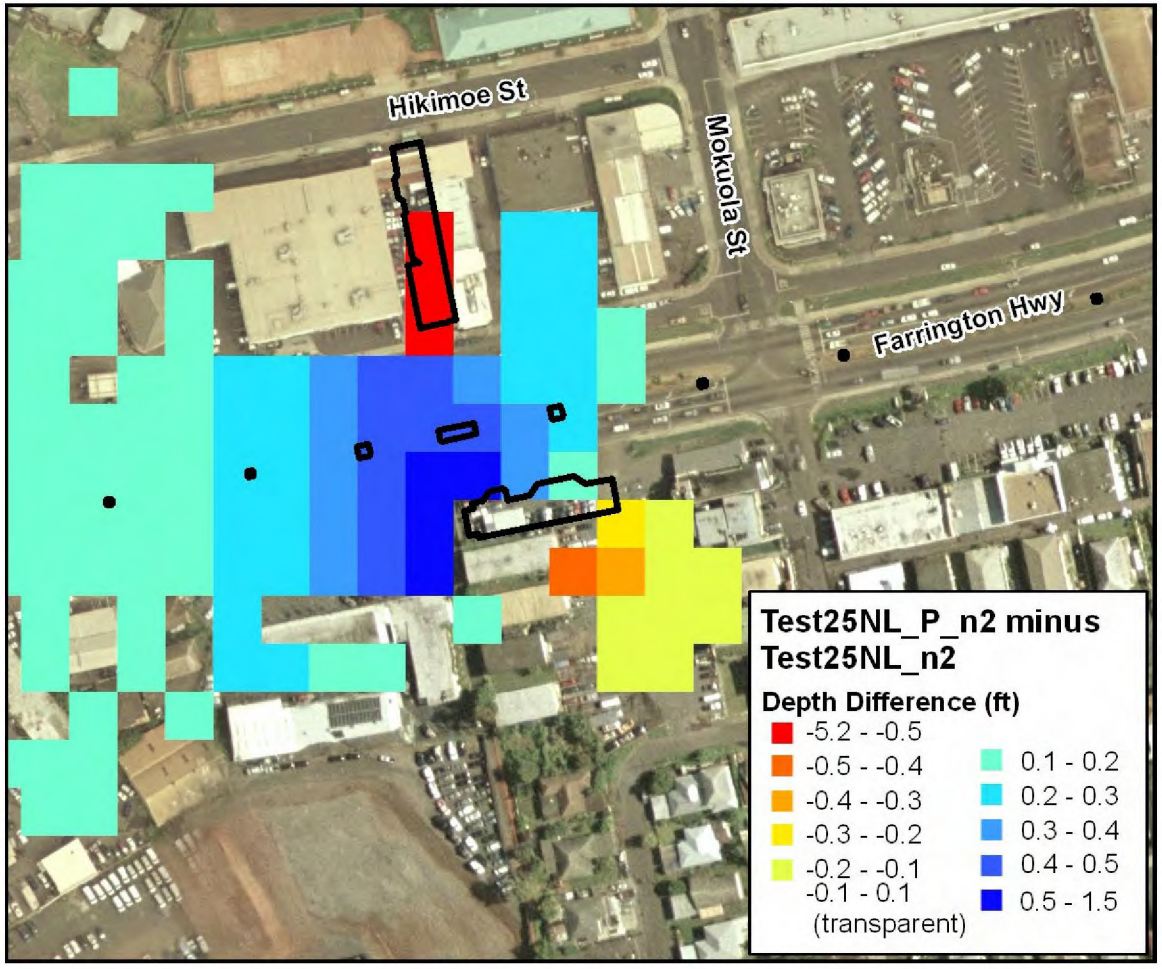

Figure 6.4. Project-related changes in 100-year flood maximum depth in vicinity of project station with doubled surface roughness.

7. SUMMARY

The main objectives of this advanced study undertaken by Northwest Hydraulic Consultants (NHC) were to evaluate existing flow conditions in the Waipahu floodplain area, assess impacts from the proposed Honolulu High-Capacity Transit Corridor Project (HHCTCP) features on 100-year flood levels within the Waipahu area, and to devise a "no-rise" mitigation alternative. NHC's work involved field inspection of the project area, collection of the data needed for computer simulation of the flooding in the vicinity of the project Waipahu Transit Station, hydrologic analysis, one-dimensional (1-d) hydraulic modeling of in-stream flows (using HEC-RAS model), two-dimensional (2-d) modeling of overland flooding (using FLO-2D model), and evaluation of various project alternatives. Results from these models were used to better understand local flow conditions (depths and velocities) that may affect the project and to help PB's design team to develop project designs that are safe and comply with DPP and FEMA floodplain development regulations.

The study area encompasses approximately 1.6 square miles of heavily urbanized Waipahu Town and open lands between the West and Middle Lochs of Pearl Harbor. The stream network in the study area is comprised by Waikele Stream, Kapakahi Stream, and Wailani Canal, as well as a few small drainage canals. During heavy winter rainfalls, the Waipahu Town area is subject to frequent flooding from the local streams. The major source of overland flooding is Waikele Stream. During large flood events, flow from Waikele Stream combines with flows from the other smaller flooding sources which include local runoff to Kapakahi Stream and Wailani Canal.

A field inspection of the study area was completed on January 15, 2009 and involved documentation of channel and floodplain conditions of the streams in the area, pertinent hydraulic features, locations where ponding and ineffective flow are likely, and high water mark locations left during the December 11, 2008 flood event. Following the site inspection, NHC developed a detailed program for collecting channel survey data needed to support model development and hydraulic analyses. The survey was conducted by ControlPoint Surveying, Inc. (ControlPoint) in the spring of 2009. Additional topographic data used in this study included 1-ft contour interval Light Detection and Ranging Data (LiDAR)-based topography data within the HHCTCP corridor obtained by John Chance Land Surveys, Inc. (JCLS) in 2007 (official data for the project) and 5 ft by 5 ft grid FEMA LiDAR data developed in 2006 for the FEMA Hurricane Study and covering the entire island coast including Waipahu Town area. For use in this study, NHC developed a modified topographic data set in which the 2006 FEMA LiDAR data along the project corridor were replaced with the 2007 project topography data.

Up-to-date hydrological data were developed to support hydraulic analysis of potential impacts of the proposed HHCTCP facilities on 100-year flood levels in the Waipahu Town area. Based on the most recent streamflow data, NHC updated 100-year peak flow values and developed 100-year design hydrographs for Waikele Stream, Kapakahi Stream, and Wailani Canal for use in the 1-d and 2-d models.

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Using present day topographic data, channel cross sections, and hydrologic information, NHC developed up-to-date 1-d HEC-RAS models of Waikele Stream, Kapakahi Stream, and Wailani Canal. The main intent of these 1-d HEC-RAS models was to support the development and application of the more sophisticated 2-d FLO-2D model of the entire Waipahu floodplain area. The HEC-RAS model of Waikele Stream was used to simulate flow characteristics for with-levee conditions (with the existing levee between Waikele Stream and the Waipahu Cultural Garden Park in place) and for no-levee conditions (to comply with FEMA's regulations which do not recognize this levee as providing flood protection to the area). Hydrographs simulated from the 1-d model for Waikele Stream in-channel flows below Farrington Highway and breakout flows from Waikele Stream to the Waipahu area upstream of Farrington Highway were used to specify upstream inflow boundary conditions for the 2-d model developed for the Waipahu floodplain area. HEC-RAS depth-discharge results for hydraulic structures on Waikele Stream, Kapakahi Stream, and Wailani Canal were used to specify rating tables at these structures in the 2-d floodplain model.

A set of coarse grid and fine grid 2-d FLO-2D models of the Waipahu Town area was developed to simulate the 100-year flooding in the vicinity of the proposed Waipahu Transit Station. Coarse grid models (with 50 ft grid cell size) were used to simulate area-wide flooding characteristics, while fine grid models (with 10 ft grid cell size) were used to simulate details of flooding in the vicinity of the project structures. Flood simulations were conducted for existing and project conditions (with the Waikele Stream levee and Wailani Canal flood walls and without the levee and flood walls). Results of these simulations indicated that the 100-year flood event will cause widespread flooding in the Waipahu Town area for both with-levee and no-levee conditions. Overall flooding pattern is generally similar for both these flooding scenarios. Flood waters breaking out of Waikele Stream upstream of Farrington Highway will flow in the easterly direction north of Farrington Highway, overtop Farrington Highway, and then travel southerly in a wide band between Kapakahi Stream and Wailani Canal. Capacities of Kapakahi Stream and Wailani Canal are insufficient to accommodate overland inflows from Waikele Stream which will result in significant overland flooding along both these streams.

According to the FLO-2D model results, construction of the originally designed project station features will reduce local floodplain storage capacity, confine flow along Farrington Highway, create significant obstruction to the swift overland flow (specifically adjacent to the south entrance building), and thereby will increase water levels along Farrington Highway by up to 0.1-0.4 ft for with-levee conditions and by 0.2-0.5 ft for no-levee conditions. According to the 2-d model results, placement of the 6-foot diameter guideway support columns along the Farrington Highway median will not affect flood depths (within the accuracy of the FLO-2D model) due to the relatively small size of the columns, shallow flow depths and wide spacing between columns.

Because the original project designs for the north and south entrance buildings caused a rise in water levels, three additional project alternatives were evaluated using the NHC-developed FLO-2D models: (Alt 1) original north entrance building only, with the south entrance building removed completely, (Alt 2) north entrance building with the south entrance building footprint reduced to approximately half it's original size, and (Alt 3) includes an elevated (above the 100-year flood level) north entrance plaza and the north entrance building that fully fills the gap

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between the existing adjacent buildings and a new south entrance building in the same approximate location with the same maximum external ground floor dimensions (approximately 40 ft east to west by 20 ft north to south) of the existing building it replaces.

The FLO-2D model results indicate that Project Alternative 1 (no south entrance building) will produce no detectable impact on flood levels along Farrington Highway. However, as presently proposed this alternative allows transit flow through a narrow gap (presently blocked by existing buildings) along the north entrance building thereby increasing flow depths in this gap by 0.1-0.5 ft which violates the "no rise" policy. The rise caused by flow between the buildings can be eliminated by including an elevated plaza (above the 100-year flood level)all around the north entrance building.

According to the FLO-2D model results, the effect of Project Alternative 2 will be quite significant. The size of the south entrance building in the second alternative is approximately half of the original design, but it will be located in an extremely unfavorable area of highly dynamic flow and will produce significant obstruction to overbank flows spilling over Farrington Highway at this location. With Project Alternative 2, the maximum inundation depths will increase at Farrington Highway between the station entrance buildings by 0.1-0.3 ft for with-levee conditions and by 0.1-0.5 ft for no-levee conditions.

FLO-2D model results show that Project Alternative 3 will have no detectable impact (within the inherent accuracy of the model) on 100-year flood elevations in the Waipahu Town area. The model shows no change in flooding pattern, inundation extent, and maximum inundation depths (within the inherent accuracy of the model). Therefore, results from these analyses provide PB's design team with a clearer understanding of local flow conditions and sufficient information to prepare a safe project alternative that should comply with FEMA's and DPP's "no-rise" policy. The preferred alternative includes a new north entrance building to be constructed at the originally selected station location, but the plaza and ground floor of the north entrance building should be raised above the 100-year flood elevation (above 13.5 ft HILOCAL). The raised plaza should occupy the entire width between the existing structures to match the effects of the presently blocked north-south gap between the existing structures. The south entrance building should be constructed in the same approximate location with the same maximum external ground floor dimensions (approximately 40 ft east to west by 20 ft north to south) of the existing building it replaces. However, if it is decided to not construct the south entrance building at all, the existing small building should be left in place (or replaced by a structure with similar maximum external ground floor dimensions) to preserve present day flow conditions. A supplemental report that explicitly addresses the no-rise and no net loss of floodplain storage requirements for DPP and FEMA was also prepared by NHC, titled "No Rise Analysis for Waipahu Transit Center Station and HHCTCP Guideway Columns – Waikele, Kapakahi and Wailani Streams" (NHC April 30, 2010).

Peak overbank inflow from Waikele Stream to the Waipahu area used in this study is higher than that used by FEMA in their flood inundation study. Therefore, estimated project impacts are also conservative, which adds a safety factor to the results of this study.

AR00041444

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AR00041445

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APPENDIX. 100-YEAR FLOOD INUNDATION MAPS

(for description of models refer to Table 6.1 in Chapter 6)

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Appendix A. Existing conditions flooding (with levee, no project)

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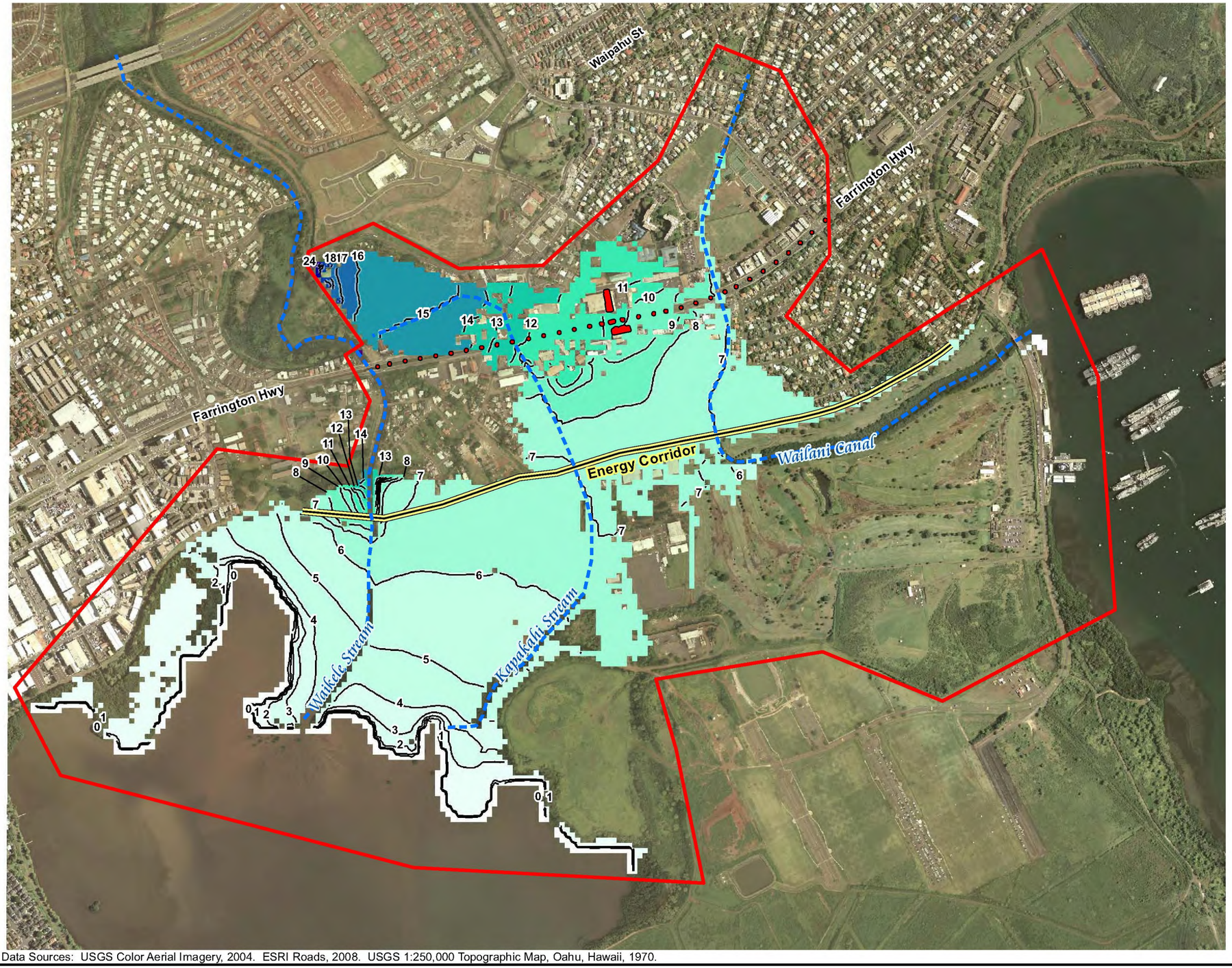


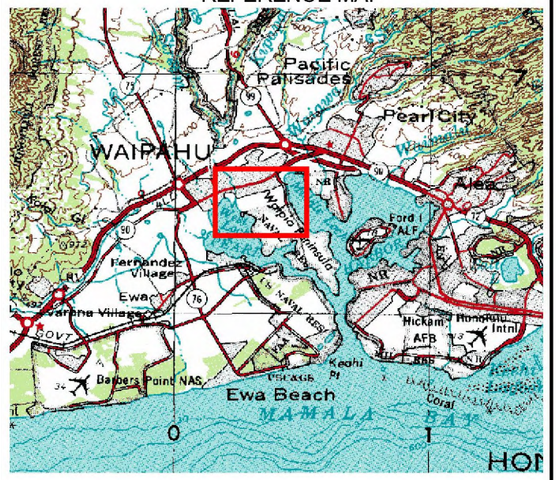
100-year Flood Maximum Floodplain Water Surface Elevations (Test25)

Scale - 1:12,000 1 inch = 1,000 feet

0 500 1,000 2,000 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



LEGEND

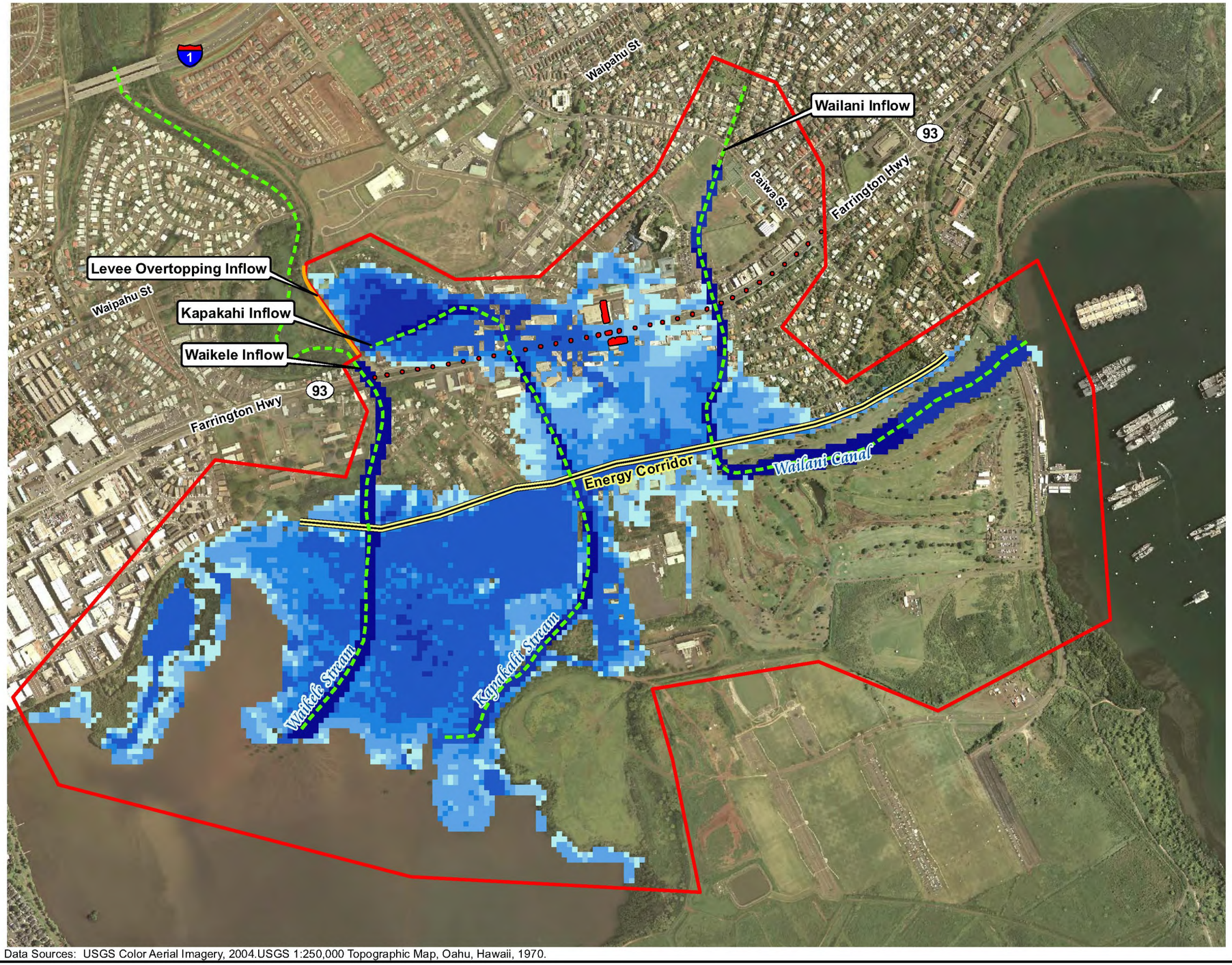
- Model Area
- Project Buildings and Piers/Columns
- Streams & Canals
- Energy Corridor
- WSE Contours (1ft)

Water Surface Elevation (in ft)

- Less than 0ft
- 0 - 2
- 2 - 4
- 4 - 6
- 6 - 8
- 8 - 10
- 10 - 12
- 12 - 14
- 14 - 16
- 16 - 18
- 18 - 20
- 20 - 22
- 22 - 24
- 24 +

Data Sources: USGS Color Aerial Imagery, 2004. ESRI Roads, 2008. USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

Figure A-1 - 100-year Flood Maximum Floodplain Water Surface Elevations: With Levee, No Project (Coarse Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

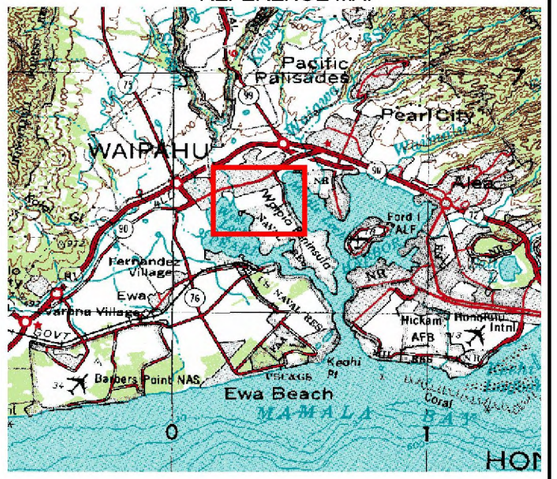
100-year Flood Maximum Inundation Depths (Test25)

Scale - 1:12,000 1 inch = 1,000 feet

0 500 1,000 2,000 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



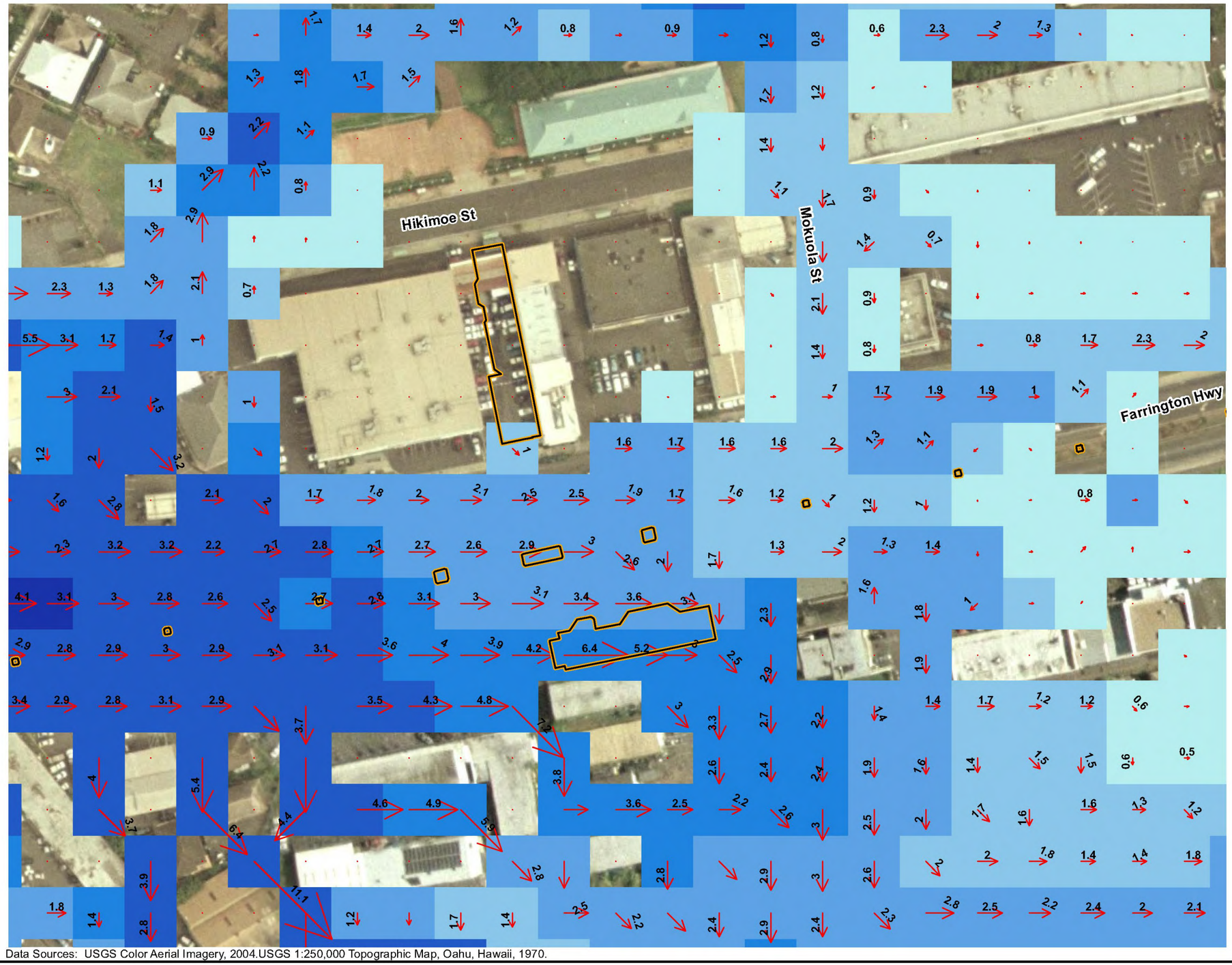
LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Streams & Canals
- Energy Corridor
- Levee

Max Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 2.0
- 2.0 - 3.0
- 3.0 - 5.0
- 5.0 - 9.0
- 9.0 +

Figure A-2 - 100-year Flood Maximum Inundation Depths: With Levee, No Project (Coarse Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

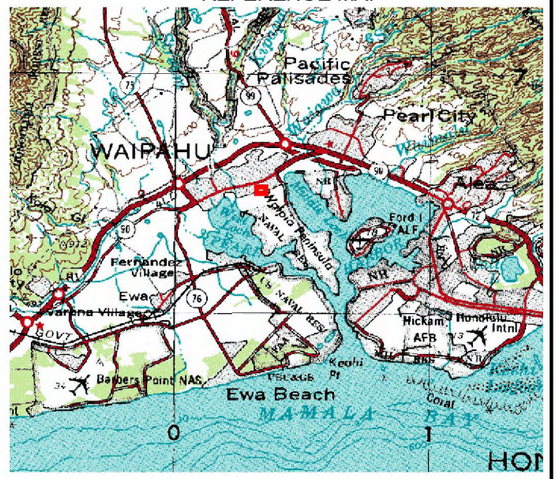
100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station (Test25)

Scale - 1:1,200 1 inch = 100 feet

0 50 100 200 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Streams & Canals
- Velocity Vector (ft/s)

Max Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 2.0
- 2.0 - 3.0
- 3.0 - 5.0
- 5.0 - 9.0
- 9.0 +

Figure A-3 - 100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station: With Levee, No Project (Coarse Grid Model)

Appendix B. Existing conditions flooding (no levee, no project)

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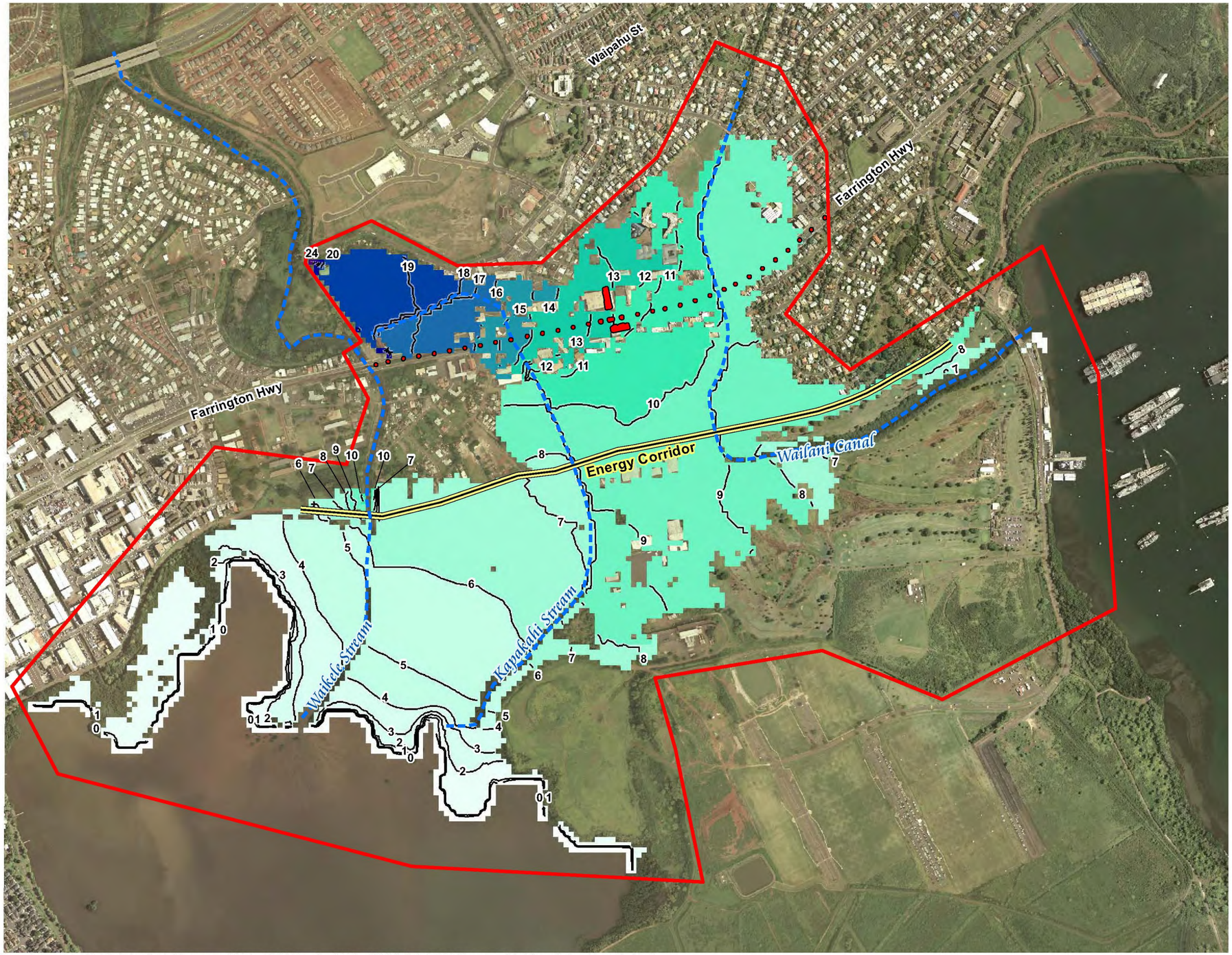


Data Sources: USGS Color Aerial Imagery, 2004. ESRI Roads, 2008. USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

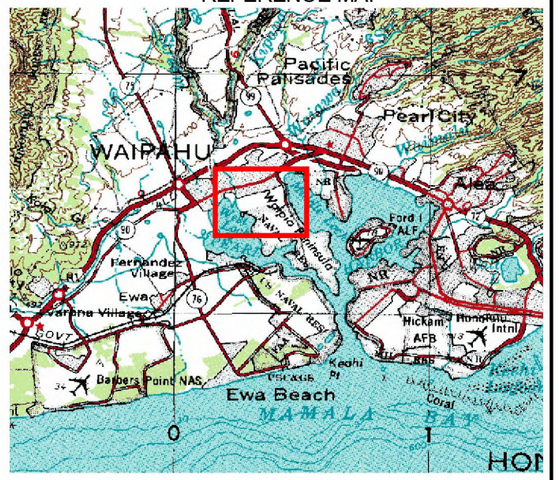
100-year Flood Maximum Floodplain Water Surface Elevations (Test25NL)

Scale - 1:12,000 1 inch = 1,000 feet

0 500 1,000 2,000 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



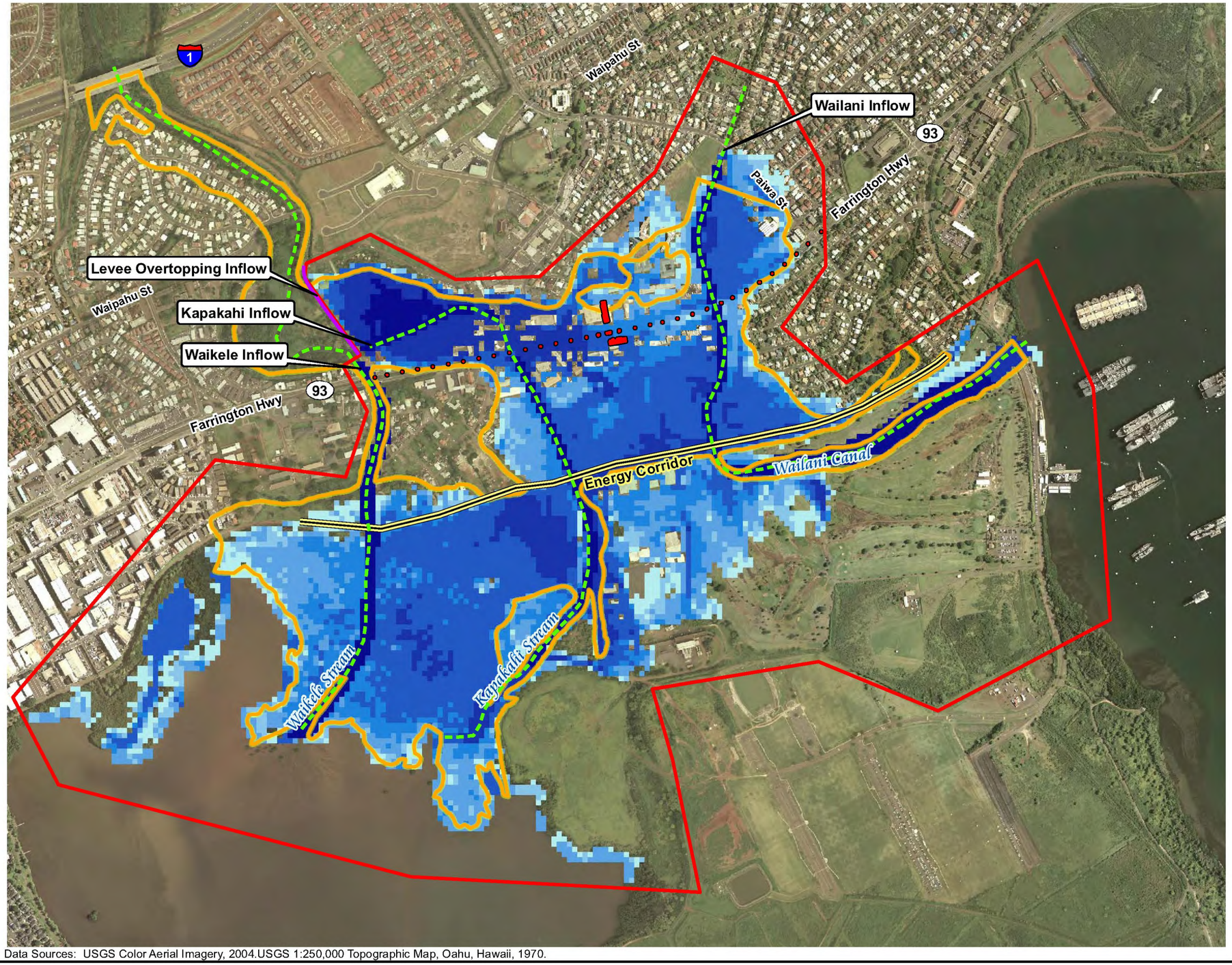
LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Streams & Canals
- Energy Corridor
- WSE Contours (1ft)

Water Surface Elevation (in ft)

- Less than 0ft
- 0 - 2
- 2 - 4
- 4 - 6
- 6 - 8
- 8 - 10
- 10 - 12
- 12 - 14
- 14 - 16
- 16 - 18
- 18 - 20
- 20 - 22
- 22 - 24
- 24 +

Figure B-1 - 100-year Flood Maximum Floodplain Water Surface Elevations: No Levee, No Project (Coarse Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

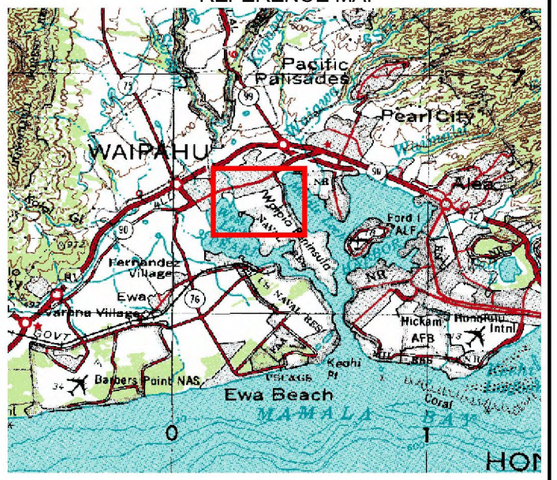
100-year Flood Maximum Inundation Depths (Test25NL)

Scale - 1:12,000 1 inch = 1,000 feet

0 500 1,000 2,000 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



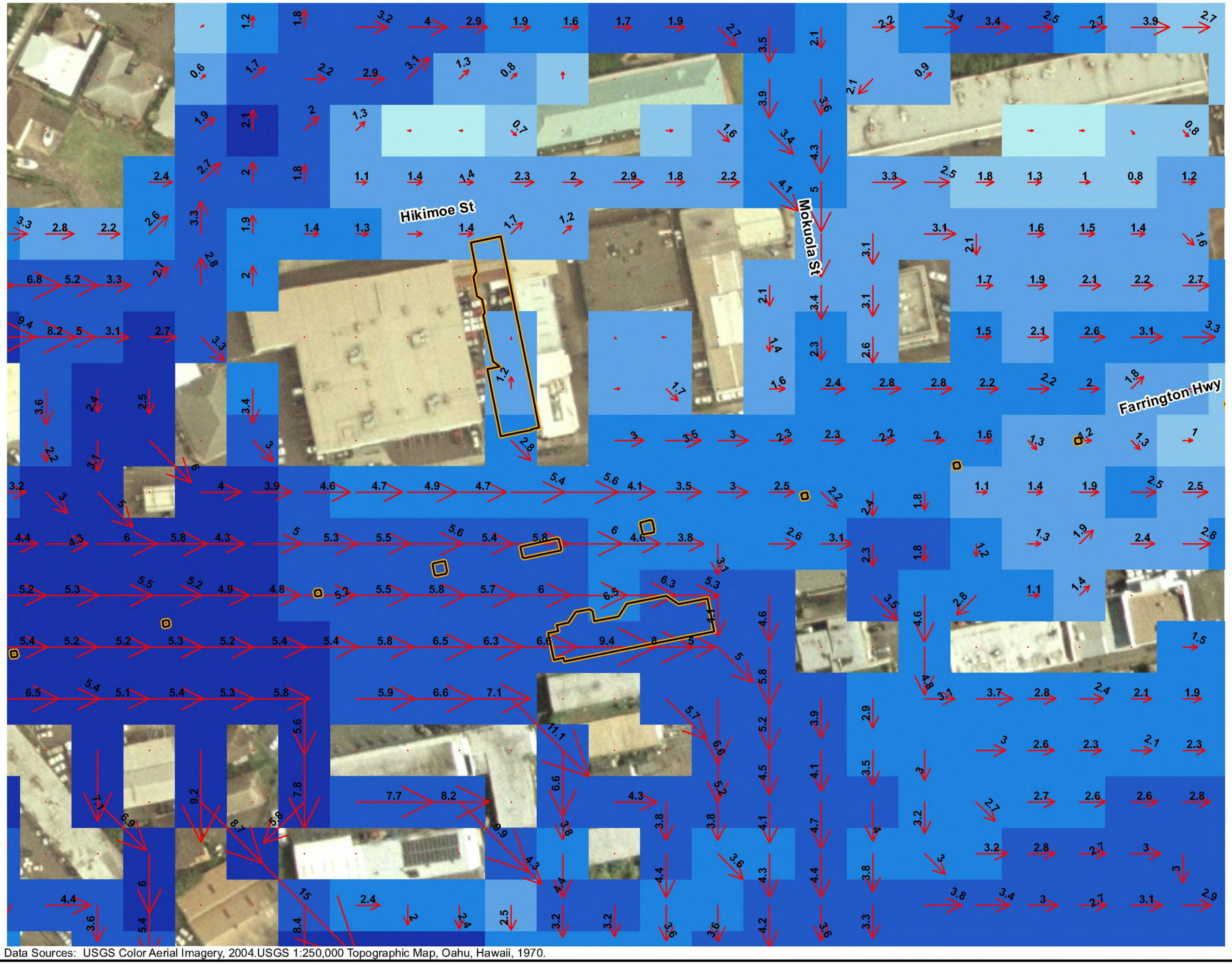
LEGEND

- Model Area
- Project Buildings and Piers/Columns
- FEMA AE Flood Zone
- Streams & Canals
- Energy Corridor
- Levee

Max Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 2.0
- 2.0 - 3.0
- 3.0 - 5.0
- 5.0 - 9.0
- 9.0 +

Figure B-2 - 100-year Flood Maximum Inundation Depths: No Levee, No Project (Coarse Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

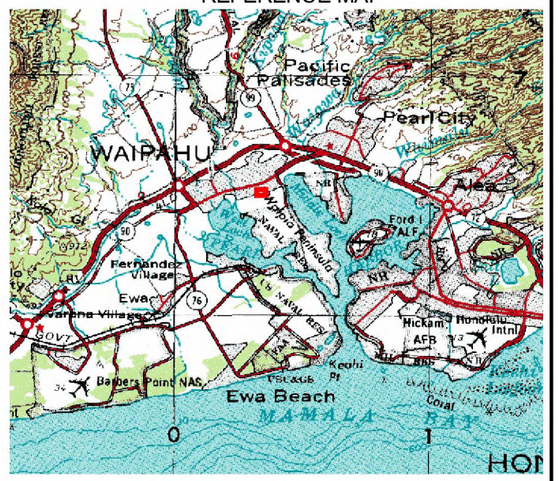
100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station (Test25NL)

Scale - 1:1,200 1 inch = 100 feet

0 50 100 200 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



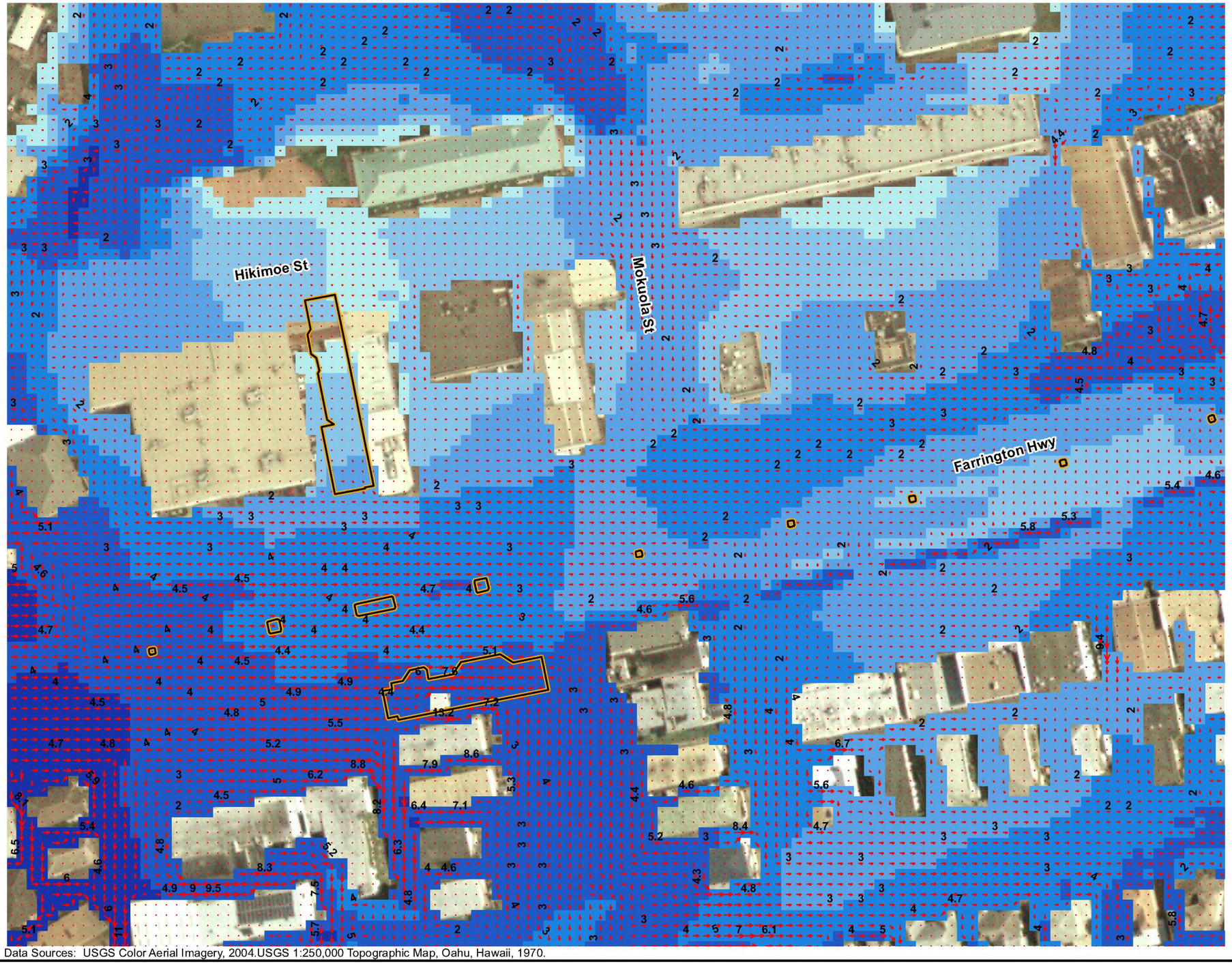
LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Velocity Vector (ft/s)

Max Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 2.0
- 2.0 - 3.0
- 3.0 - 5.0
- 5.0 - 9.0
- 9.0 +

Figure B-3 - 100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station: No Levee, No Project (Coarse Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

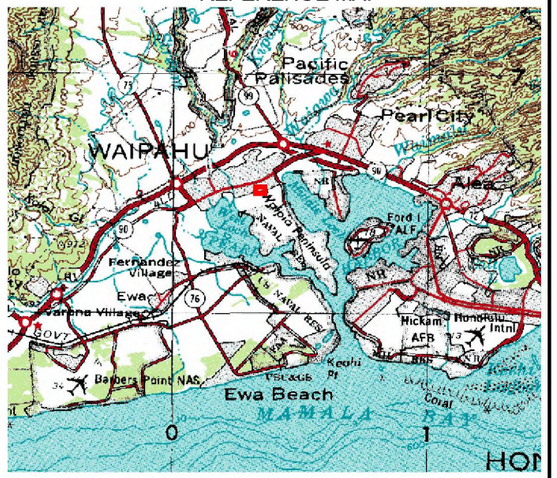
100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station (SD1_Test1NL)

Scale - 1:1,200 1 inch = 100 feet

0 50 100 200 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Streams & Canals
- Velocity Vector (ft/s)

Max Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 2.0
- 2.0 - 3.0
- 3.0 - 5.0
- 5.0 - 9.0
- 9.0 +

Figure B-4 - 100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station: No Levee, No Project (Fine Grid Model)

Appendix C. Project conditions flooding (with levee, with project)

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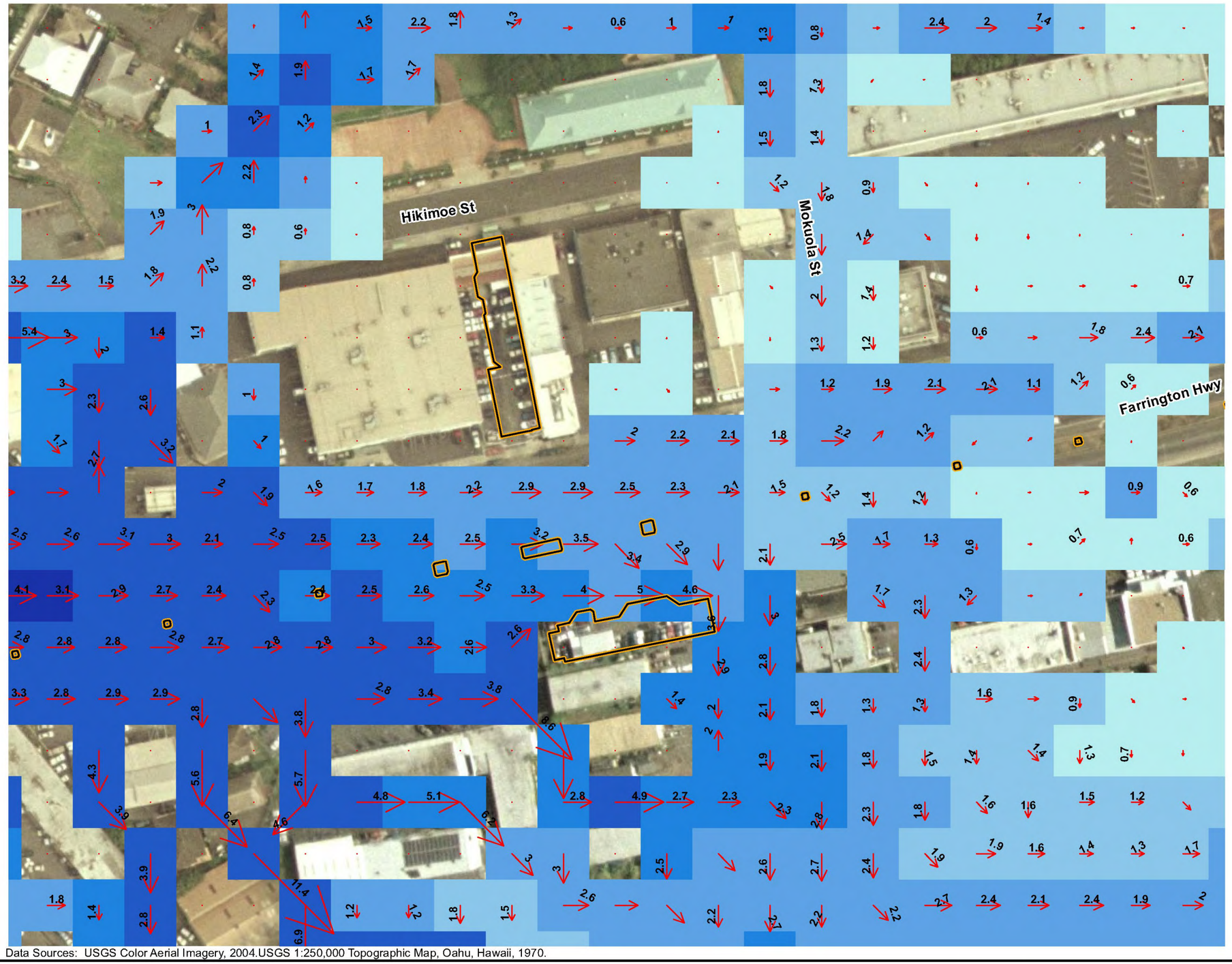


Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

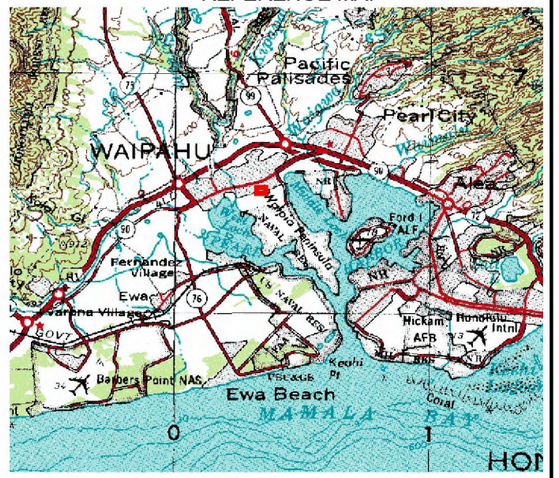
100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station (Test25_P)

Scale - 1:1,200 1 inch = 100 feet

0 50 100 200 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



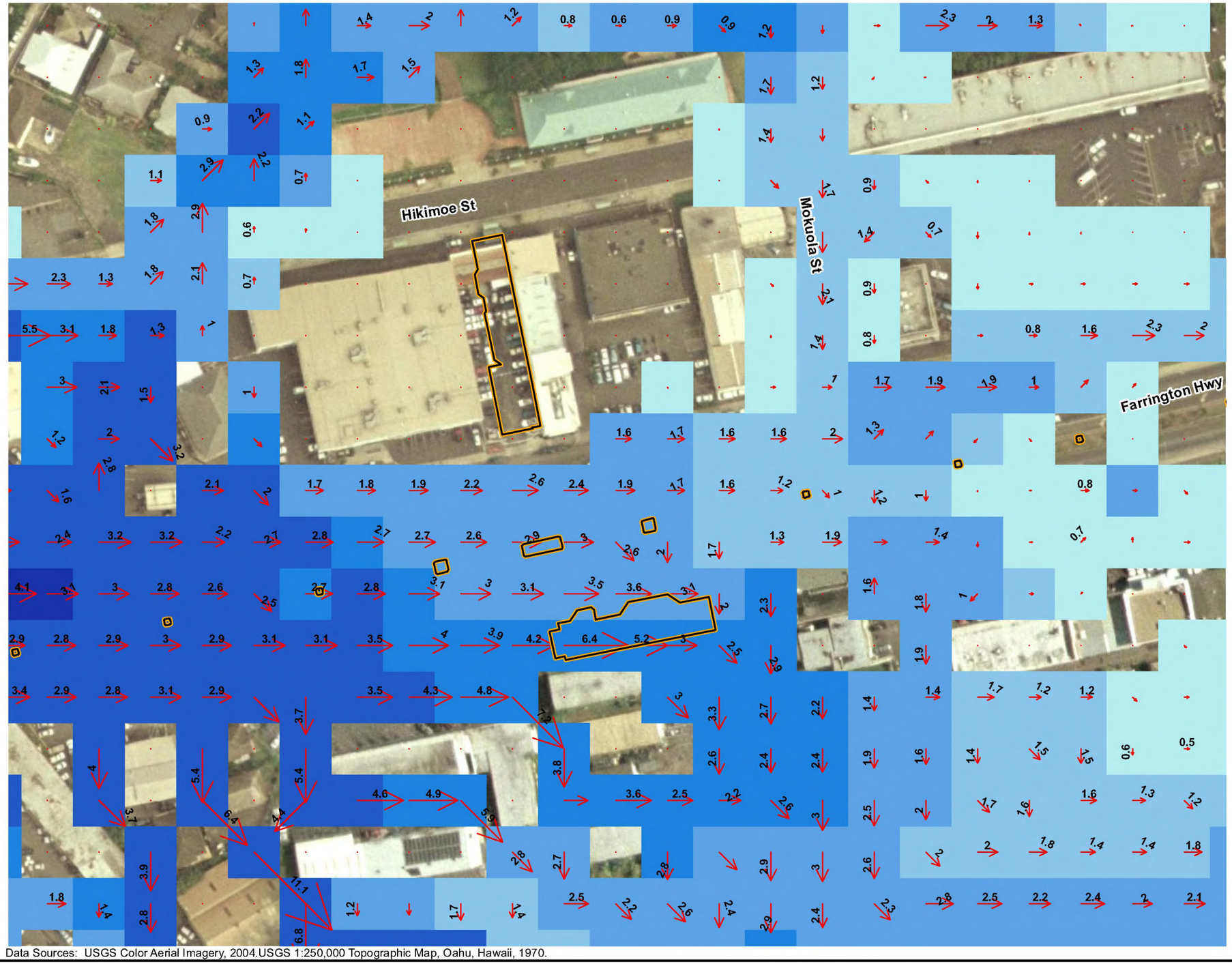
LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Velocity Vector (ft/s)

Max Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 2.0
- 2.0 - 3.0
- 3.0 - 5.0
- 5.0 - 9.0
- 9.0 +

Figure C-1 - 100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station: With Levee, Original Project (Coarse Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

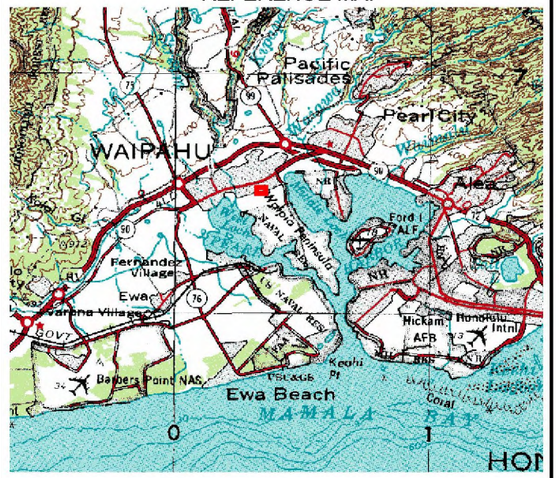
100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station (Test25_P_North)

Scale - 1:1,200 1 inch = 100 feet

0 50 100 200 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



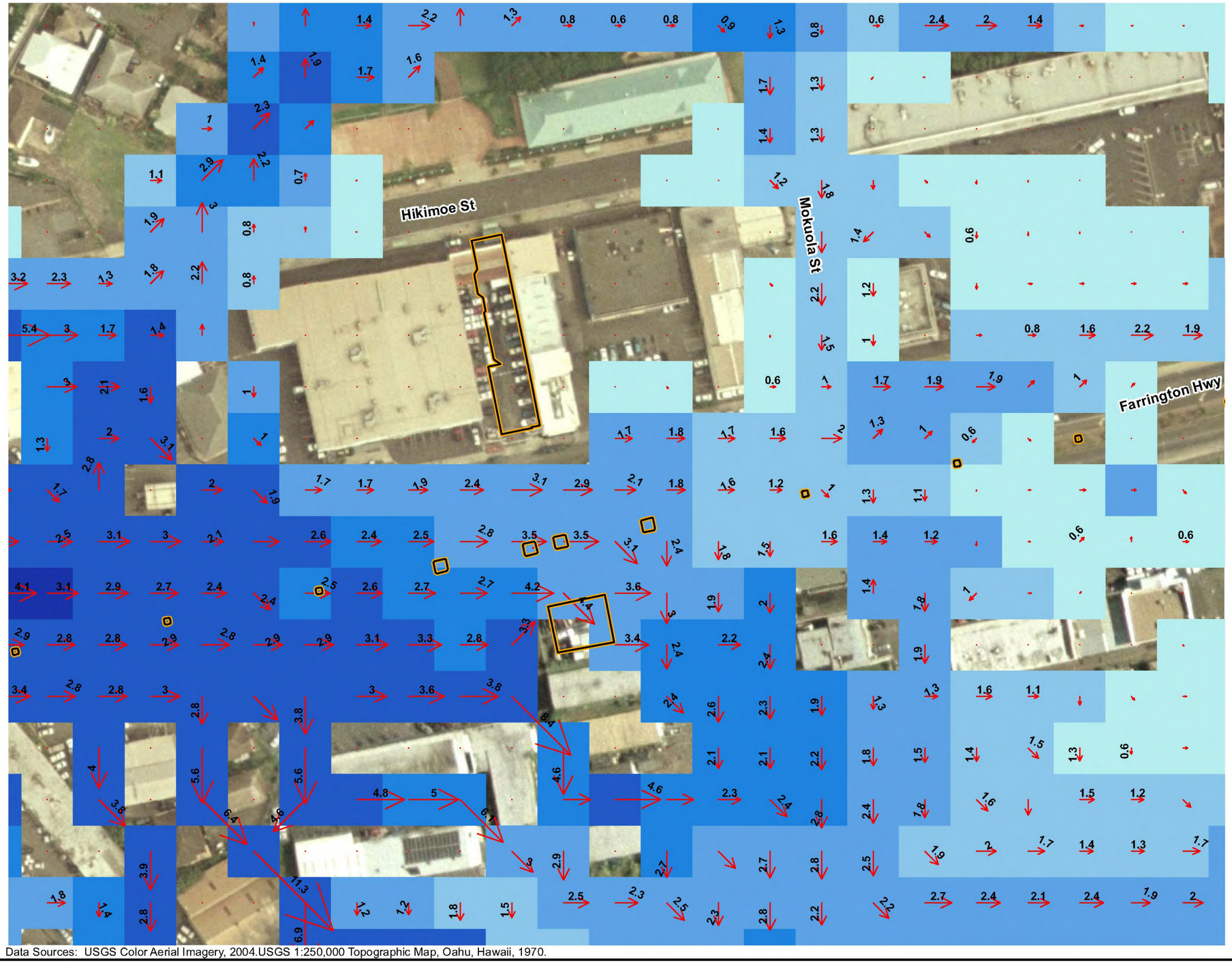
LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Velocity Vector (ft/s)

Max Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 2.0
- 2.0 - 3.0
- 3.0 - 5.0
- 5.0 - 9.0
- 9.0 +

Figure C-2 - 100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station: With Levee, Project Alternative 1 (Coarse Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

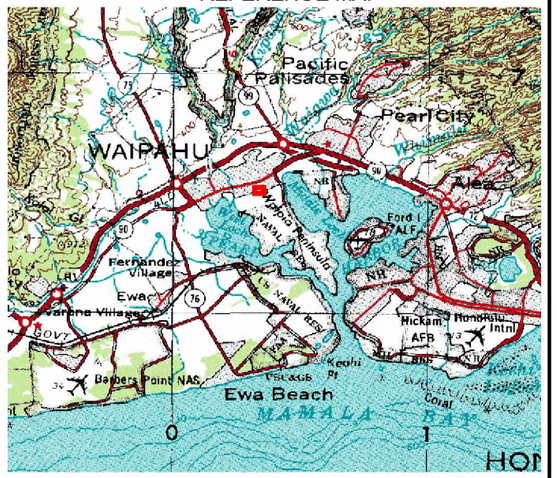
100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station (Test25_P_North_PB)

Scale - 1:1,200 1 inch = 100 feet

0 50 100 200 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



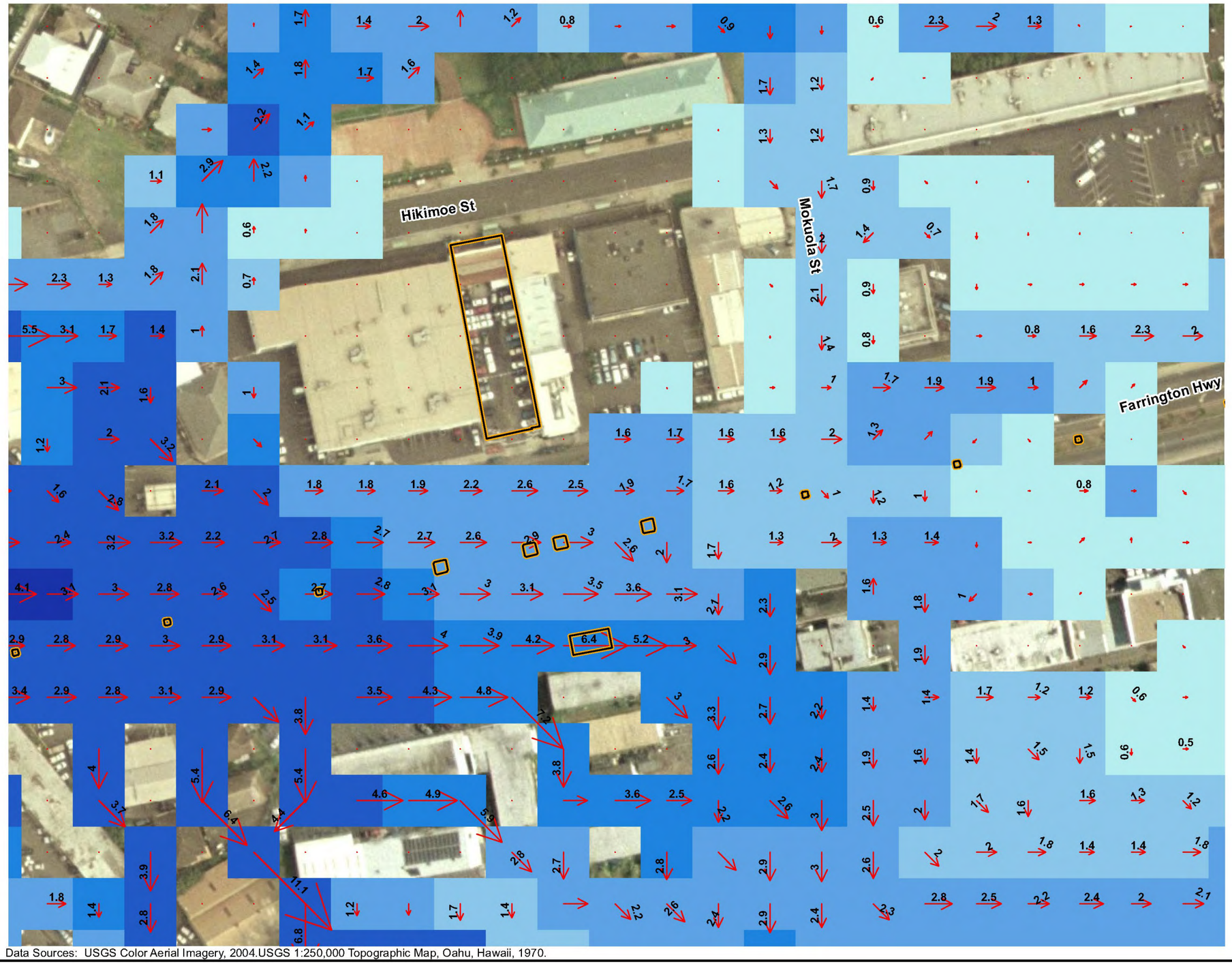
LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Velocity Vector (ft/s)

Max Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 2.0
- 2.0 - 3.0
- 3.0 - 5.0
- 5.0 - 9.0
- 9.0 +

Figure C-3 - 100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station: With Levee, Project Alternative 2 (Coarse Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

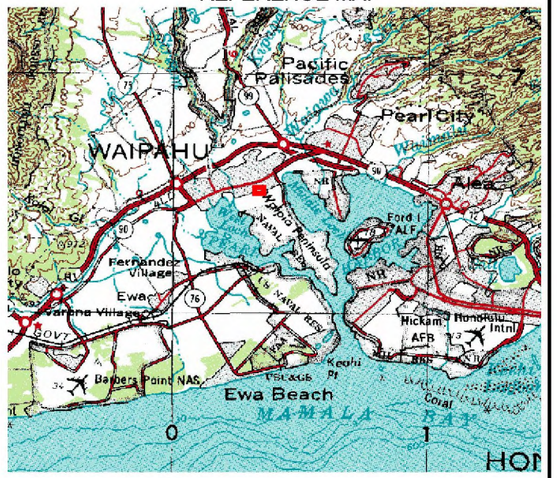
100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station (Test25_P_North_NHC)

Scale - 1:1,200 1 inch = 100 feet

0 50 100 200 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Velocity Vector (ft/s)

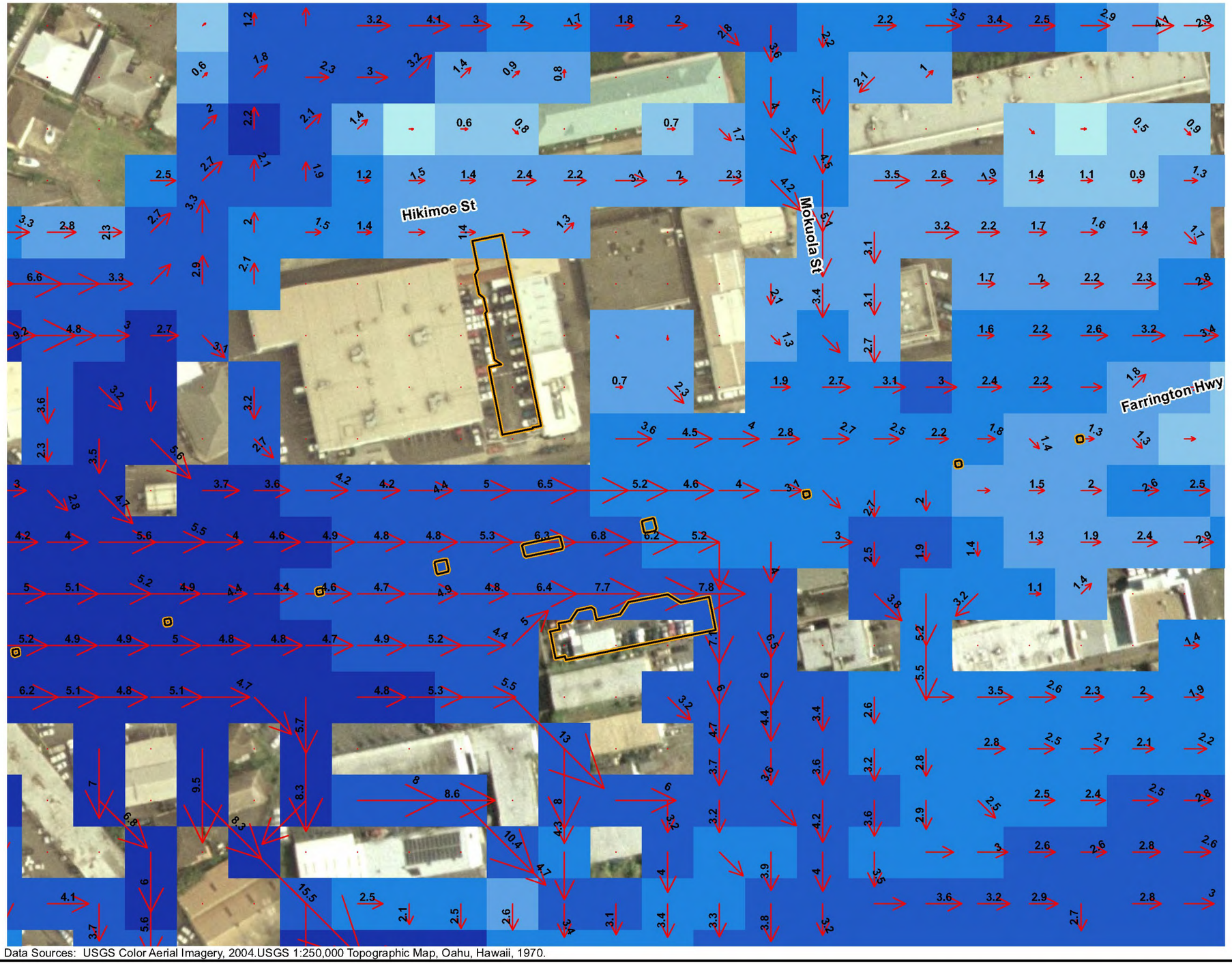
Max Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 2.0
- 2.0 - 3.0
- 3.0 - 5.0
- 5.0 - 9.0
- 9.0 +

Figure C-4 - 100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station: With Levee, Project Alternative 3 (Coarse Grid Model)

Appendix D. Project conditions flooding (no levee, with project)

AR00041462



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

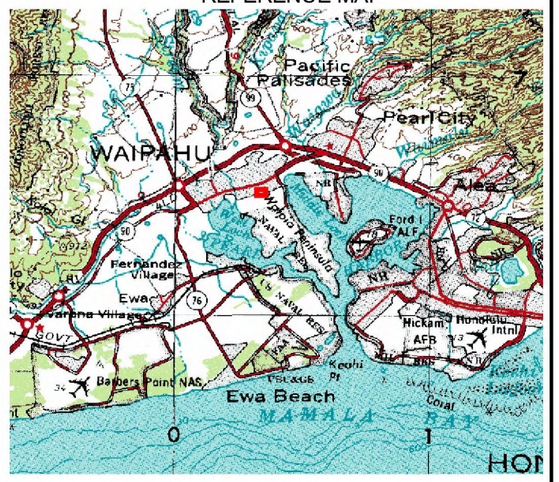
100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station (Test25NL_P)

Scale - 1:1,200 1 inch = 100 feet

0 50 100 200 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



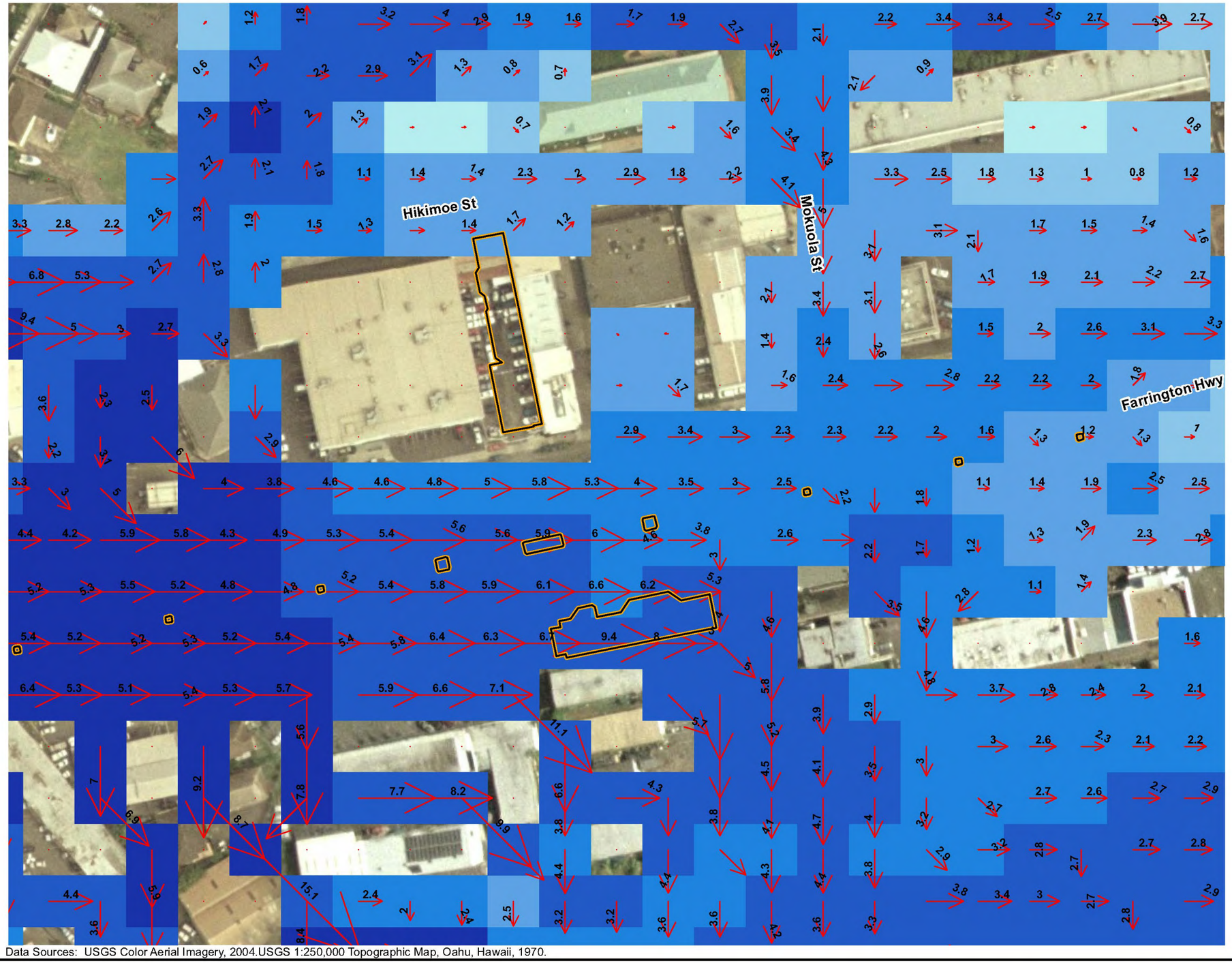
LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Velocity Vector (ft/s)

Max Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 2.0
- 2.0 - 3.0
- 3.0 - 5.0
- 5.0 - 9.0
- 9.0 +

Figure D-1 - 100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station: No Levee, Original Project (Coarse Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

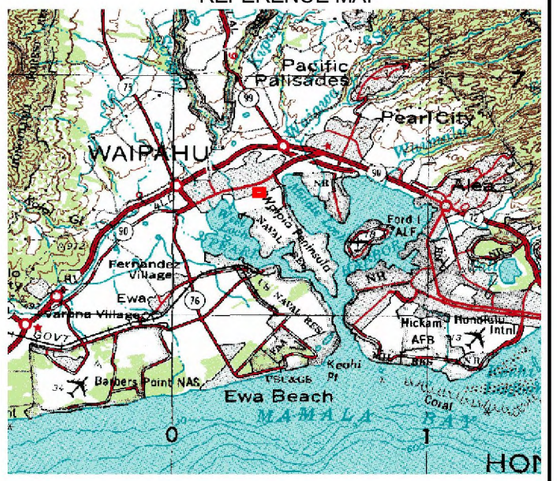
100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station (Test25NL_P_North)

Scale - 1:1,200 1 inch = 100 feet

0 50 100 200 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



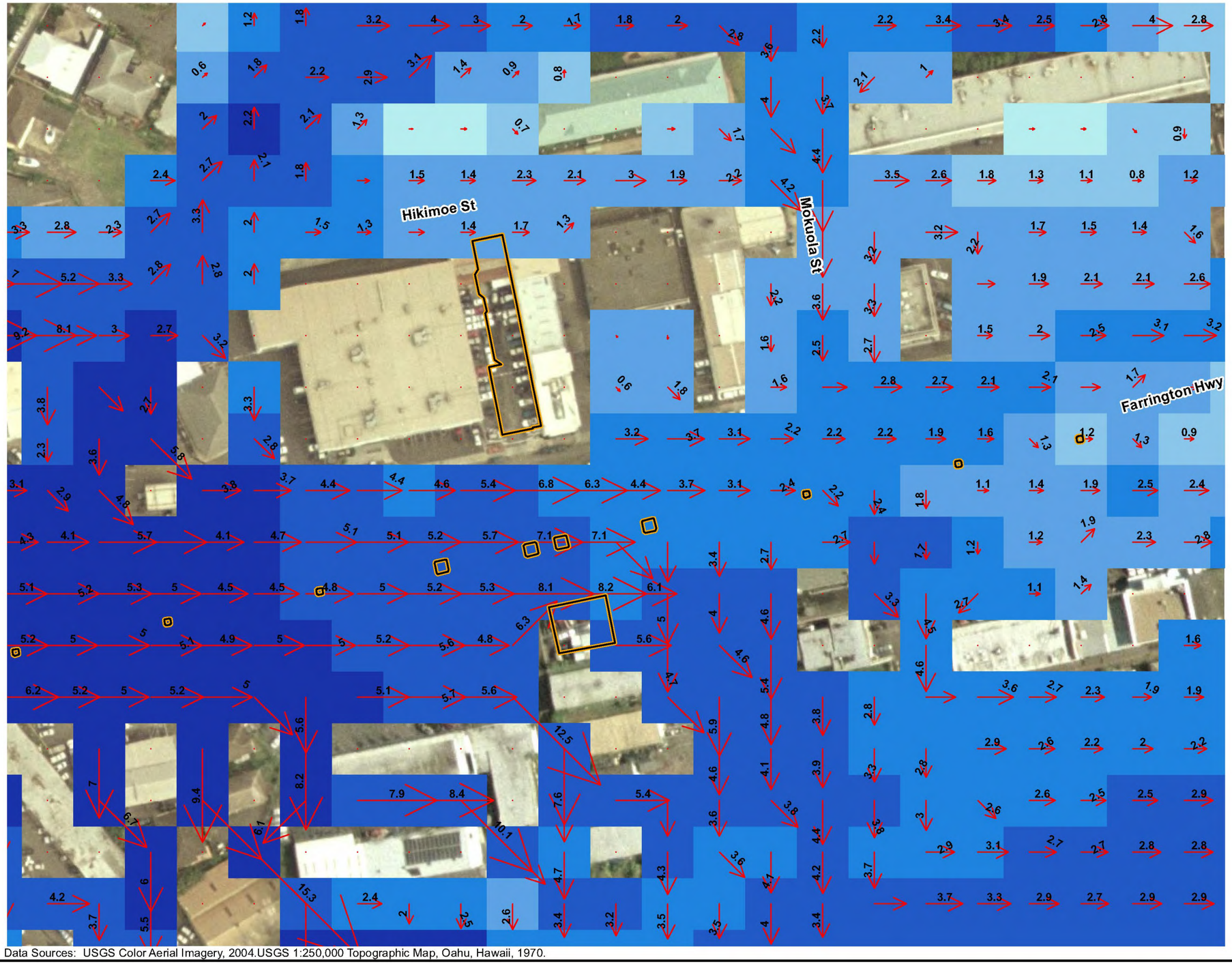
LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Velocity Vector (ft/s)

Max Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 2.0
- 2.0 - 3.0
- 3.0 - 5.0
- 5.0 - 9.0
- 9.0 +

Figure D-2 - 100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station: No Levee, Project Alternative 1 (Coarse Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

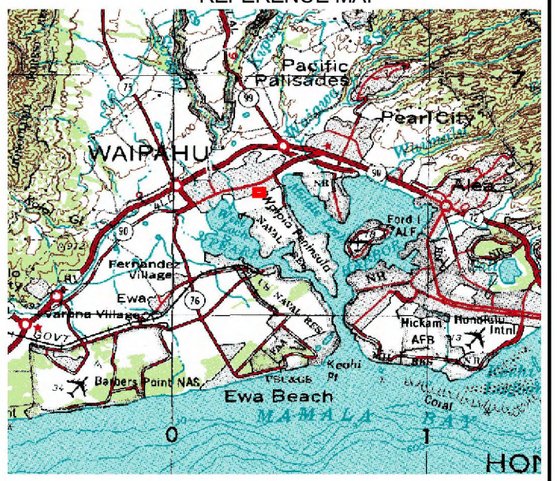
100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station (Test25NL_P_North_PB)

Scale - 1:1,200 1 inch = 100 feet

0 50 100 200 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



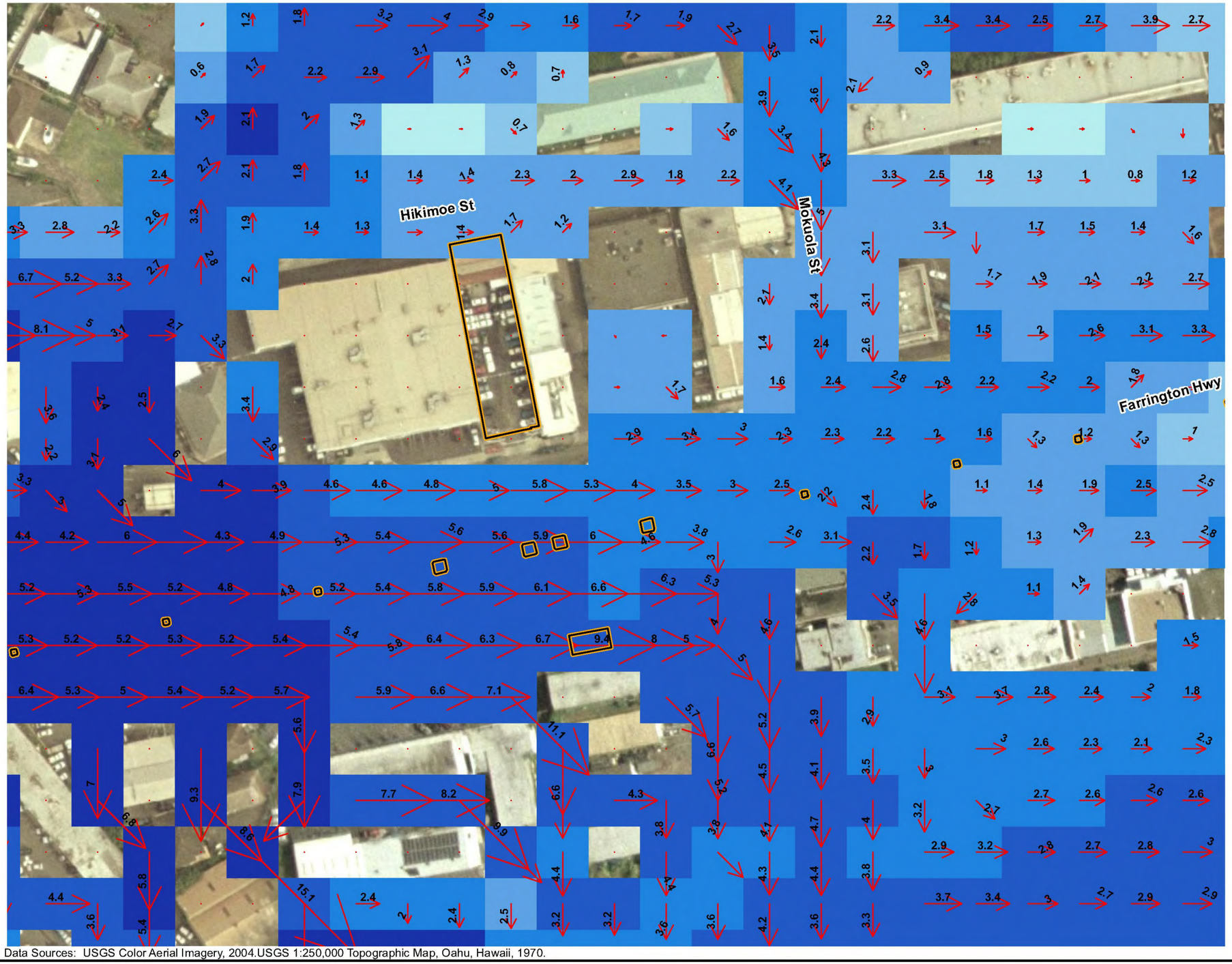
LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Velocity Vector (ft/s)

Max Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 2.0
- 2.0 - 3.0
- 3.0 - 5.0
- 5.0 - 9.0
- 9.0 +

Figure D-3 - 100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station: No Levee, Project Alternative 2 (Coarse Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

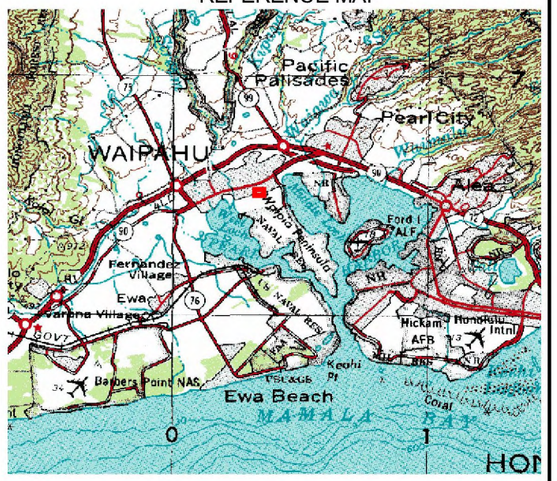
100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station (Test25NL_P_North_NHC)

Scale - 1:1,200 1 inch = 100 feet

0 50 100 200 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



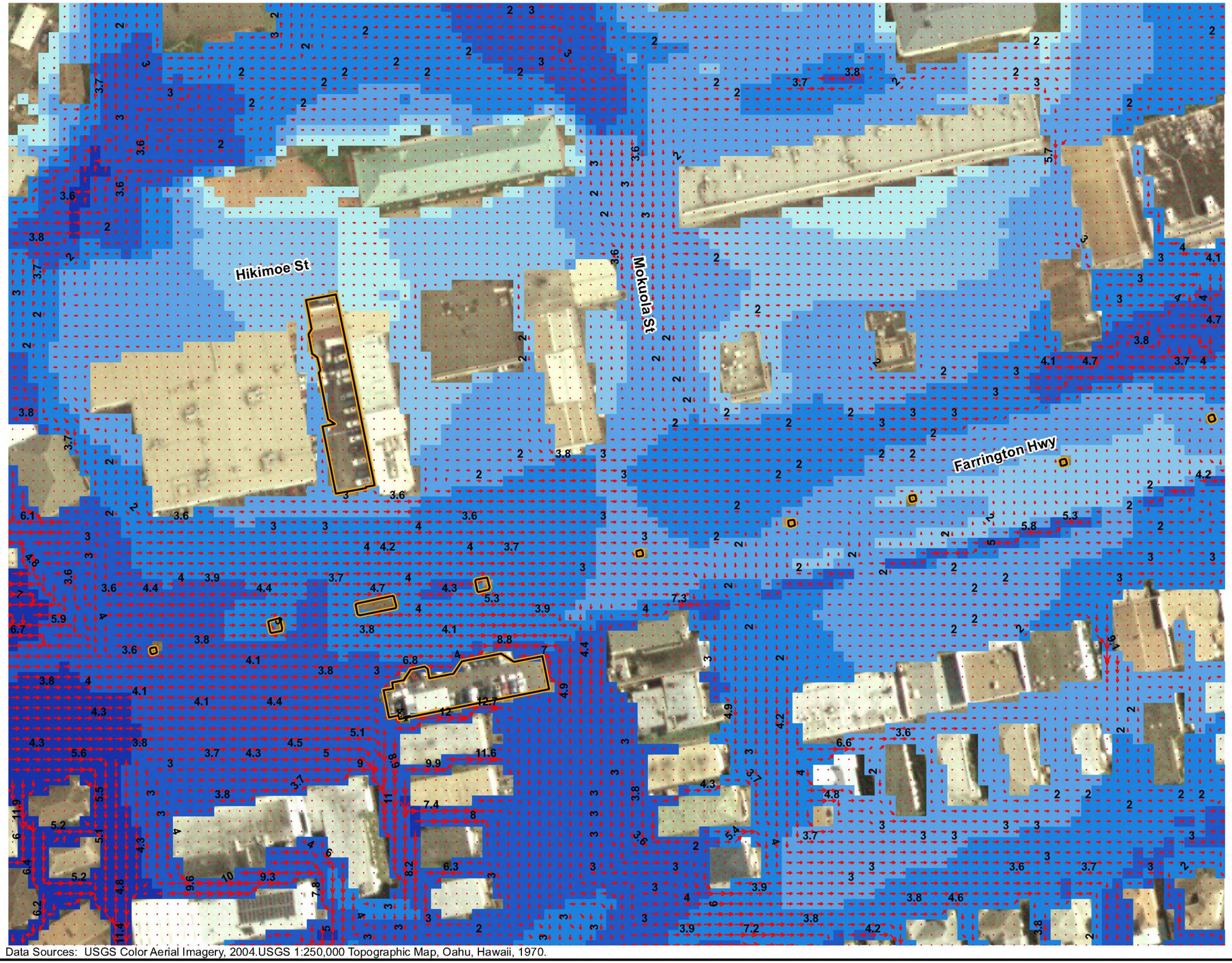
LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Velocity Vector (ft/s)

Max Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 2.0
- 2.0 - 3.0
- 3.0 - 5.0
- 5.0 - 9.0
- 9.0 +

Figure D-4 - 100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station: No Levee, Project Alternative 3 (Coarse Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

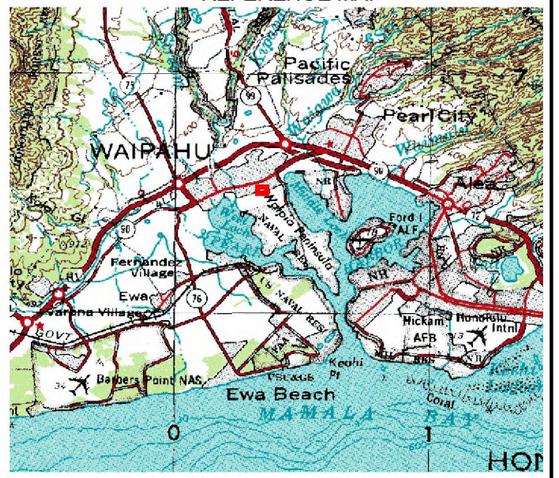
100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station (SD1_Test1NL_P)

Scale - 1:1,200 1 inch = 100 feet

0 50 100 200 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



LEGEND

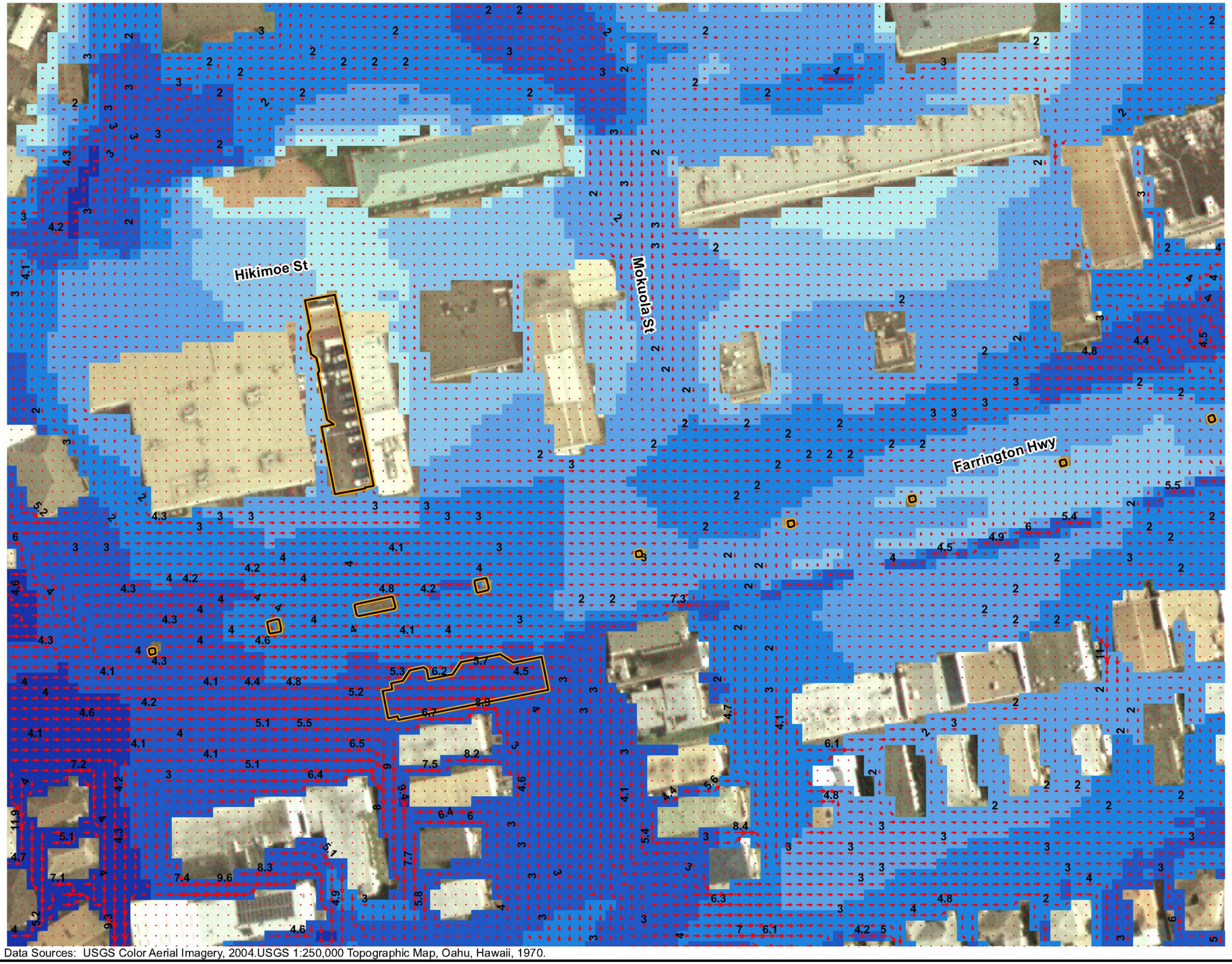
Project Buildings and Piers/Columns

Velocity Vector (ft/s)

Max Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 2.0
- 2.0 - 3.0
- 3.0 - 5.0
- 5.0 - 9.0
- 9.0 +

Figure D-5 - 100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station: No Levee, Original Project (Fine Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

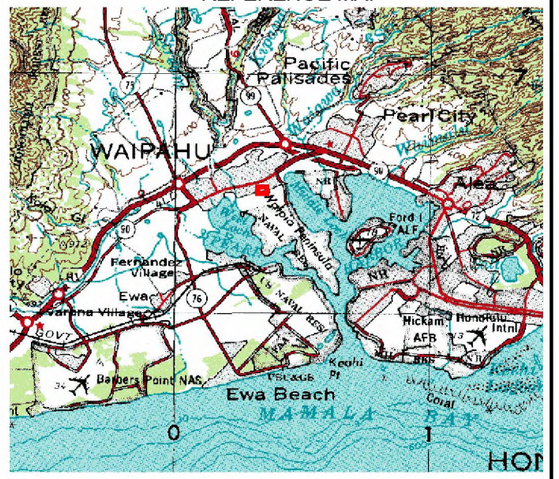
100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station (SD1_Test1NL_P_North)

Scale - 1:1,200 1 inch = 100 feet

0 50 100 200 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



LEGEND

Project Buildings and Piers/Columns

Velocity Vector (ft/s)

Max Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 2.0
- 2.0 - 3.0
- 3.0 - 5.0
- 5.0 - 9.0
- 9.0 +

Figure D-6 - 100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station: No Levee, Project Alternative 1 (Fine Grid Model)

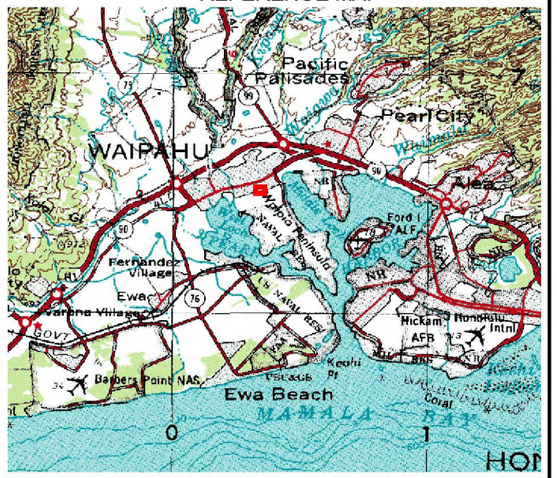
100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station (SD1_Test1NL_P_North_PB)

Scale - 1:1,200 1 inch = 100 feet

0 50 100 200 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



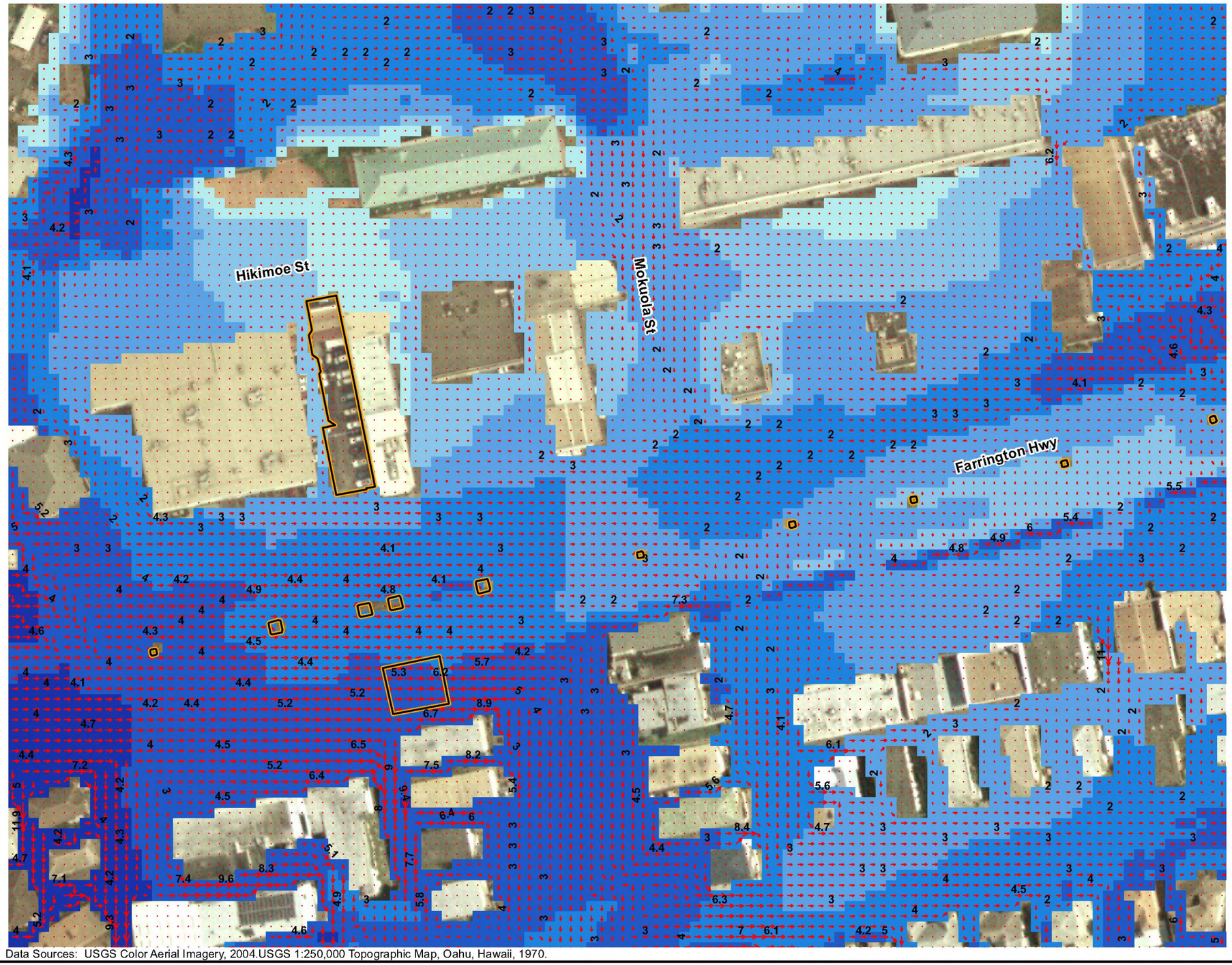
LEGEND

Project Buildings and Piers/Columns

Velocity Vector (ft/s)

Max Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 2.0
- 2.0 - 3.0
- 3.0 - 5.0
- 5.0 - 9.0
- 9.0 +



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

Figure D-7 - 100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station: No Levee, Project Alternative 2 (Fine Grid Model)

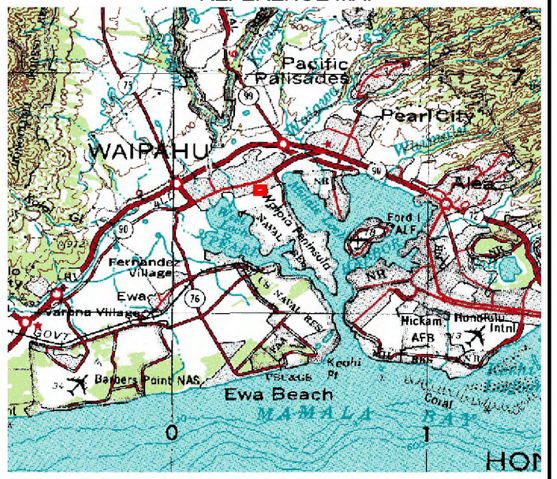
100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station (SD1_Test1NL_P_North_NHC)

Scale - 1:1,200 1 inch = 100 feet

0 50 100 200 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



LEGEND

Project Buildings and Piers/Columns

Velocity Vector (ft/s)

Max Depth (ft)

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 2.0
- 2.0 - 3.0
- 3.0 - 5.0
- 5.0 - 9.0
- 9.0 +

Hikimoe St

Mokuola St

Farrington Hwy

Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

Figure D-8 - 100-year Flood Maximum Inundation Depths and Velocity Vectors in Vicinity of Project Station: No Levee, Project Alternative 3 (Fine Grid Model)

Appendix E. Differential maps (with levee, project conditions minus existing conditions)

AR00041471

Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

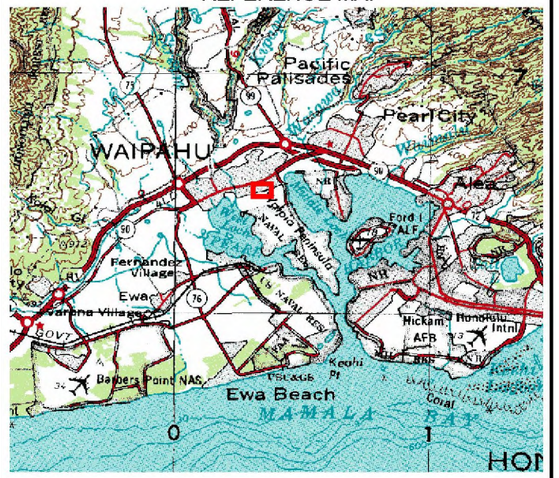
100-year Flood Maximum Inundation Depth Differences in Vicinity of Project Station (Test25_P minus Test25)

Scale - 1:2,400 1 inch = 200 feet

0 100 200 400 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



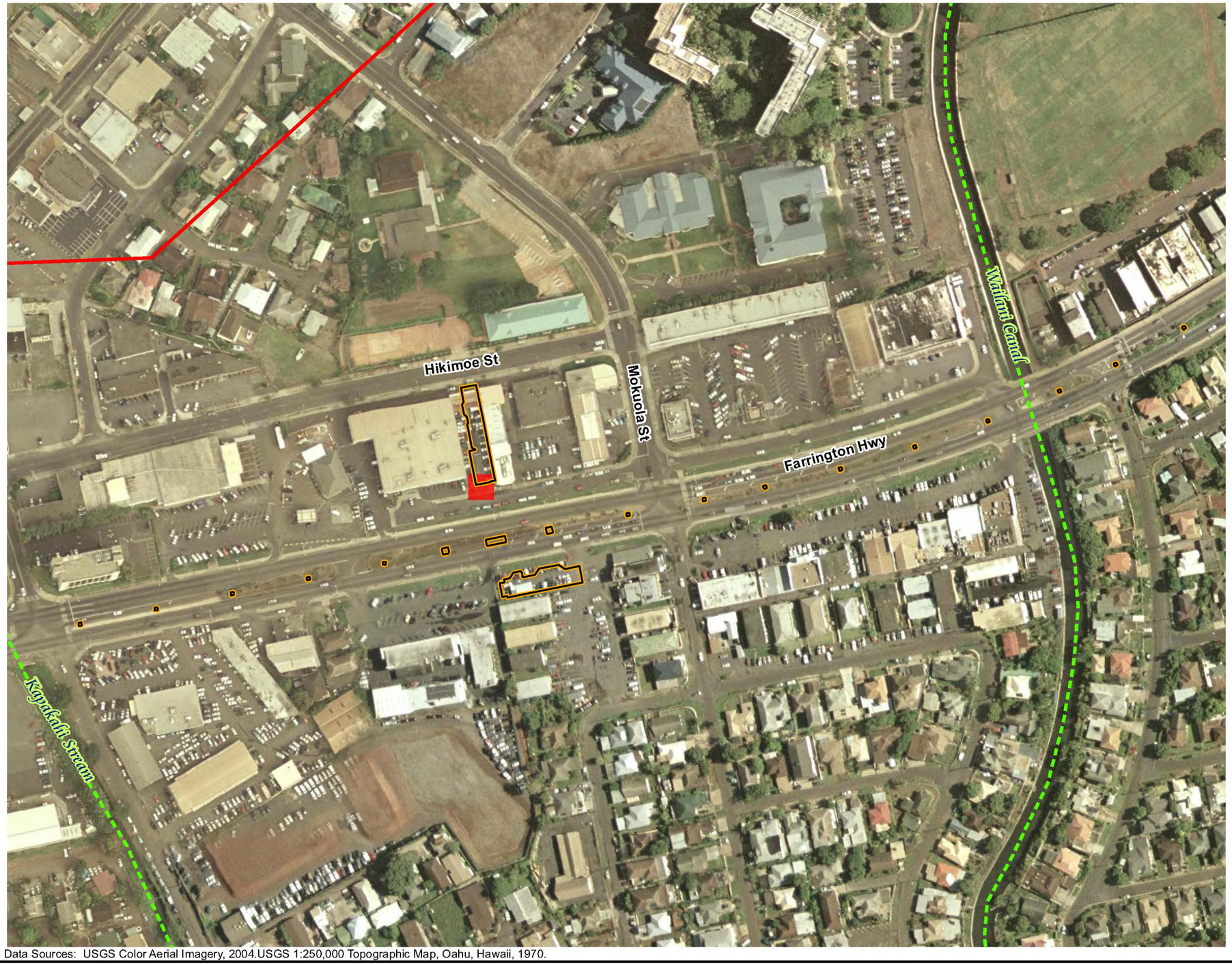
LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Streams & Canals

Depth Difference (ft)

-4.1 - -0.5	0.1 - 0.2
-0.5 - -0.4	0.2 - 0.3
-0.4 - -0.3	0.3 - 0.4
-0.3 - -0.2	0.4 - 0.5
-0.2 - -0.1	0.5 +
-0.1 - 0.1 (transparent)	

Figure E-1 - 100-year Flood Maximum Inundation Depth Differences in Vicinity of Project Station: With Levee, Original Project (Coarse Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

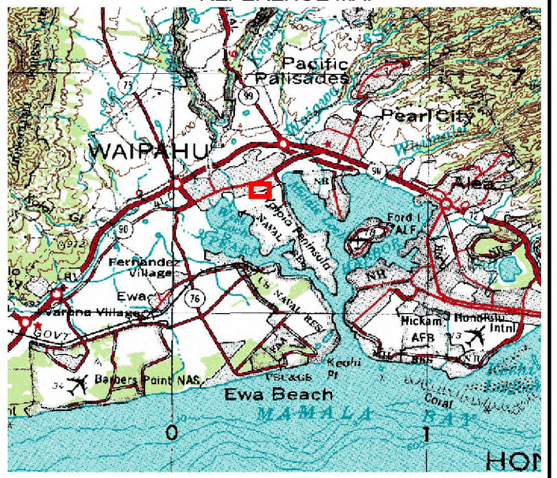
100-year Flood Maximum Inundation Depth Differences in Vicinity of Project Station (Test25_P_North minus Test25)

Scale - 1:2,400 1 inch = 200 feet

0 100 200 400 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Streams & Canals

Depth Difference (ft)

- -4.1 - -0.5
- -0.5 - -0.4
- -0.4 - -0.3
- -0.3 - -0.2
- -0.2 - -0.1
- -0.1 - 0.1 (transparent)
- 0.1 - 0.2
- 0.2 - 0.3
- 0.3 - 0.4
- 0.4 - 0.5
- 0.5 +

Figure E-2 - 100-year Flood Maximum Inundation Depth Differences in Vicinity of Project Station: With Levee, Project Alternative 1 (Coarse Grid Model)

Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

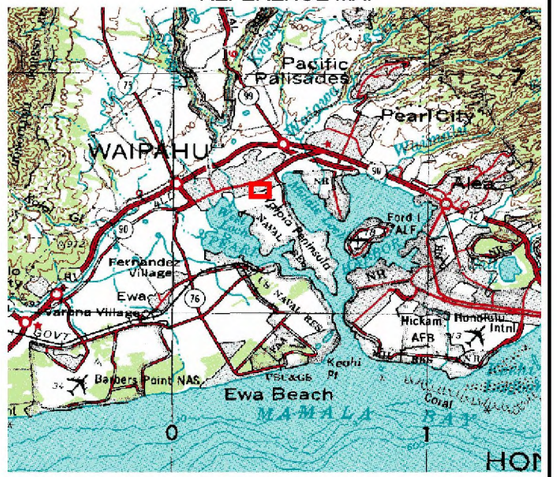
100-year Flood Maximum Inundation Depth Differences in Vicinity of Project Station (Test25_P_North_PB minus Test25)

Scale - 1:2,400 1 inch = 200 feet

0 100 200 400 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



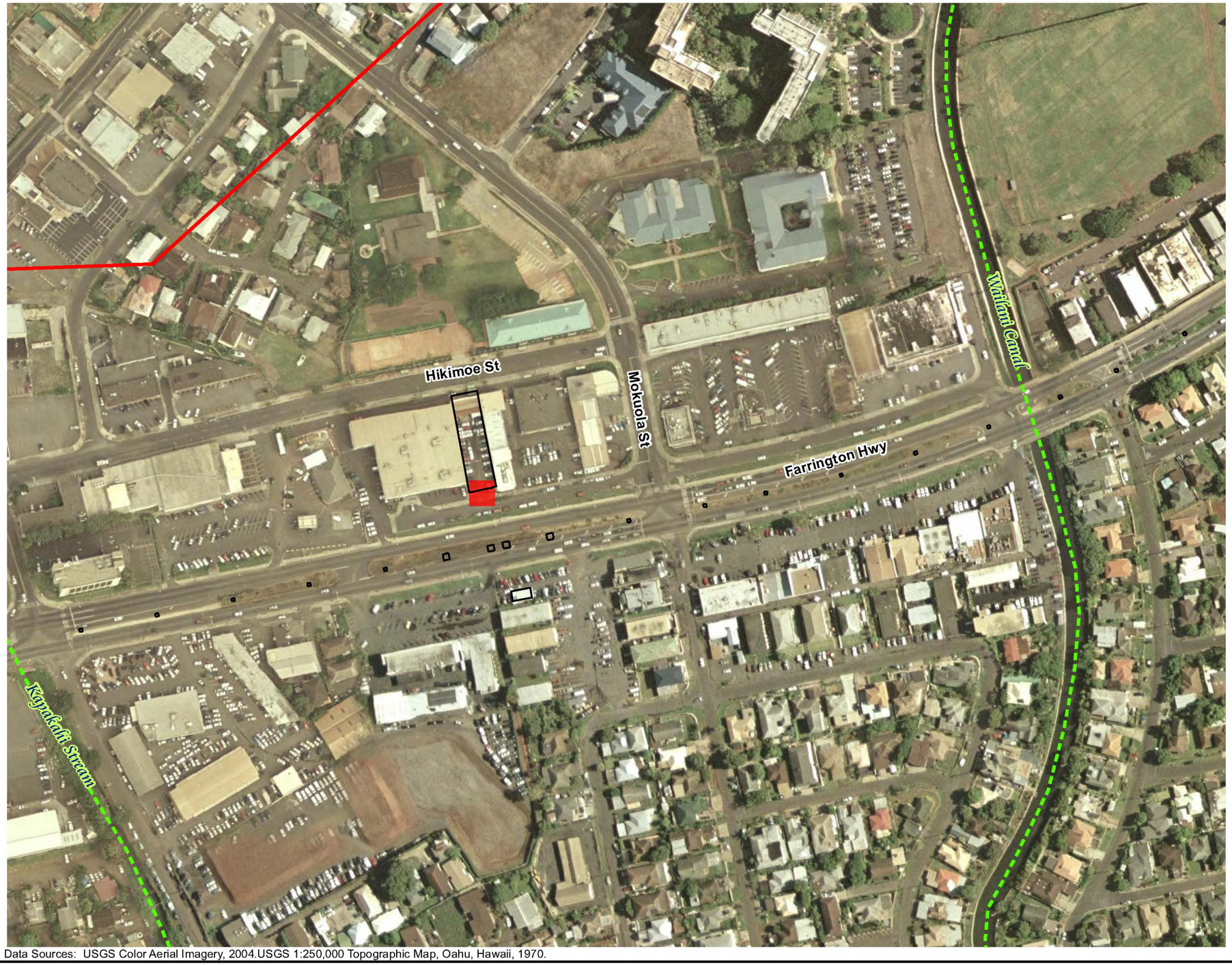
LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Streams & Canals

Depth Difference (ft)

- -4.1 - -0.5
- -0.5 - -0.4
- -0.4 - -0.3
- -0.3 - -0.2
- -0.2 - -0.1
- -0.1 - 0.1 (transparent)
- 0.1 - 0.2
- 0.2 - 0.3
- 0.3 - 0.4
- 0.4 - 0.5
- 0.5 +

Figure E-3 - 100-year Flood Maximum Inundation Depth Differences in Vicinity of Project Station: With Levee, Project Alternative 2 (Coarse Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

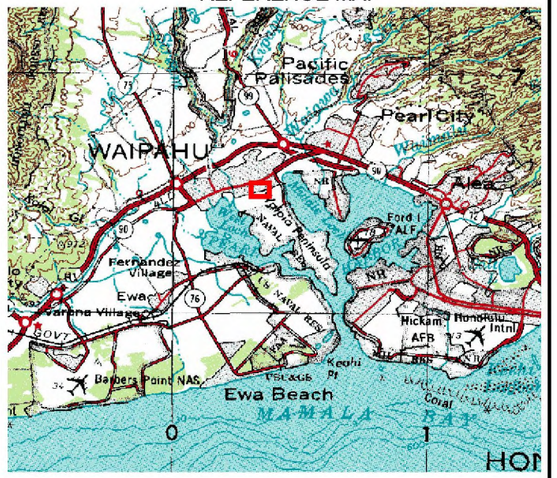
100-year Flood Maximum Inundation Depth Differences in Vicinity of Project Station (Test25_P_North_NHC minus Test25)

Scale - 1:2,400 1 inch = 200 feet

0 100 200 400 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



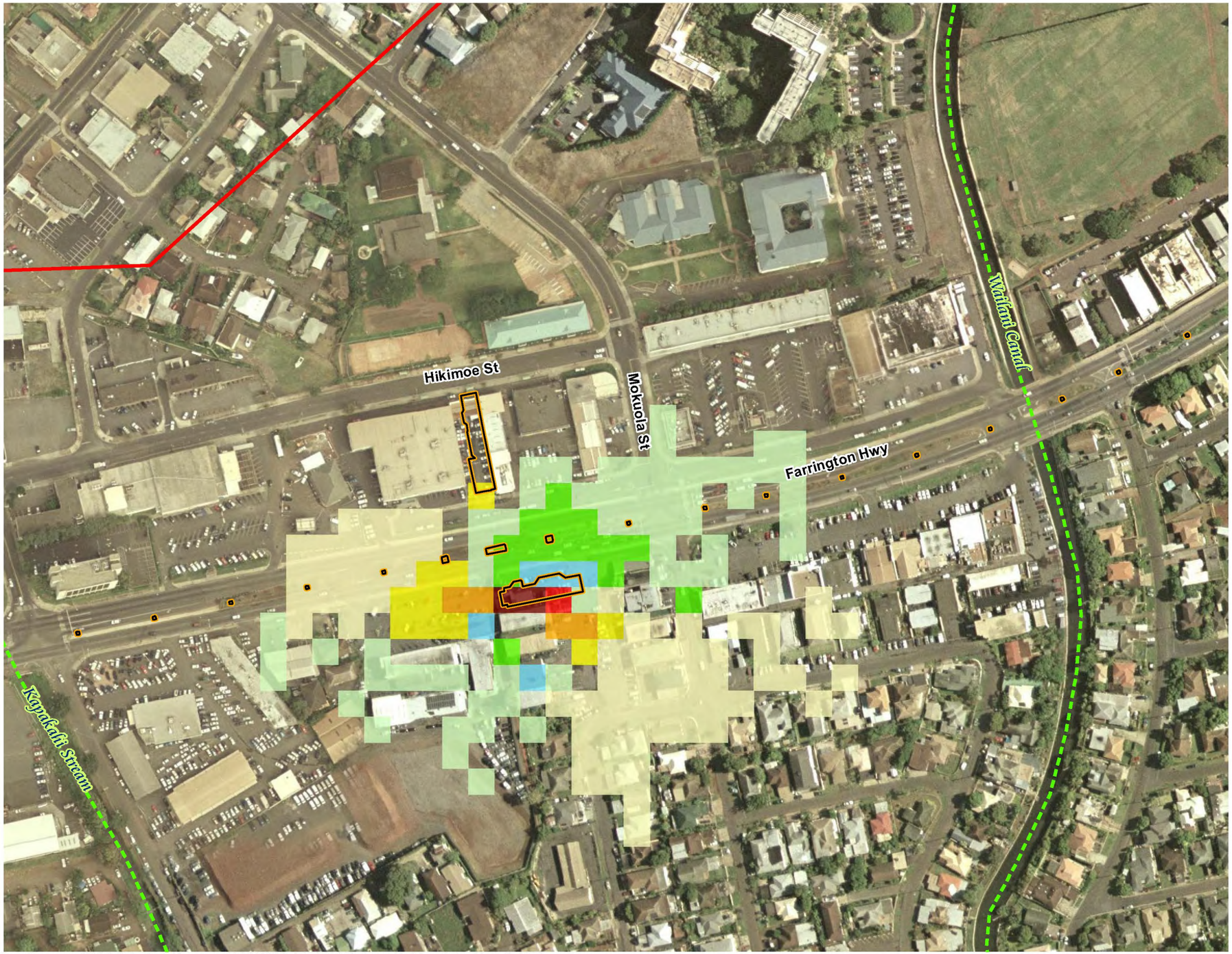
LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Streams & Canals

Depth Difference (ft)

- -0.6 - -0.5
- -0.5 - -0.4
- -0.4 - -0.3
- -0.3 - -0.2
- -0.2 - -0.1
- -0.1 - 0.1 (transparent)
- 0.1 - 0.2
- 0.2 - 0.3
- 0.3 - 0.4
- 0.4 - 0.5
- 0.5 +

Figure E-4 - 100-year Flood Maximum Inundation Depth Differences in Vicinity of Project Station: With Levee, Project Alternative 3 (Coarse Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

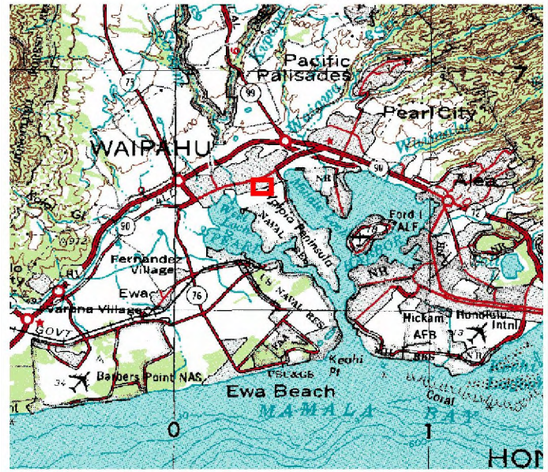
100-year Flood Velocity Differences in Vicinity of Project Station (Test25_P minus Test25)

Scale - 1:2,400 1 inch = 200 feet

0 100 200 400 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



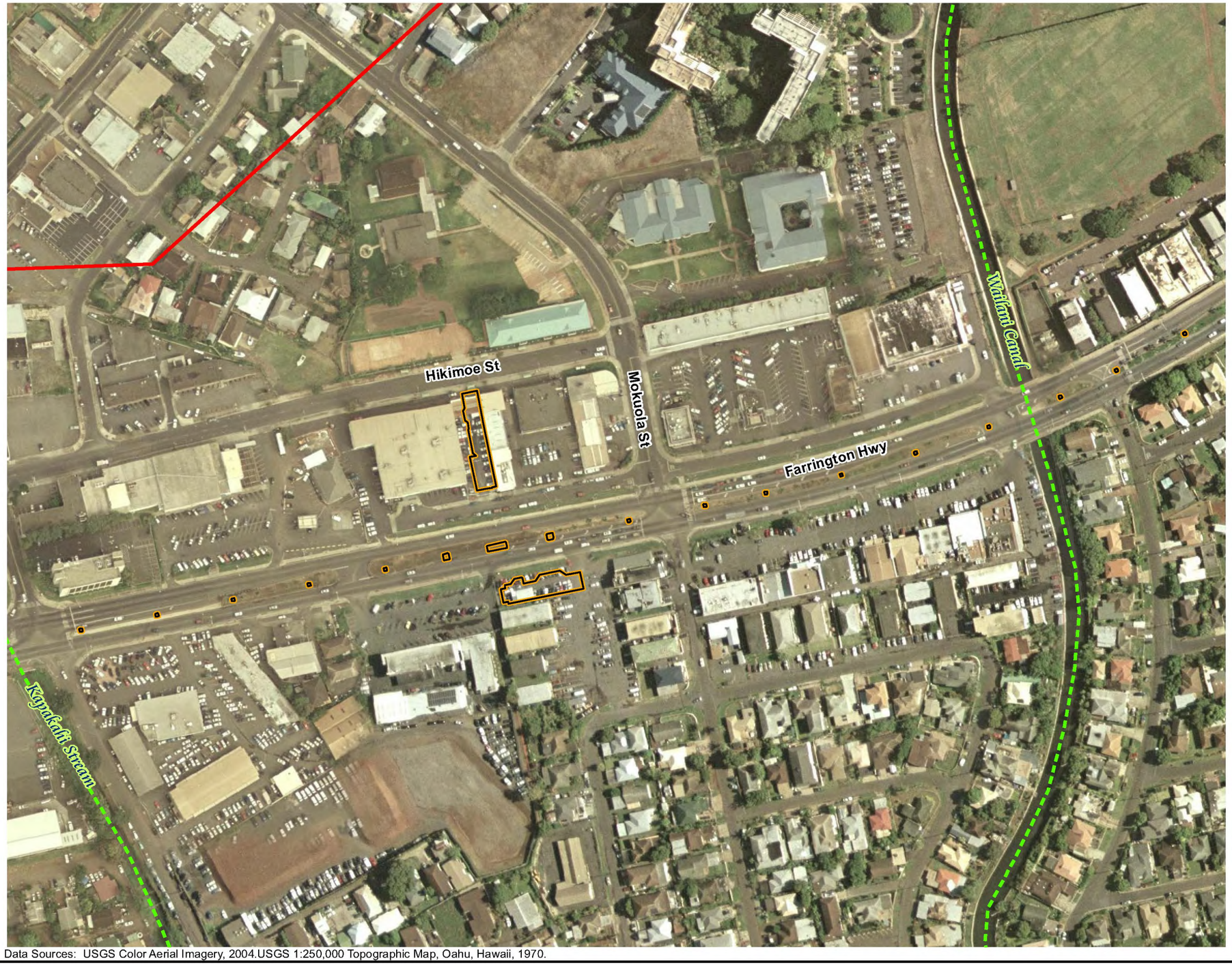
LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Streams & Canals

Velocity Difference (ft/s)

-3.0 -	0.1 - 0.5
-3.0 - -2.0	0.5 - 1.0
-2.0 - -1.0	1.0 - 2.0
-1.0 - -0.5	2.0 - 3.0
-0.5 - -0.1	3.0 +
-0.1 - 0.1 (transparent)	

Figure E-5 - 100-year Flood Velocity Differences in Vicinity of Project Station: With Levee, Original Project (Coarse Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

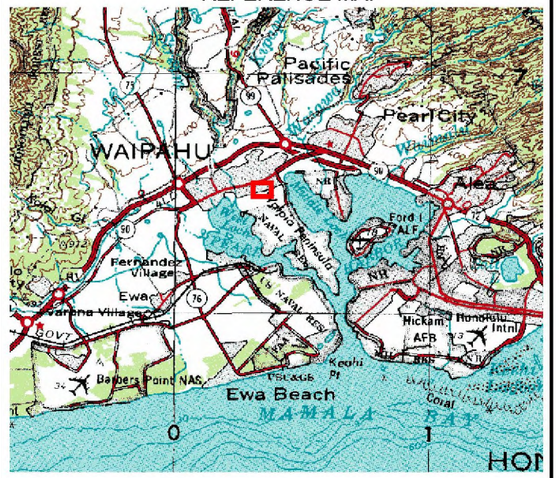
100-year Flood Velocity Differences in Vicinity of Project Station (Test25_P_North minus Test25)

Scale - 1:2,400 1 inch = 200 feet

0 100 200 400 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



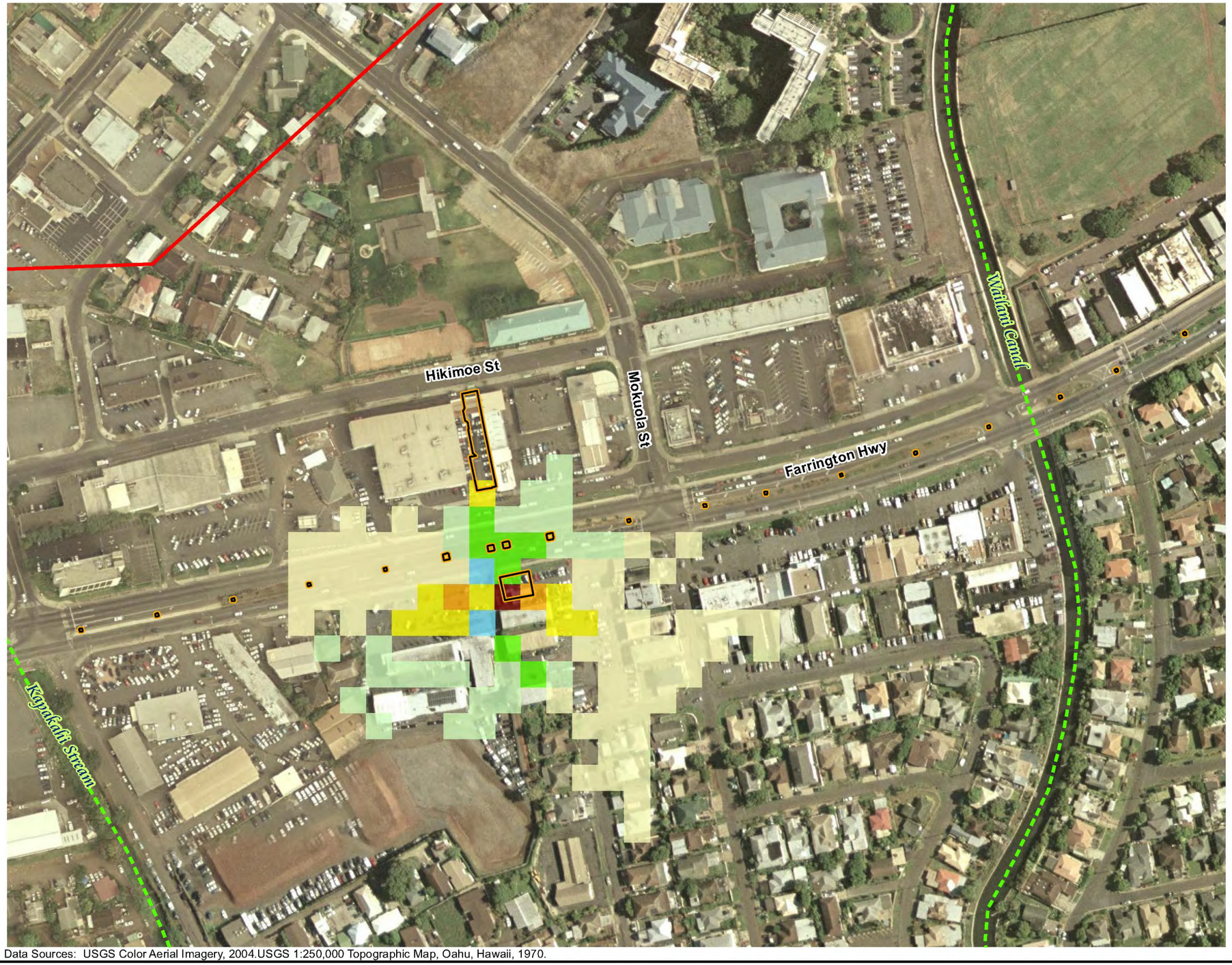
LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Streams & Canals

Velocity Difference (ft/s)

-3.0 -	0.1 - 0.5
-3.0 - -2.0	0.5 - 1.0
-2.0 - -1.0	1.0 - 2.0
-1.0 - -0.5	2.0 - 3.0
-0.5 - -0.1	3.0 +
-0.1 - 0.1 (transparent)	

Figure E-6 - 100-year Flood Velocity Differences in Vicinity of Project Station: With Levee, Project Alternative 1 (Coarse Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

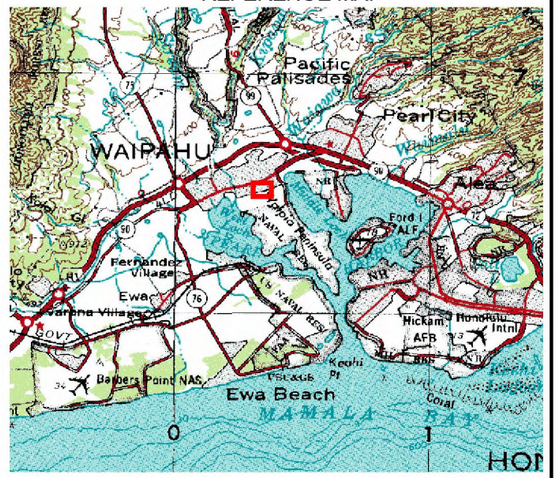
100-year Flood Velocity Differences in Vicinity of Project Station (Test25_P_North_PB minus Test25)

Scale - 1:2,400 1 inch = 200 feet

0 100 200 400 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



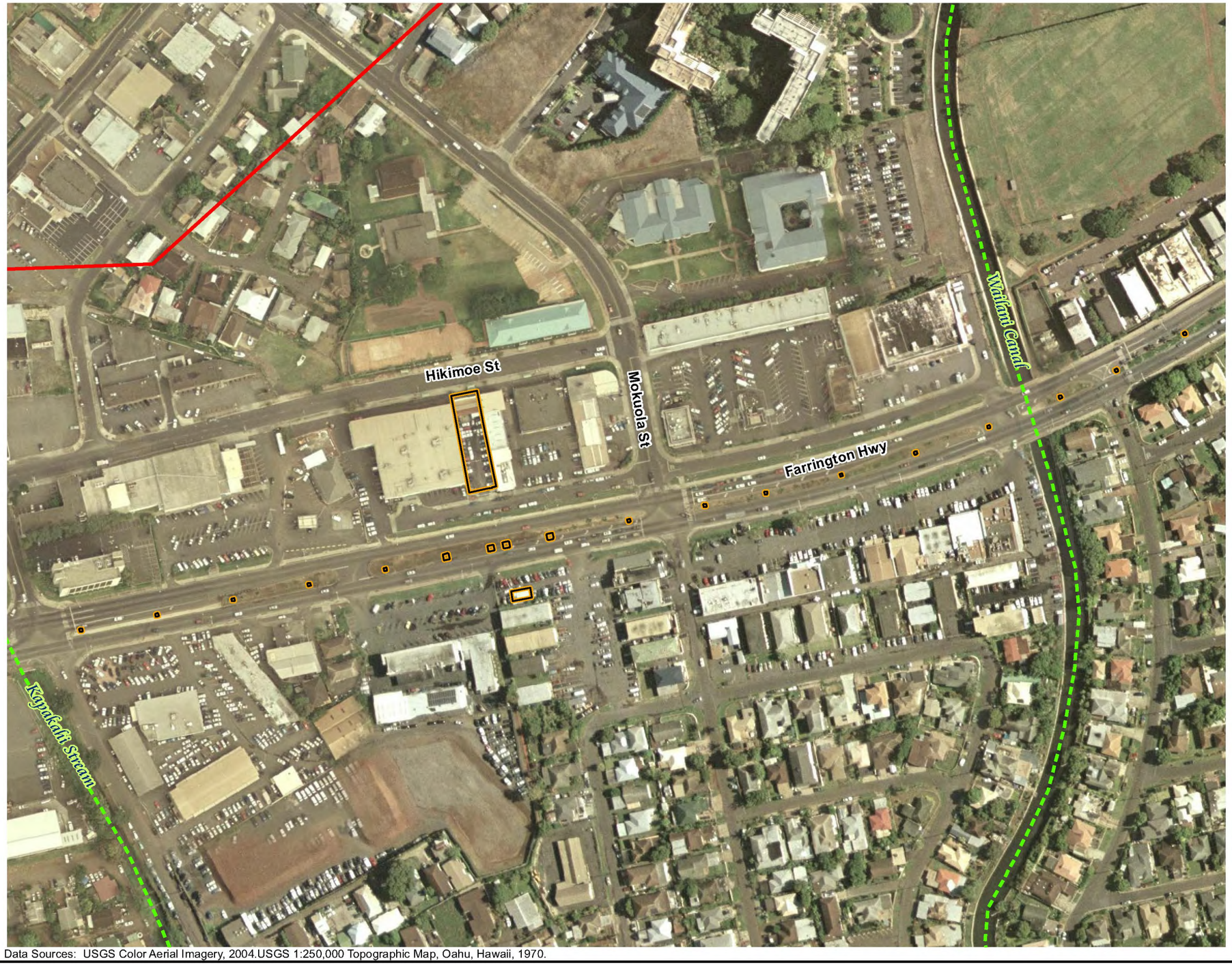
LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Streams & Canals

Velocity Difference (ft/s)

-3.0 -	0.1 - 0.5
-3.0 - -2.0	0.5 - 1.0
-2.0 - -1.0	1.0 - 2.0
-1.0 - -0.5	2.0 - 3.0
-0.5 - -0.1	3.0 +
-0.1 - 0.1 (transparent)	

Figure E-7 - 100-year Flood Velocity Differences in Vicinity of Project Station: With Levee, Project Alternative 2 (Coarse Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

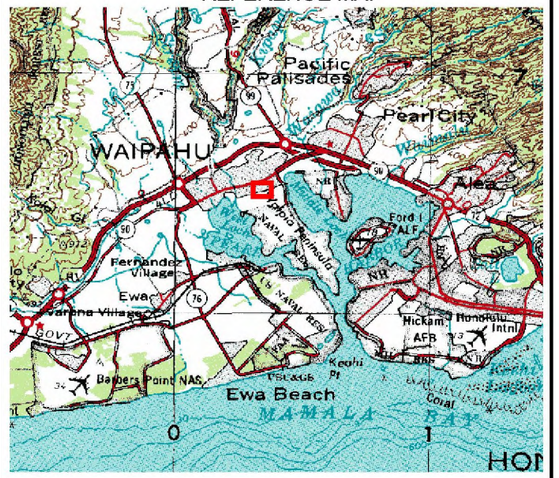
100-year Flood Velocity Differences in Vicinity of Project Station (Test25_P_North_NHC minus Test25)

Scale - 1:2,400 1 inch = 200 feet

0 100 200 400 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Streams & Canals

Velocity Difference (ft/s)

-3.0 -	0.1 - 0.5
-3.0 - -2.0	0.5 - 1.0
-2.0 - -1.0	1.0 - 2.0
-1.0 - -0.5	2.0 - 3.0
-0.5 - -0.1	3.0 +
-0.1 - 0.1 (transparent)	

Figure E-8 - 100-year Flood Velocity Differences in Vicinity of Project Station: With Levee, Project Alternative 3 (Coarse Grid Model)

Appendix F. Differential maps (no levee, project conditions minus existing conditions)

AR00041480

Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

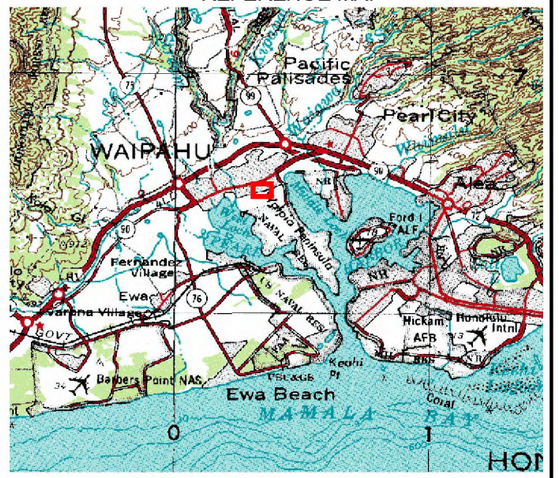
100-year Flood Maximum Inundation Depth Differences in Vicinity of Project Station (Test25NL_P minus Test25NL)

Scale - 1:2,400 1 inch = 200 feet

0 100 200 400 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



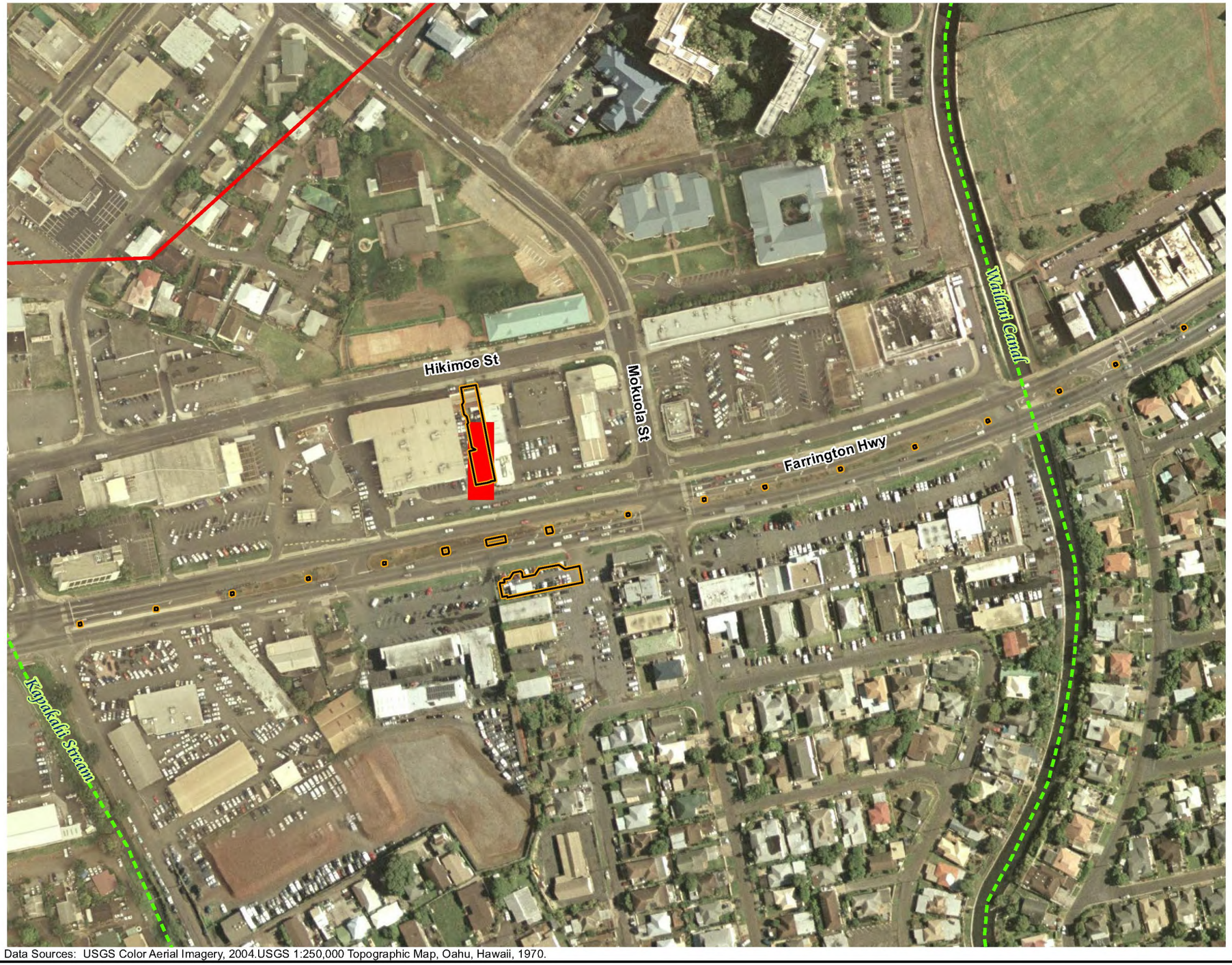
LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Streams & Canals

Depth Difference (ft)

-4.1 - -0.5	0.1 - 0.2
-0.5 - -0.4	0.2 - 0.3
-0.4 - -0.3	0.3 - 0.4
-0.3 - -0.2	0.4 - 0.5
-0.2 - -0.1	0.5 +
-0.1 - 0.1 (transparent)	

Figure F-1 - 100-year Flood Maximum Inundation Depth Differences in Vicinity of Project Station: No Levee, Original Project (Coarse Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

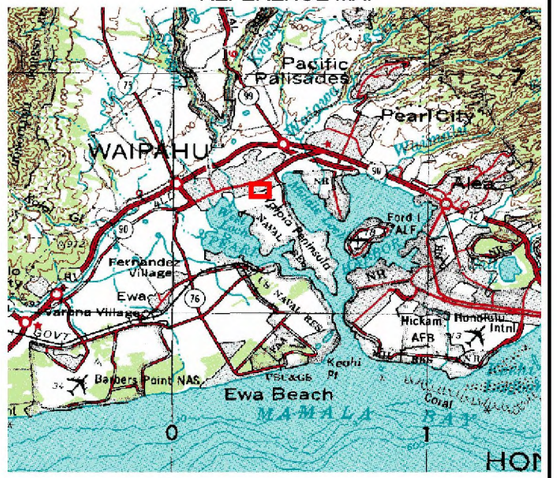
100-year Flood Maximum Inundation Depth Differences in Vicinity of Project Station (Test25NL_P_North minus Test25NL)

Scale - 1:2,400 1 inch = 200 feet

0 100 200 400 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Streams & Canals

Depth Difference (ft)

- -2.1- -0.5
- -0.5 - -0.4
- -0.4 - -0.3
- -0.3 - -0.2
- -0.2 - -0.1
- -0.1 - 0.1 (transparent)
- 0.1 - 0.2
- 0.2 - 0.3
- 0.3 - 0.4
- 0.4 - 0.5
- 0.5 +

Figure F-2 - 100-year Flood Maximum Inundation Depth Differences in Vicinity of Project Station: No Levee, Project Alternative 1 (Coarse Grid Model)

Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

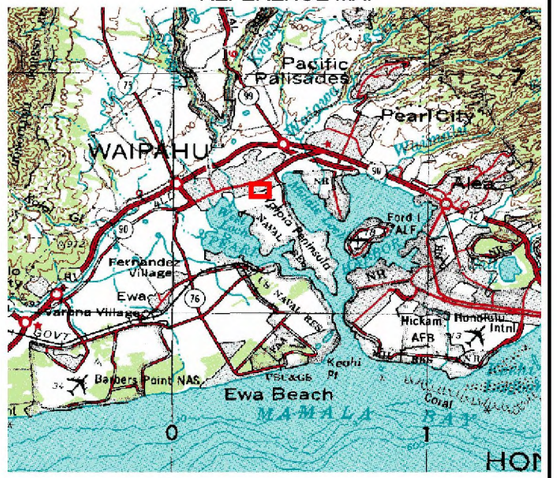
100-year Flood Maximum Inundation Depth Differences in Vicinity of Project Station (Test25NL_P_North_PB minus Test25NL)

Scale - 1:2,400 1 inch = 200 feet

0 100 200 400 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



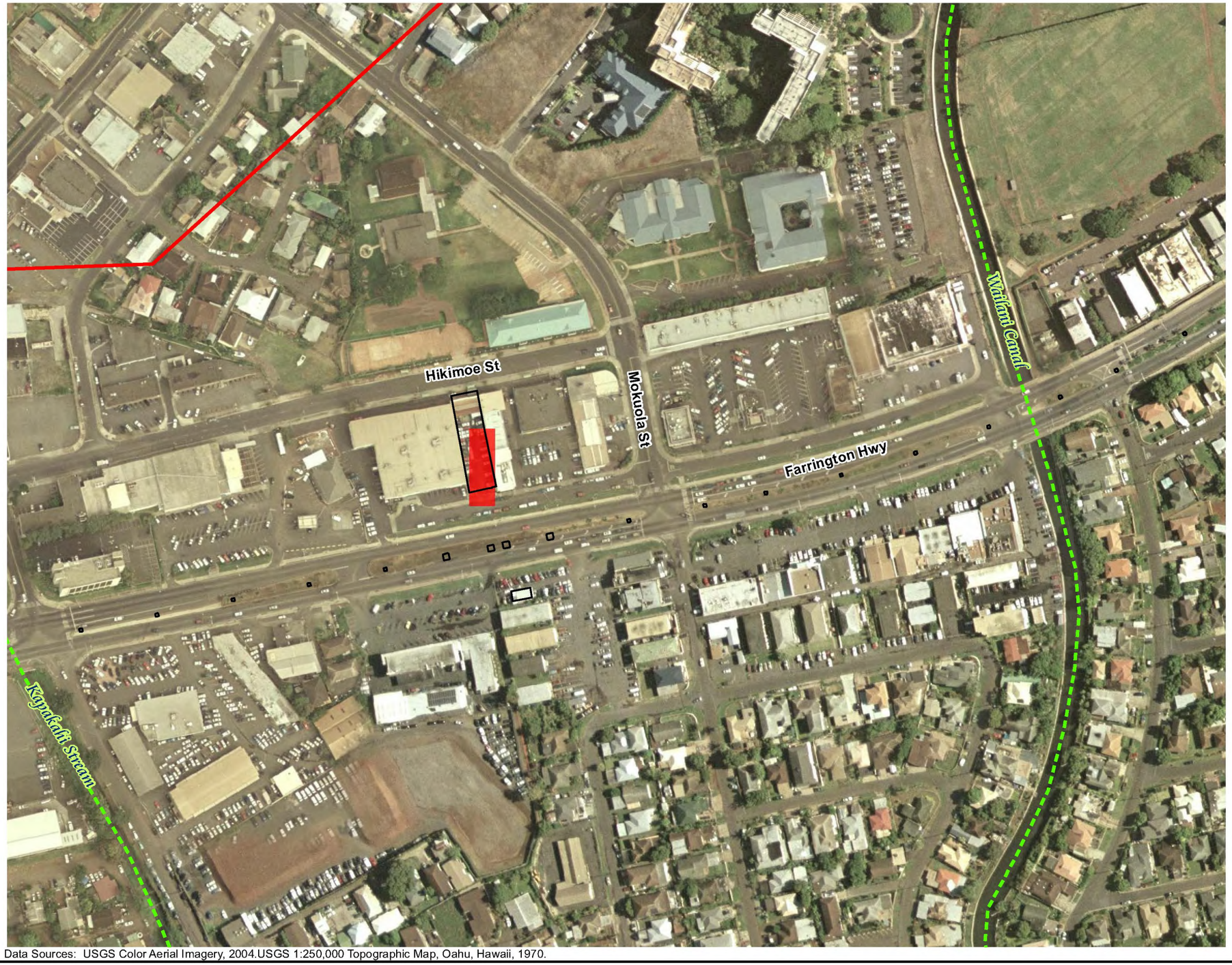
LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Streams & Canals

Depth Difference (ft)

- -4.1 - -0.5
- -0.5 - -0.4
- -0.4 - -0.3
- -0.3 - -0.2
- -0.2 - -0.1
- -0.1 - 0.1 (transparent)
- 0.1 - 0.2
- 0.2 - 0.3
- 0.3 - 0.4
- 0.4 - 0.5
- 0.5 +

Figure F-3 - 100-year Flood Maximum Inundation Depth Differences in Vicinity of Project Station: No Levee, Project Alternative 2 (Coarse Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

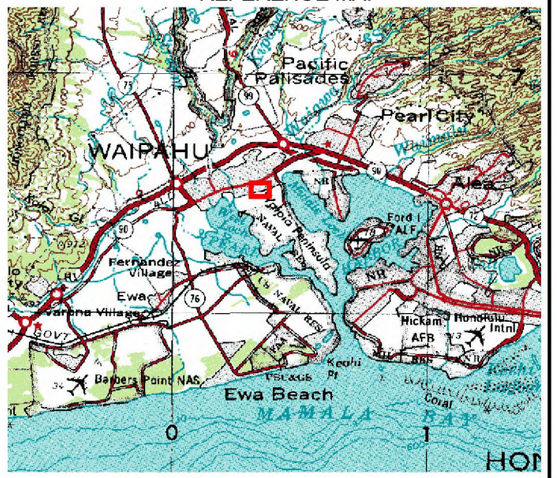
100-year Flood Maximum Inundation Depth Differences in Vicinity of Project Station (Test25NL_P_North_NHC minus Test25NL)

Scale - 1:2,400 1 inch = 200 feet

0 100 200 400 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



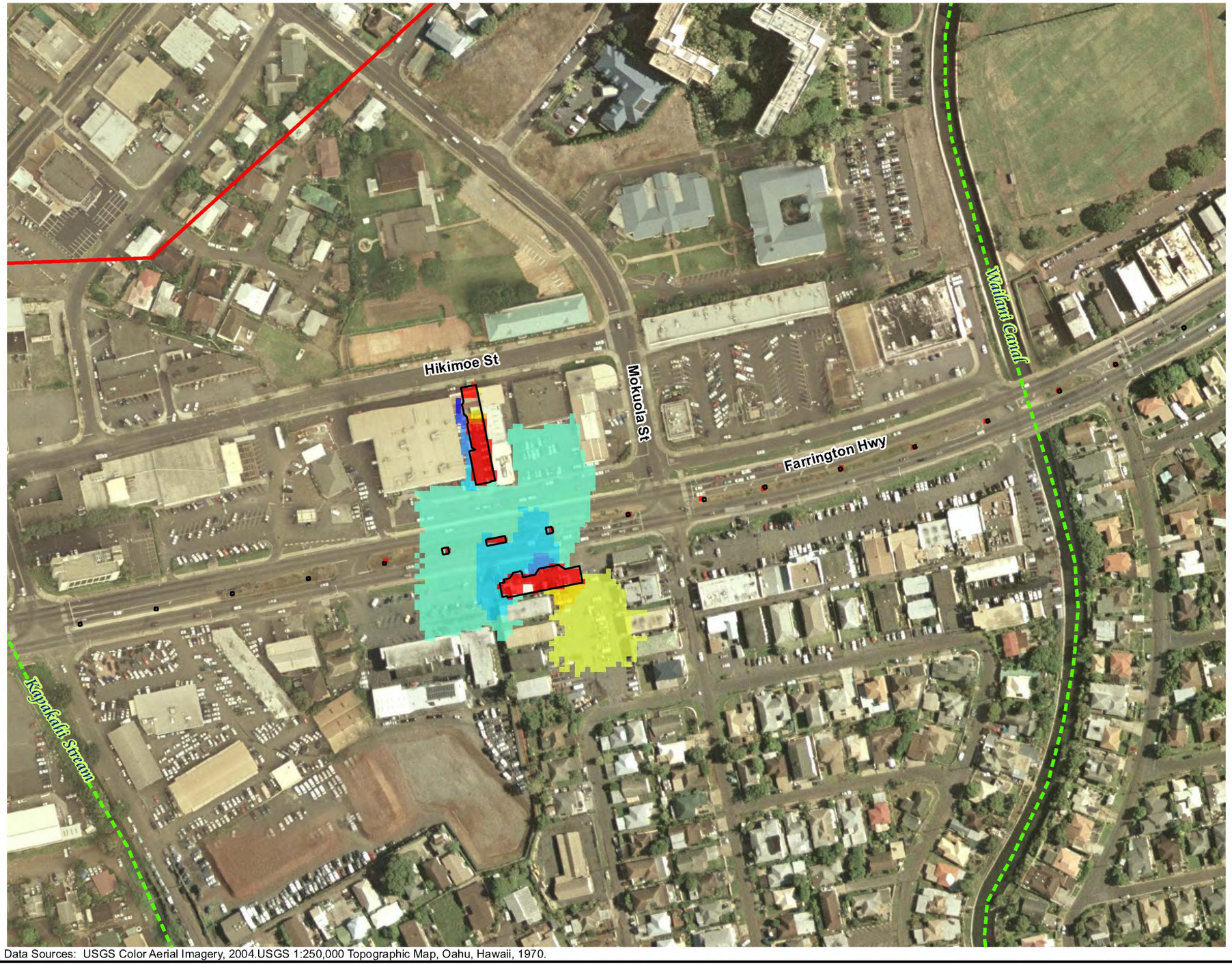
LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Streams & Canals

Depth Difference (ft)

-2.1 - -0.5	0.1 - 0.2
-0.5 - -0.4	0.2 - 0.3
-0.4 - -0.3	0.3 - 0.4
-0.3 - -0.2	0.4 - 0.5
-0.2 - -0.1	0.5 +
-0.1 - 0.1 (transparent)	

Figure F-4 - 100-year Flood Maximum Inundation Depth Differences in Vicinity of Project Station: No Levee, Project Alternative 3 (Coarse Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

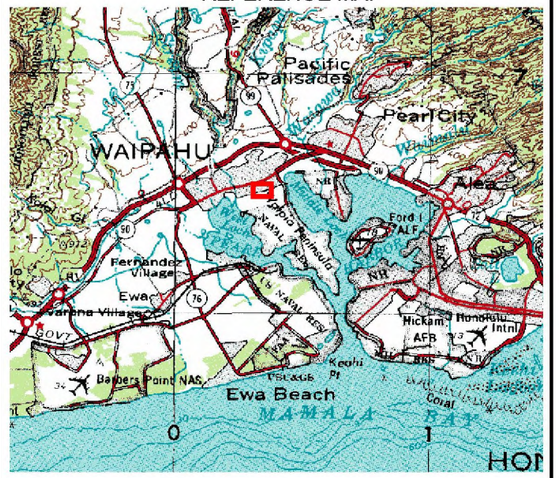
100-year Flood Maximum Inundation Depth Differences in Vicinity of Project Station (SD1_Test1NL_P minus SD1_Test1NL)

Scale - 1:2,400 1 inch = 200 feet

0 100 200 400 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



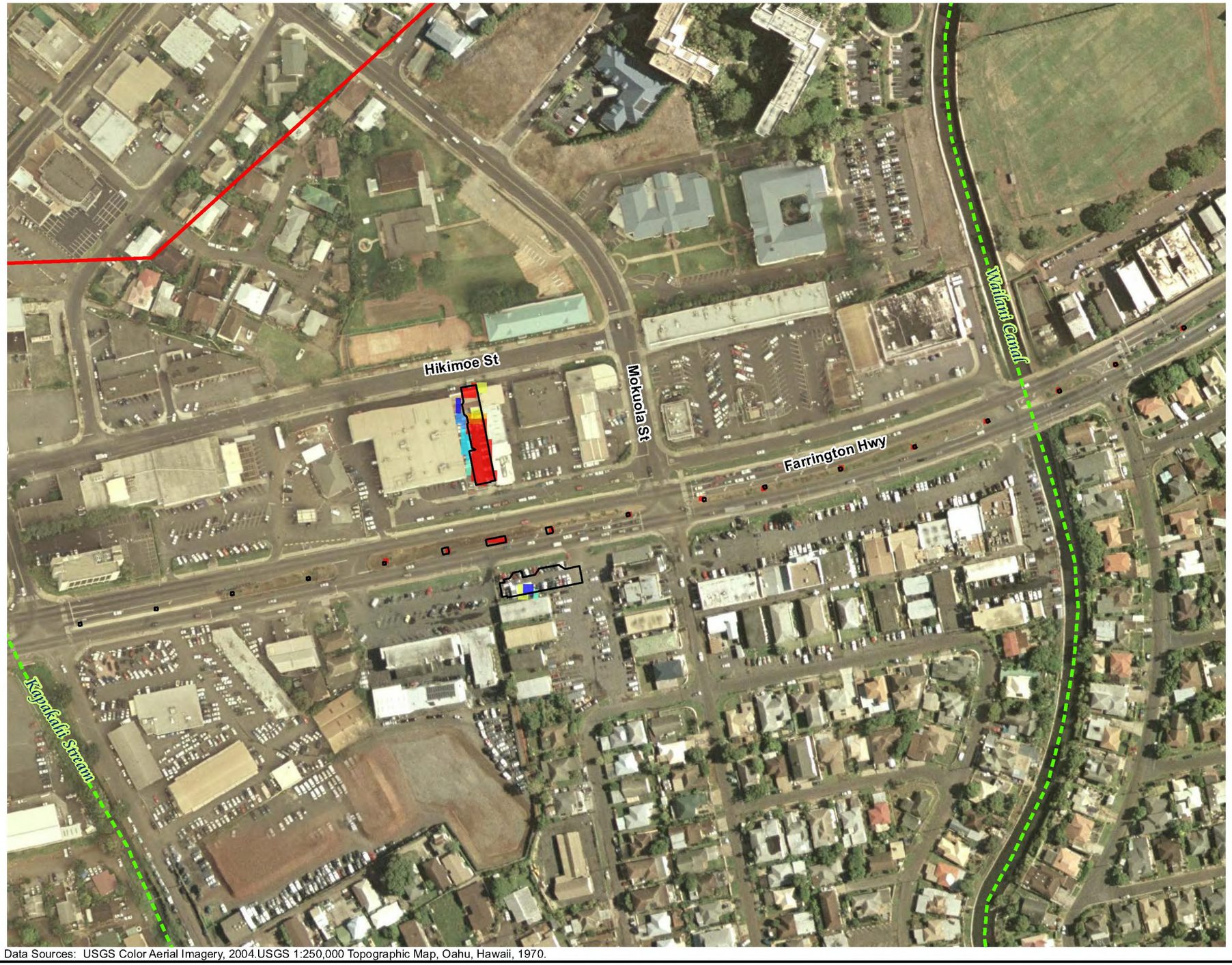
LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Streams & Canals

Depth Difference (ft)

-4.8 - -0.5	0.1 - 0.2
-0.5 - -0.4	0.2 - 0.3
-0.4 - -0.3	0.3 - 0.4
-0.3 - -0.2	0.4 - 0.5
-0.2 - -0.1	0.5 +
-0.1 - 0.1 (transparent)	

Figure F-5 - 100-year Flood Maximum Inundation Dept hDifferences in Vicinity of Project Station: No Levee, Original Project (Fine Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

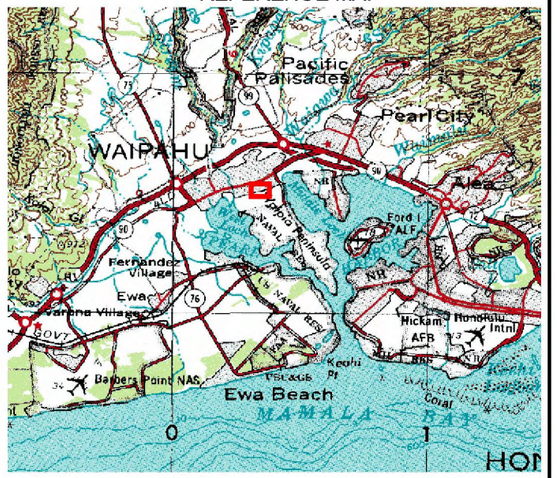
100-year Flood Maximum Inundation Depth Differences in Vicinity of Project Station (SD1_Test1NL_P_North minus SD1_Test1NL)

Scale - 1:2,400 1 inch = 200 feet

0 100 200 400 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Streams & Canals

Depth Difference (ft)

-4.8 - -0.5	0.1 - 0.2
-0.5 - -0.4	0.2 - 0.3
-0.4 - -0.3	0.3 - 0.4
-0.3 - -0.2	0.4 - 0.5
-0.2 - -0.1	0.5 +
-0.1 - 0.1 (transparent)	

Figure F-6 - 100-year Flood Maximum Inundation Depth Differences in Vicinity of Project Station: No Levee, Project Alternative 1 (Fine Grid Model)

Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

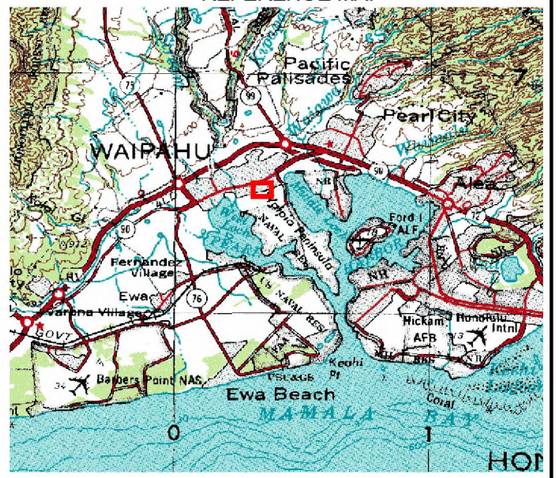
100-year Flood Maximum Inundation Depth Differences in Vicinity of Project Station (SD1_Test1NL_P_North_PB minus SD1_Test1NL)

Scale - 1:2,400 1 inch = 200 feet

0 100 200 400 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



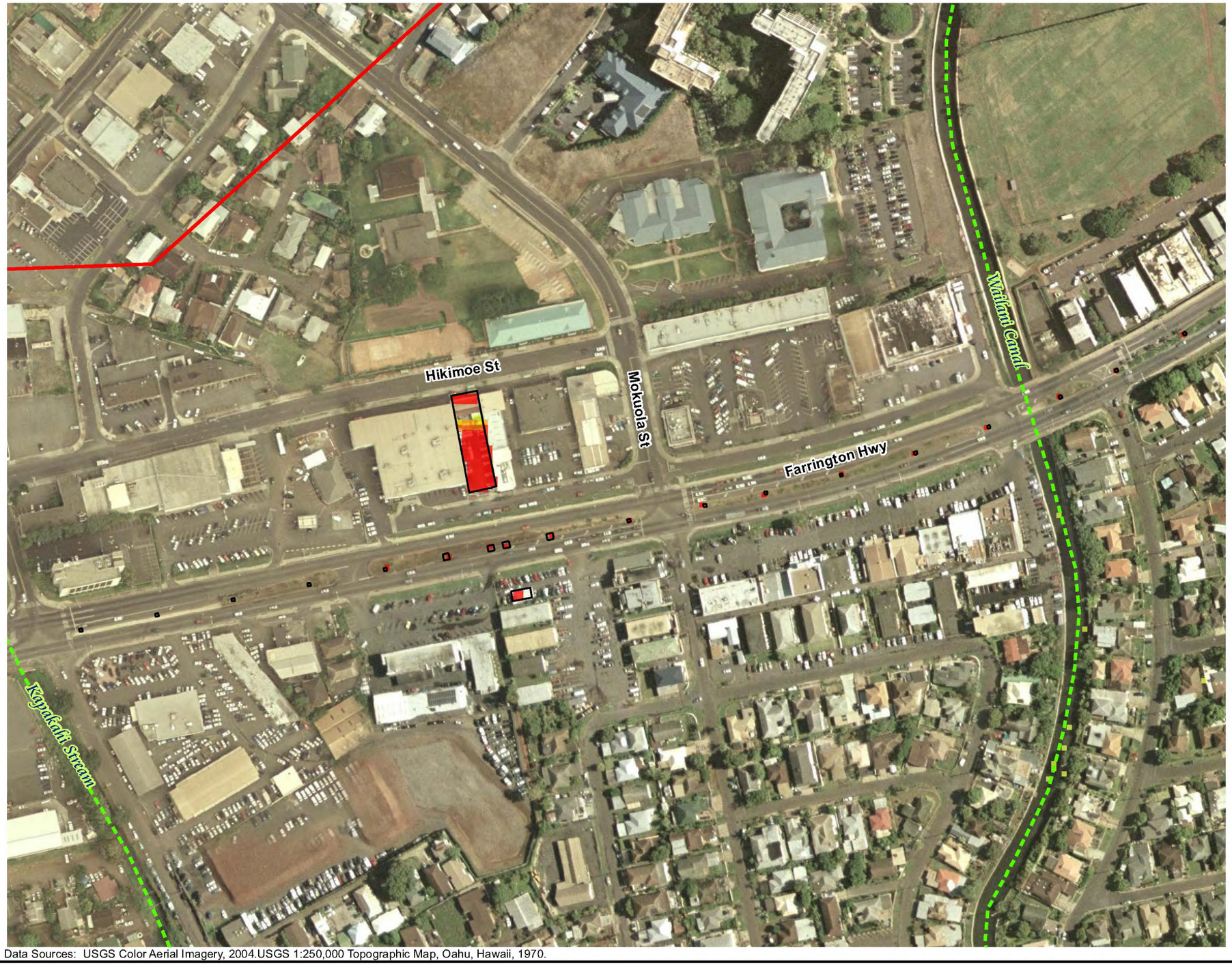
LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Streams & Canals

Depth Difference (ft)

-4.1 - -0.5	0.1 - 0.2
-0.5 - -0.4	0.2 - 0.3
-0.4 - -0.3	0.3 - 0.4
-0.3 - -0.2	0.4 - 0.5
-0.2 - -0.1	0.5 +
-0.1 - 0.1 (transparent)	

Figure F-7 - 100-year Flood Maximum Inundation Depth Differences in Vicinity of Project Station: No Levee, Project Alternative 2 (Fine Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

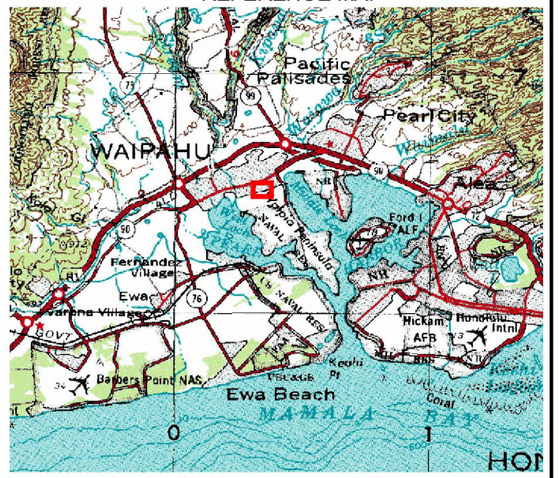
100-year Flood Maximum Inundation Depth Differences in Vicinity of Project Station (SD1_Test1NL_P_North_NHC minus SD1_Test1NL)

Scale - 1:2,400 1 inch = 200 feet

0 100 200 400 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



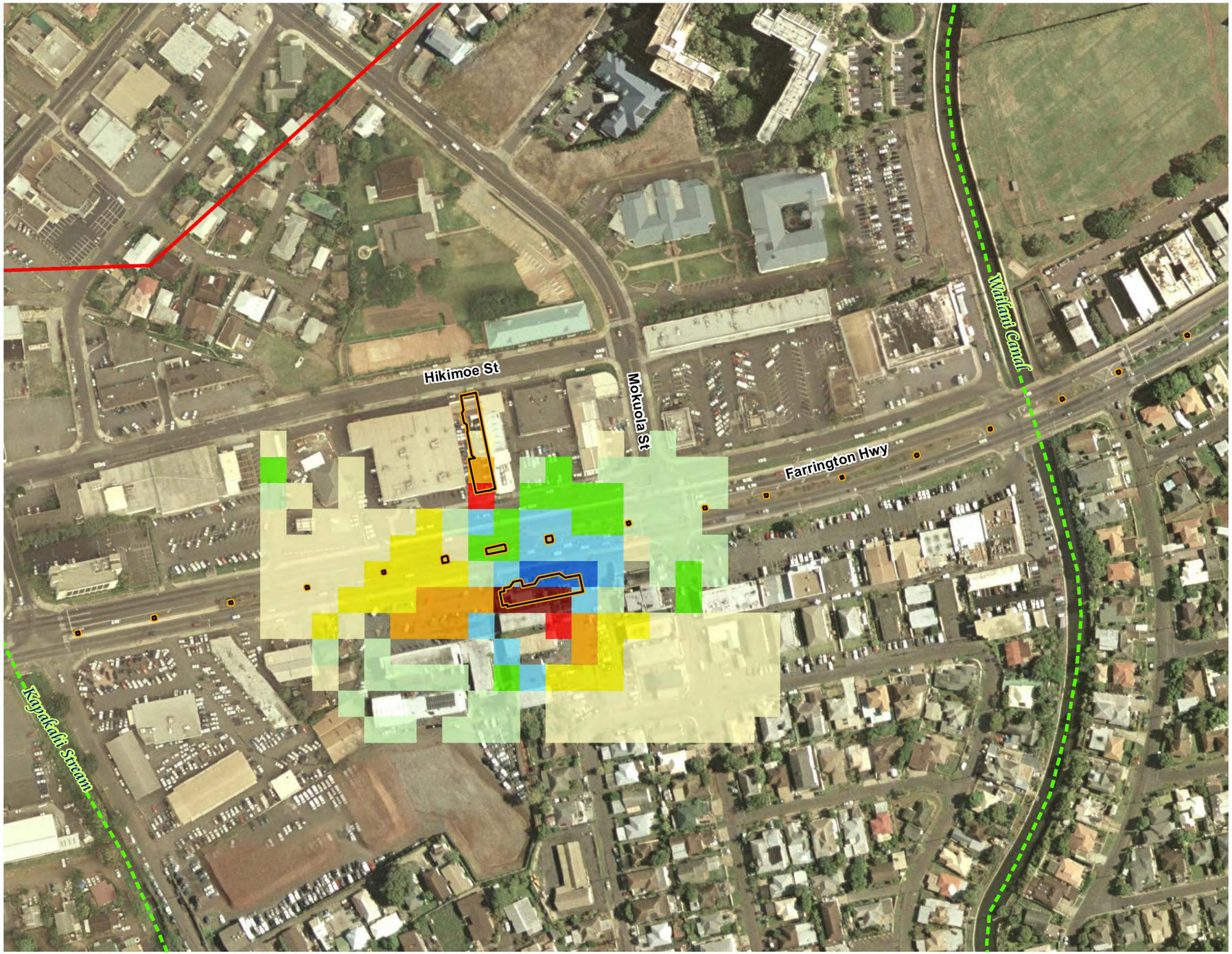
LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Streams & Canals

Depth Difference (ft)

- -4.8 - -0.5
- -0.5 - -0.4
- -0.4 - -0.3
- -0.3 - -0.2
- -0.2 - -0.1
- -0.1 - 0.1 (transparent)
- 0.1 - 0.2
- 0.2 - 0.3
- 0.3 - 0.4
- 0.4 - 0.5
- 0.5 +

Figure F-8 - 100-year Flood Maximum Inundation Depth Differences in Vicinity of Project Station: No Levee, Project Alternative 3 (Fine Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

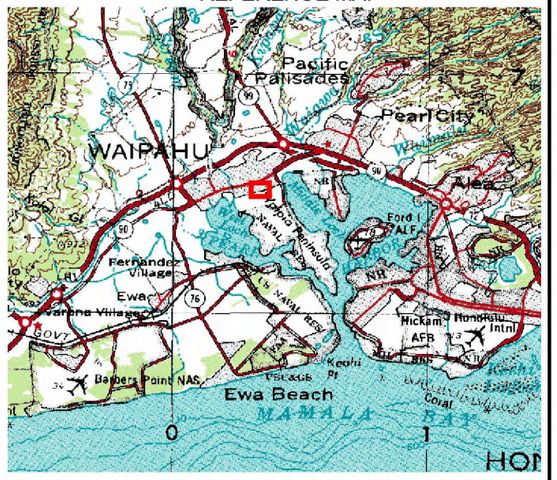
100-year Flood Velocity Differences in Vicinity of Project Station (Test25NL_P minus Test25NL)

Scale - 1:2,400 1 inch = 200 feet

0 100 200 400 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



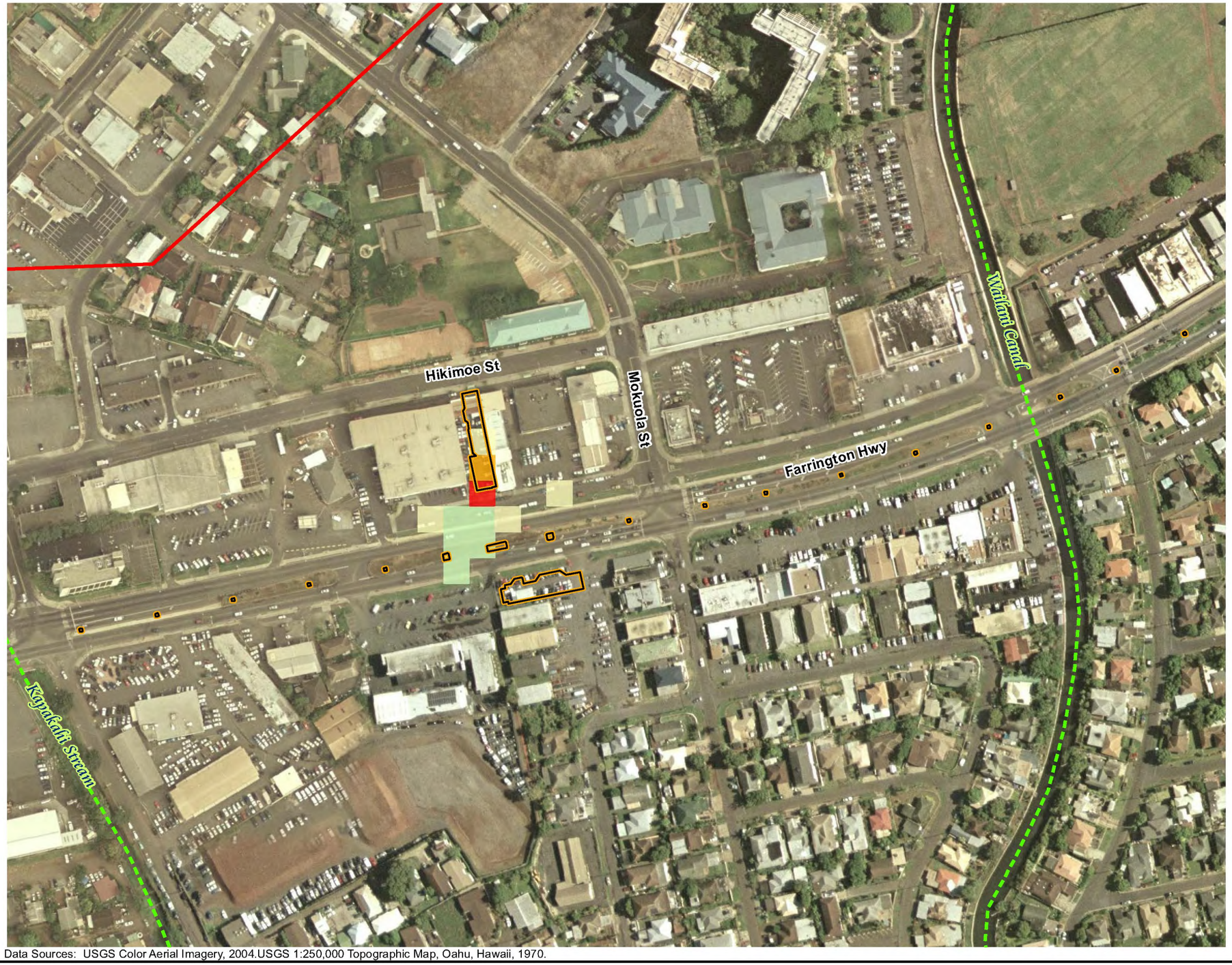
LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Streams & Canals

Velocity Difference (ft/s)

-3.0 -	0.1 - 0.5
-3.0 - -2.0	0.5 - 1.0
-2.0 - -1.0	1.0 - 2.0
-1.0 - -0.5	2.0 - 3.0
-0.5 - -0.1	3.0 +
-0.1 - 0.1 (transparent)	

Figure F-9 - 100-year Flood Velocity Differences in Vicinity of Project Station: No Levee, Original Project (Coarse Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

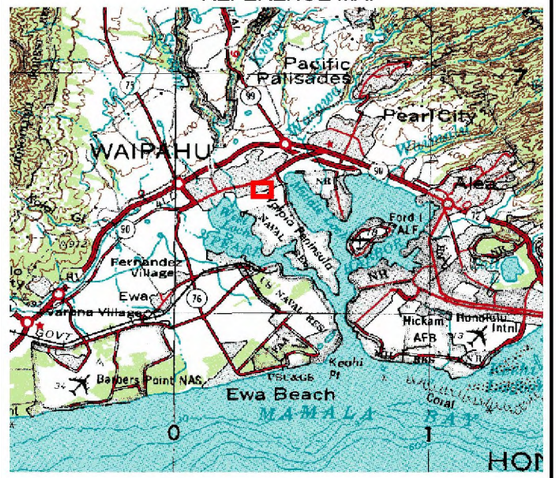
100-year Flood Velocity Differences in Vicinity of Project Station (Test25NL _P_North minus Test25NL)

Scale - 1:2,400 1 inch = 200 feet

0 100 200 400 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



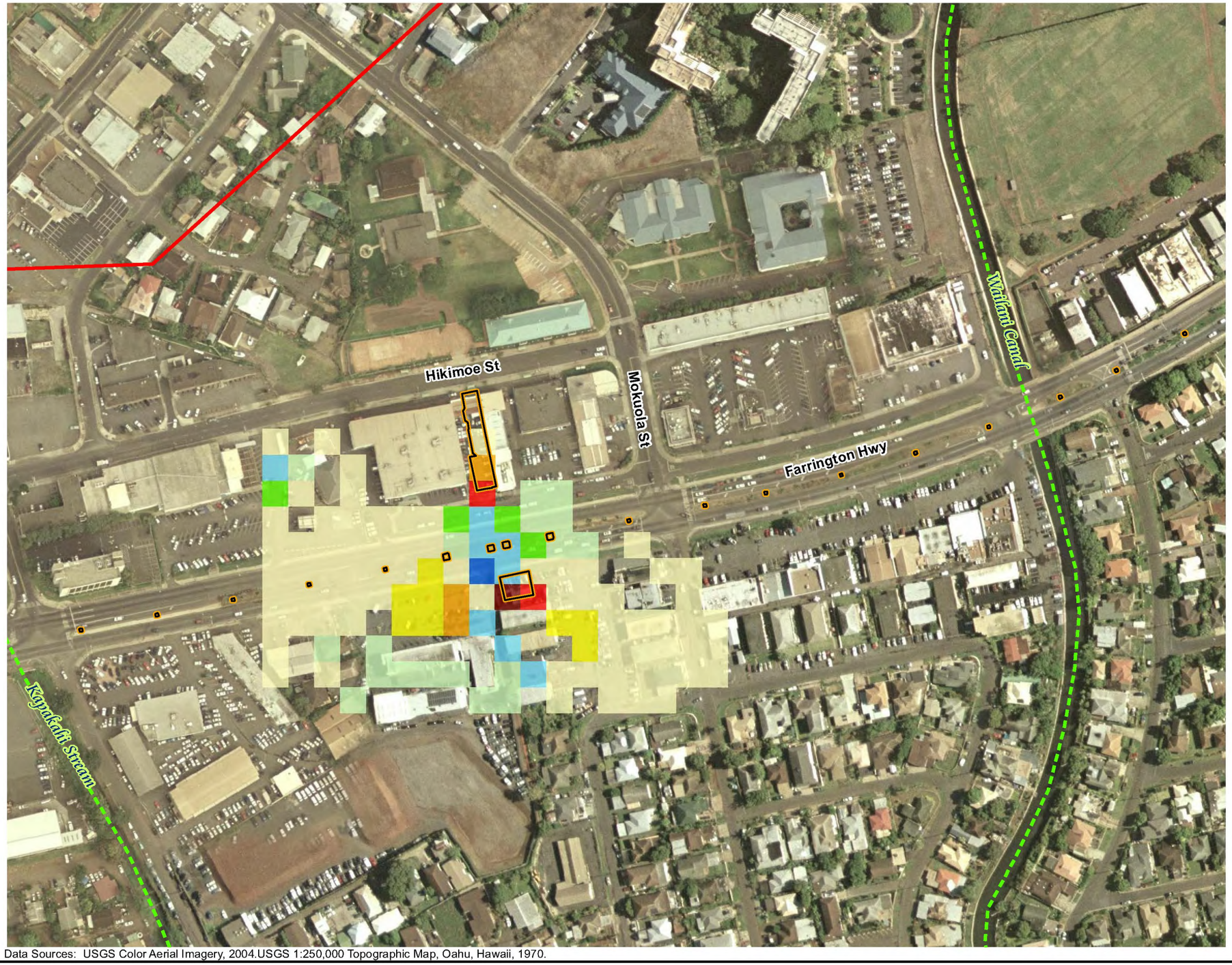
LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Streams & Canals

Velocity Difference (ft/s)

-3.0 -	0.1 - 0.5
-3.0 - -2.0	0.5 - 1.0
-2.0 - -1.0	1.0 - 2.0
-1.0 - -0.5	2.0 - 3.0
-0.5 - -0.1	3.0 +
-0.1 - 0.1 (transparent)	

Figure F-10 - 100-year Flood Velocity Differences in Vicinity of Project Station: No Levee, Project Alternative 1 (Coarse Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

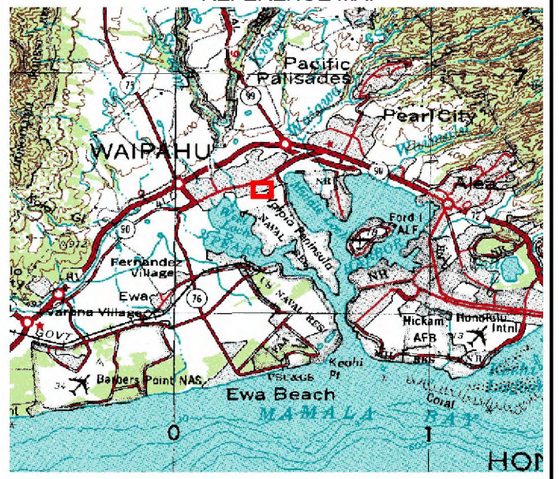
100-year Flood Velocity Differences in Vicinity of Project Station (Test25NL_P_North_PB minus Test25NL)

Scale - 1:2,400 1 inch = 200 feet

0 100 200 400 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



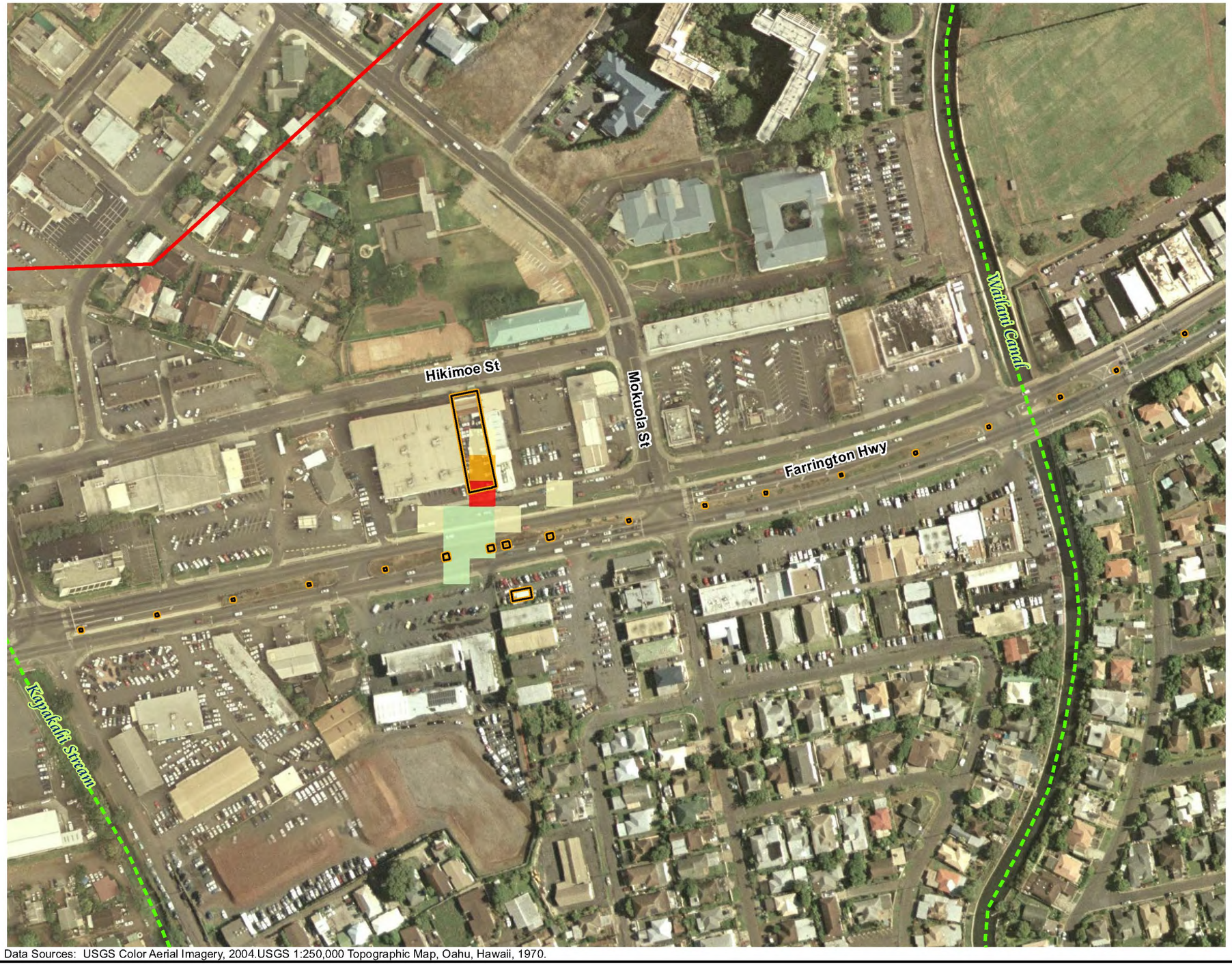
LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Streams & Canals

Velocity Difference (ft/s)

-3.0 -	0.1 - 0.5
-3.0 - -2.0	0.5 - 1.0
-2.0 - -1.0	1.0 - 2.0
-1.0 - -0.5	2.0 - 3.0
-0.5 - -0.1	3.0 +
-0.1 - 0.1 (transparent)	

Figure F-11 - 100-year Flood Velocity Differences in Vicinity of Project Station: No Levee, Project Alternative 2 (Coarse Grid Model)



Data Sources: USGS Color Aerial Imagery, 2004.USGS 1:250,000 Topographic Map, Oahu, Hawaii, 1970.

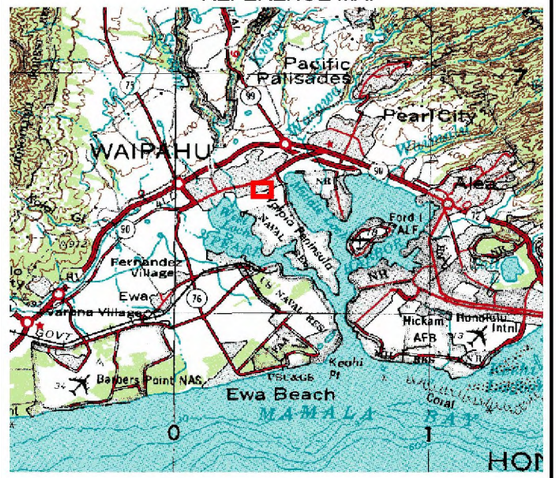
100-year Flood Velocity Differences in Vicinity of Project Station (Test25NL_P_North_NHC minus Test25NL)

Scale - 1:2,400 1 inch = 200 feet

0 100 200 400 Feet

HI State Plane, Zone 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	November 2009

REFERENCE MAP



LEGEND

- Model Area
- Project Buildings and Piers/Columns
- Streams & Canals

Velocity Difference (ft/s)

-3.0 -	0.1 - 0.5
-3.0 - -2.0	0.5 - 1.0
-2.0 - -1.0	1.0 - 2.0
-1.0 - -0.5	2.0 - 3.0
-0.5 - -0.1	3.0 +
-0.1 - 0.1 (transparent)	

Figure F-12 - 100-year Flood Velocity Differences in Vicinity of Project Station: No Levee, Project Alternative 3 (Coarse Grid Model)

NO-RISE ANALYSIS FOR HHCTCP GUIDEWAY COLUMNS - WAIAWA STREAM

DRAFT

Prepared for:

Parsons Brinckerhoff Inc.
1001 Bishop Street, Suite 2400
Honolulu, HI 96813
808-531-7094

Prepared by:

Northwest Hydraulic Consultants Inc.
16300 Christensen Road, Suite 350
Seattle, WA 98188
206-241-6000

April 30, 2010

EXECUTIVE SUMMARY

The proposed Honolulu High-Capacity Transit Corridor Project (HHCTCP) is an elevated rail line that will provide rapid public transportation service in the City and County of Honolulu on the island of Oahu, Hawaii. Northwest Hydraulic Consultants (NHC) was retained to perform hydraulic evaluations with respect to and as required for planning and design of the proposed elevated guideway crossing of Waiawa Stream.

The elevated guideway will be supported on eight concrete columns located within the floodway of Waiawa Stream. The City and County of Honolulu Department of Planning and Permitting (DPP) is the lead agency responsible for Flood Hazard District compliance within the City and County of Honolulu. As mandated by FEMA, DPP requires that all segments of the HHCTCP comply with a "No-Rise" policy, i.e. have no impact on 100-year flood water levels within floodway zones.

This report contains the data required to demonstrate that the proposed guideway will cause "No-Rise". The guideway will be constructed first as part of the West Oahu / Farrington Design-Build (WOF D-B) Contract, followed at a later time by the Pearl Highlands Station and Park-and-Ride Facility. Consequently, we request that review and approval of the No-Rise analysis for the guideway be assigned first priority so that construction on that phase can begin. The No-Rise analysis for the Station / Park-and-Ride Facility is described in a separate report "No-Rise Analysis for Pearl Highlands Station and Park-and-Ride Facility – Waiawa Stream" (April 30, 2010).

The impact on water surface elevations in the floodway of the proposed guideway columns has been evaluated and compensation recommended to make sure the existing conveyance and volume are maintained. Information from the effective Flood Insurance Study (FIS) for Waiawa Stream has been used to perform hand conveyance calculations at each of the eight proposed columns. Conveyance has been calculated for existing conditions, with proposed column, and with proposed column and compensation measures. The conveyance calculations were used to design an excavated swale and floodplain benches to compensate for conveyance loss due to the proposed columns. The volume of material removed for the conveyance compensation features is much more than the volume of water within the floodway that would be displaced by the proposed columns; thus the design would satisfy the compensatory volume requirement.

Based on the provided conveyance and volume calculations, the proposed guideway construction project will not increase 100-year floodway water surface elevations and therefore satisfies the "No-Rise" requirement.

AR00041494

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AR00041495

No-Rise Analysis for HHCTCP Guideway Columns Waiawa Stream

1. INTRODUCTION

The elevated guideway for the new HHCTCP will cross Waiawa Stream near the Waiawa Interchange in the area known as the 'Banana Patch'. The guideway will be supported on eight concrete columns that will be located within the designated FEMA Floodway for Waiawa Stream. When structures are placed within the regulatory floodway, the project proponent must demonstrate that the features will cause "No-Rise" in the 100-year floodway water surface elevation. This report contains the data required to demonstrate that the proposed project will cause "No-Rise".

In addition to the guideway, the Pearl Highlands Station and Park-and-Ride Facility will eventually be constructed at this location. A separate report has been prepared to demonstrate that this facility meets the "No-Rise" requirement. The report is titled "No-Rise Analysis for Pearl Highlands Station and Park-and-Ride Facility – Waiawa Stream" (April 30, 2010). We have elected to submit separate "No-Rise" reports for the guideway and the Station / Park-and-Ride Facility to the City and County of Honolulu Department of Transportation Services, Rapid Transit Division (RTD) and Department of Planning and Permitting (DPP) because the two projects will be built at different times. The guideway will be built first, followed at a later time by the Station / Park-and-Ride Facility. Consequently, we request that review and approval of the No-Rise analysis for the guideway be assigned first priority so that construction on that phase can begin. Review and approval of the No-Rise analysis for the Station / Park-and-Ride Facility can then follow at a later time.

AR00041496

2. PROPOSED COLUMN CONFIGURATION

The proposed column configuration for the guideway and the location of the FEMA Floodway are presented in Figure 1. There are eight guideway columns located within the effective FEMA Floodway. The columns will vary in size depending upon the load they carry. They will be either 6 feet square or 8 feet square. The amount of water that will be blocked by each column during a 100-year flood will vary and will depend upon the elevation of the ground surrounding each pier. The guideway itself, will be elevated well above the 100-year water surface elevation.

AR00041497

3. EVALUATION OF IMPACT: HAND CONVEYANCE CALCULATIONS

Explicit procedures have been provided by FEMA demonstrating how the conveyance calculations are to be computed. We have done this and the results are presented in Table 1. The effective FEMA FIS for Waiawa Stream was used to provide the Floodway water surface elevations. These were obtained directly from the effective HEC-2 model output provided by FEMA (Appendix B). The base flood elevation and cross section lines shown in the DFIRM were used to interpolate the floodway water surface at the proposed column locations. The Floodway boundaries along Waiawa Stream were used as shown in the DFIRM. The hand calculations were performed for a cross-section at each of the eight proposed columns located in the mapped Floodway (Figure 2). Plots of the cross-sections are attached in Appendix A.

The following hand calculations were made:

1. Calculated reduction in conveyance (K) caused by the proposed columns, assuming no change to floodway water-surface elevation, and using the appropriate roughness value obtained from the effective HEC-2 model of Waiawa Stream.
2. Designed compensation at each impacted location to maintain existing conveyance. Calculated the conveyance provided by the proposed compensation measures.
3. Demonstrated that the conveyance of the compensation measures would equal or exceed the loss of conveyance caused by the proposed columns.
4. Documented that the increase in effective conveyance provided will be maintained perpetually.

The hand conveyance calculations are summarized in Table 1 and detailed results are provided in Table 2 and Appendix C. The analysis shows that the compensation measures would properly mitigate any impact of the proposed guideway columns located in the floodway of Waiawa Stream.

AR00041498

4. PROPOSED MITIGATION

The proposed guideway columns will reduce the existing conveyance within the 100-year Floodway of Waiawa Stream; therefore, mitigation is required to eliminate the impact. Figure 3 shows the proposed conveyance compensation features. A conveyance swale would be excavated from approximately 100-ft upstream of cross-section 741+90 until being discharged into Waiawa Stream at cross-section 747+60. This excavated conveyance swale would be 15-ft wide at the bottom and tie into the existing ground at 2 to 1 side slopes. Typically, the swale would be 2 to 3 feet deep, with a top width of 20 to 25-ft.

Additional compensation features would be excavated at cross-section 748+50 and 749+40. The benches at these two cross-sections would be excavated to depths of about 2 to 3 feet to provide the required conveyance. All proposed mitigation features are located outside of the Waiawa Stream and Waiawa Springs delineated ordinary high water (PB 2009). The mitigation features would be lined with short grass or rock spalls, therefore a reduced roughness value of 0.07 has been applied for the excavated areas.

The guideway column at station 738+50 is located high on the right bank. Conveyance calculations at this cross-section show that no compensation is needed to maintain existing conditions conveyance. However, this column is still included in the compensatory volume analysis.

Plots of the proposed columns and compensation features at each cross-section are attached in Appendix A.

AR00041499

5. EVALUATION OF IMPACT: COMPENSATORY VOLUME

When an obstruction is placed within a Floodway one must demonstrate that the proposed features will not displace water and therefore reduce floodplain storage. The proposed excavation of the swale and the two benches will more than compensate for the storage lost due to the columns. As presented above, the columns will displace approximately 200 cubic yards of water within the floodway during a 100-year flood. The proposed excavation involves removing approximately 800 cubic yards, which is more than is needed to satisfy this requirement. Volume calculations have been included in Appendix C.

AR00041500

6. CONCLUSION

Based on the provided Conveyance and Volume calculations, the proposed guideway construction project will not increase 100-year floodway water surface elevations and therefore, satisfies the "No-Rise" requirement. The "Certification of a "No-Rise" Determination for a Proposed Floodway Development" form required by DPP is attached in Appendix D at the back of this document.

AR00041501

REFERENCES

FEMA (2004) "Flood Insurance Study: City and County of Honolulu, Hawaii", 15003CV001A, Federal Emergency Management Agency, Revised September 30, 2004.

FEMA (2005) "DFIRM (Digital Flood Insurance Rate Map)", 15003C, City and County of Honolulu, Hawaii, Federal Emergency Management Agency, June 2, 2005.

NHC (2010), "No-Rise Analysis for Pearl Highlands Station and Park-and-Ride Facility – Waiawa Stream", report prepared for Parsons Brinckerhoff, April 30, 2010.

PB (2009) "Wetland Waters of the U.S. Study: Honolulu High-Capacity Transit Corridor Project", Parsons Brinckerhoff, Prepared for City and County of Honolulu, July 10, 2009.

AR00041502

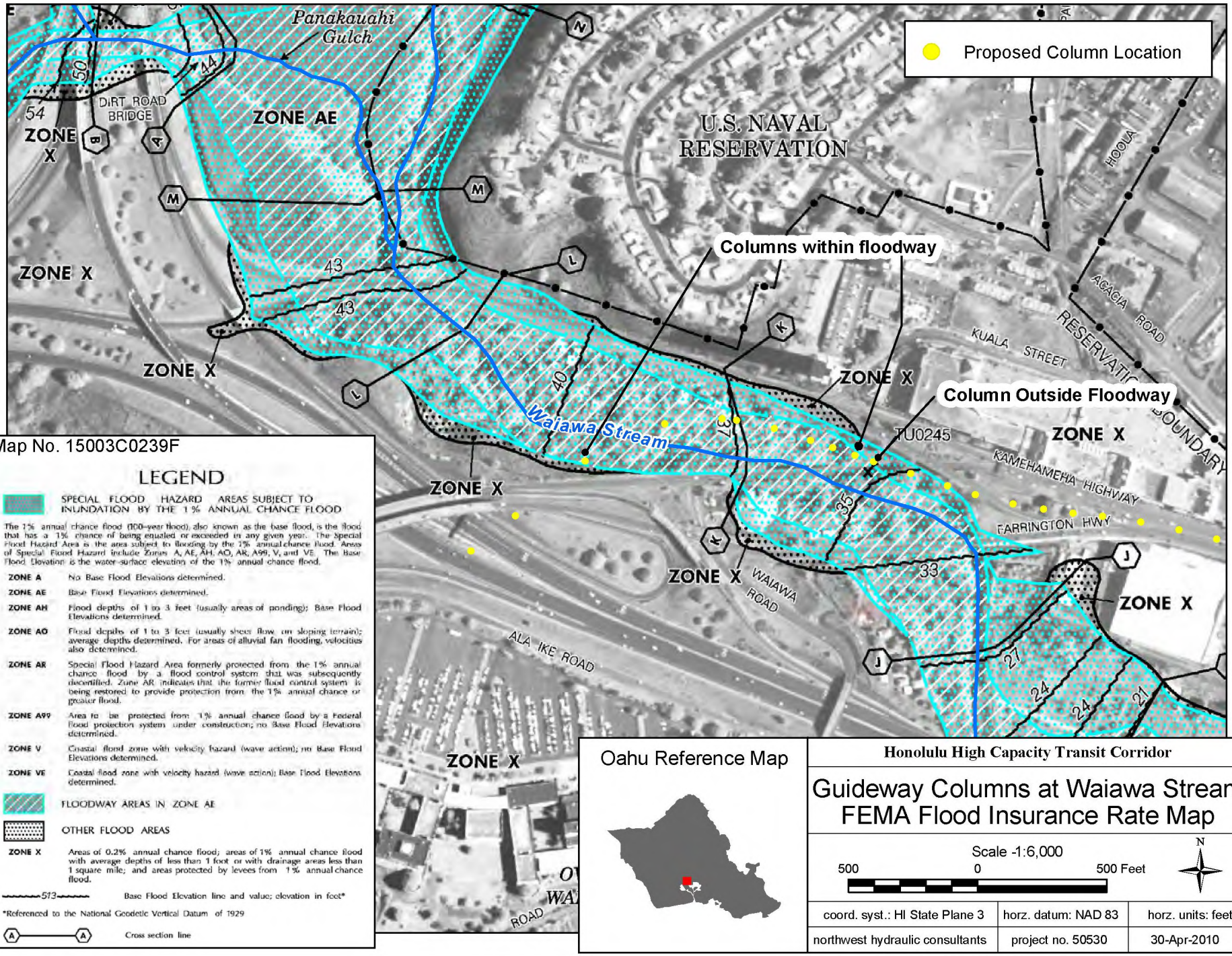


Figure 1

AR00041503

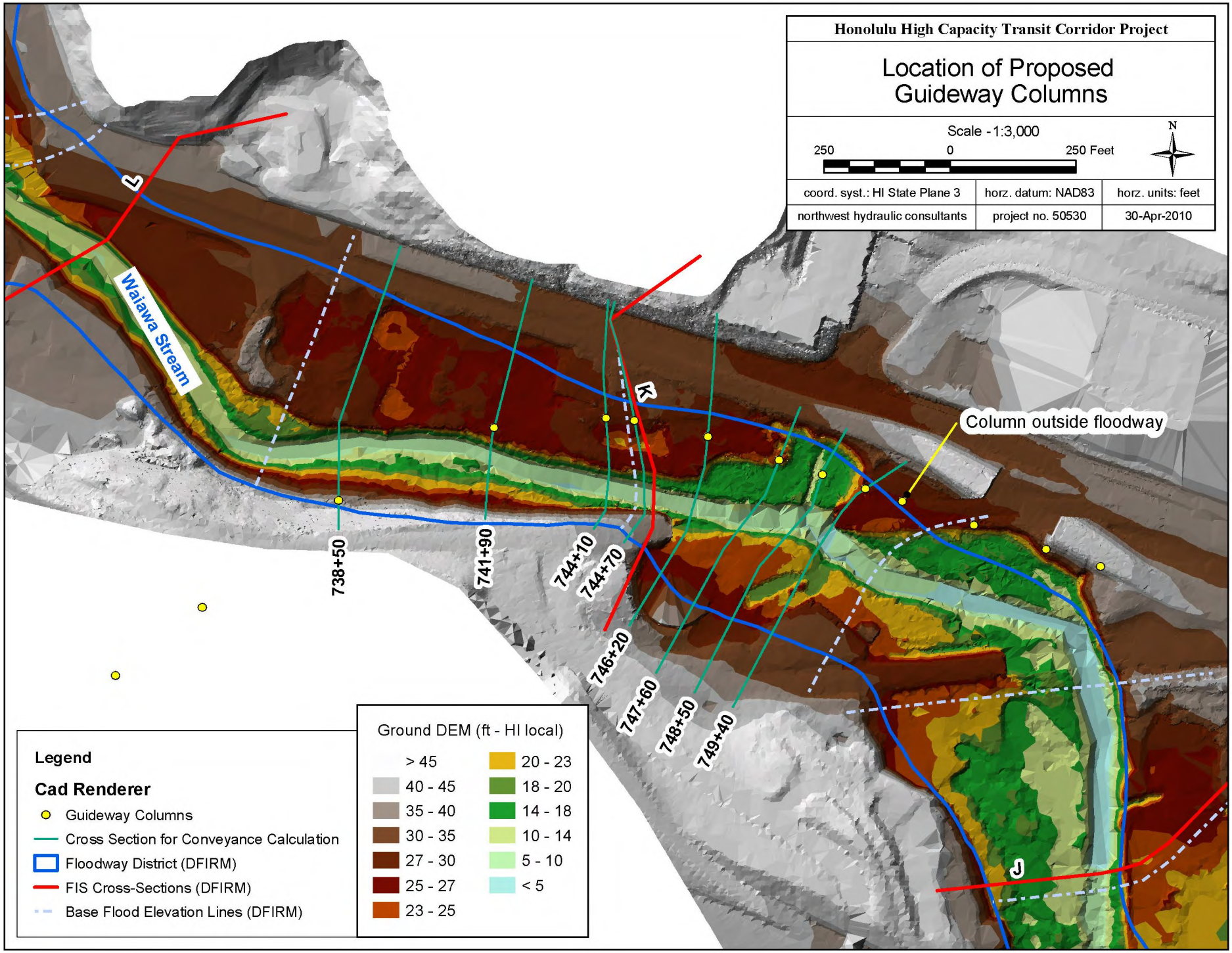


Figure 2

AR00041504

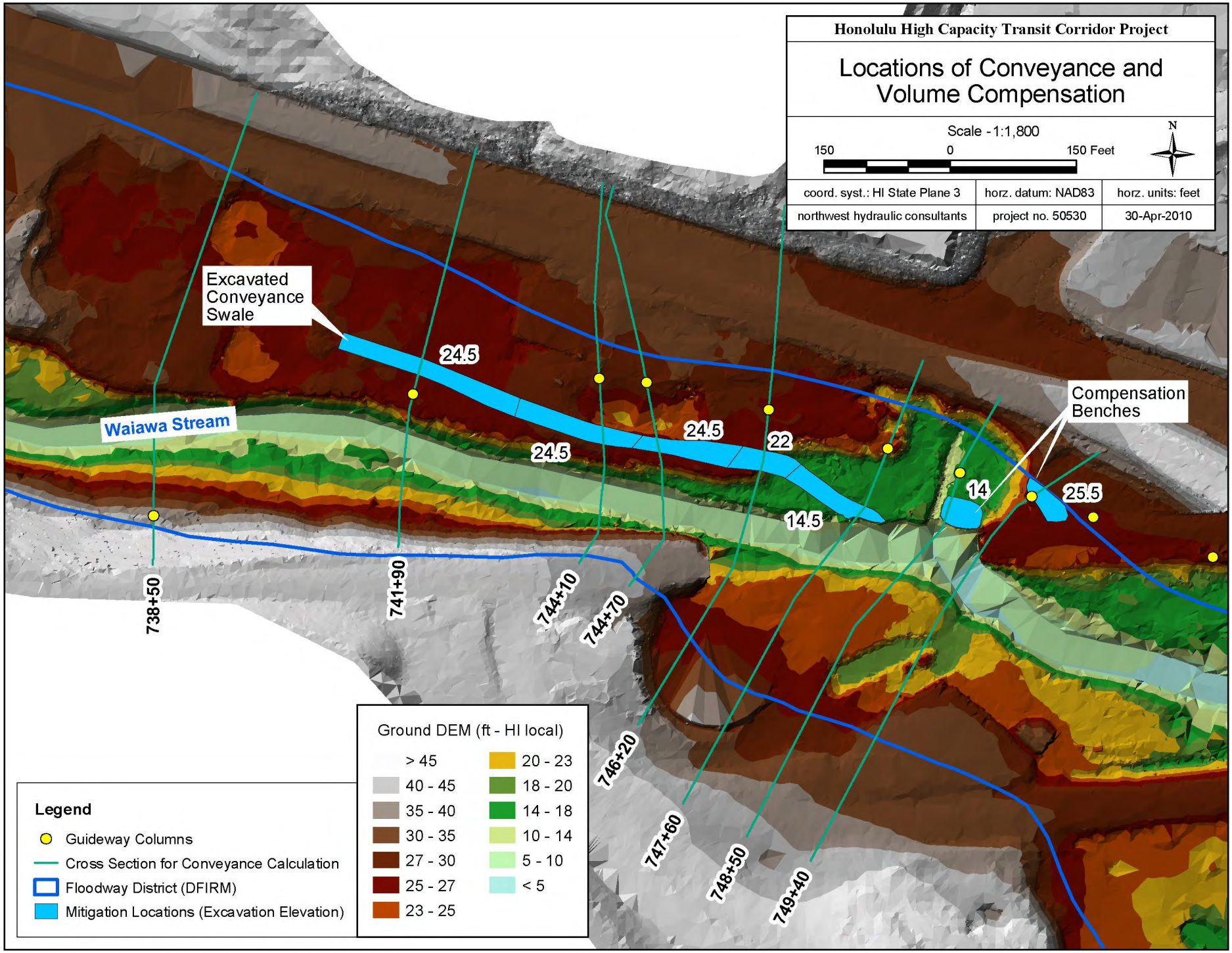


Figure 3

AR00041505

Table 1. Summary of Conveyance Calculations for Proposed Guideway Columns and Compensation Measures in Waiawa Stream Floodway

HHCTCP Column Station		Existing Conditions	With Proposed Guideway Columns		With Proposed Columns and Compensation Measures		
		Conveyance	Conveyance	Difference from Existing Conditions Conveyance	Conveyance	Difference from Existing Conditions Conveyance	Description / Comments
		cfs	cfs	cfs	cfs	cfs	
749+40	Left Overbank	114220	106984	-7236	115585	1365	Excavate and clear bench to 25.5-ft (about 2 ft deep). From Column to floodway boundary, about 15-ft wide along XS.
	Channel	288473	288473	0	288473	0	
	Right Overbank	232384	232384	0	232384	0	
	Total	635077	627841	-7236	636442	1365	
748+50	Left Overbank	270213	245971	-24243	271702	1489	Excavate and clear bench to 14.0-ft (about 2 to 3 ft deep). Locate at bank of Waiawa Stream, about 15-ft wide along XS.
	Channel	233679	233679	0	233679	0	
	Right Overbank	236147	236147	0	236147	0	
	Total	740039	715796	-24243	741528	1489	
747+60	Left Overbank	263450	251942	-11508	263959	509	Excavate and clear bench to 14.5-ft (about 2 to 3 ft deep). Locate at bank of Waiawa Stream, 15-ft wide at bottom and about 18-ft wide at top along XS.
	Channel	214721	214721	0	214721	0	
	Right Overbank	166568	166568	0	166568	0	
	Total	644739	633230	-11508	645248	509	
746+20	Left Overbank	168268	163303	-4965	173939	5671	Excavate and clear bench to 22.0-ft (about 2 to 3 ft deep). Locate at filled bank of Waiawa Stream, 15-ft wide at bottom and 25-ft wide at top along XS.
	Channel	242878	242878	0	242878	0	
	Right Overbank	125287	125287	0	125287	0	
	Total	536434	531469	-4965	542105	5671	
744+70	Left Overbank	146731	142926	-3804	150060	3329	Excavate and clear bench to 24.5-ft (about 5 ft deep). 15-ft wide at bottom and about 18-ft wide at top along XS.
	Channel	277353	277353	0	277353	0	
	Right Overbank	5027	5027	0	5027	0	
	Total	429111	425307	-3804	432440	3329	
744+10	Left Overbank	123765	118304	-5461	128783	5018	Excavate and clear bench to 24.5-ft (about 3 ft deep). 15-ft wide at bottom and about 22-ft wide at top along XS.
	Channel	300248	300248	0	300248	0	
	Right Overbank	9414	9414	0	9414	0	
	Total	433427	427967	-5461	438445	5018	
741+90	Left Overbank	199522	190882	-8640	199923	401	Excavate and clear bench to 24.5-ft (about 1 to 2 ft deep). 15-ft wide at bottom and about 20-ft wide at top along XS.
	Channel	328849	328849	0	328849	0	
	Right Overbank	52071	52071	0	52071	0	
	Total	580441	571802	-8640	580842	401	
738+50	Left Overbank	279127	279127	0	279127	0	No compensation needed for conveyance, check volume.
	Channel	362137	362137	0	362137	0	
	Right Overbank	67556	70484	2928	70484	2928	
	Total	708820	711748	2928	711748	2928	

Note: Differences in conveyance are calculated such that positive values are an increase in conveyance over existing conditions, while negative values are a loss in conveyance from existing conditions.

Table 2. Detailed Conveyance Calculations for the Proposed Guideway Columns and Compensation Measures in Waiawa Stream Floodway

HHCTCP Column Station		Floodway Water Surface Elevation	Floodway Water Surface Slope	Existing Conditions				With Proposed Guideway Columns					With Proposed Columns and Compensation Measures			
				Weighted Roughness	Area	Wetted Perimeter	Conveyance	Column Width	Weighted Roughness	Area	Wetted Perimeter	Conveyance	Weighted Roughness	Area	Wetted Perimeter	Conveyance
		ft	ft/ft	n	sq. ft	ft	cfs	ft	n	sq. ft	ft	cfs	n	sq. ft	ft	cfs
749+40	Left Overbank	37.46	0.0019	0.081	1298.3	123.2	114220	8	0.071	1207.8	137.7	106984	0.068	1231.5	139.9	115585
	Channel			0.080	1961.7	88.1	288473		0.080	1961.7	88.1	288473	0.080	1961.7	88.1	288473
	Right Overbank			0.100	2722.6	197.8	232384		0.100	2722.6	197.8	232384	0.100	2722.6	197.8	232384
	Total				5982.5		635077			5892.0		627841		5915.8		636442
748+50	Left Overbank	37.46	0.0015	0.100	2568.2	136.3	270213	8	0.082	2384.7	174.3	245971	0.077	2418.2	173.7	271702
	Channel			0.043	1678.5	207.5	233679		0.043	1678.5	207.5	233679	0.043	1678.5	207.5	233679
	Right Overbank			0.100	2858.9	218.1	236147		0.100	2858.9	218.1	236147	0.100	2858.9	218.1	236147
	Total				7105.6		740039			6922.0		715796		6955.5		741528
747+60	Left Overbank	37.57	0.0015	0.100	2779.3	172.5	263450	8	0.090	2663.3	193.4	251942	0.087	2686.2	193.1	263959
	Channel			0.043	1539.1	189.6	214721		0.043	1539.1	189.6	214721	0.043	1539.1	189.6	214721
	Right Overbank			0.097	2165.0	193.3	166568		0.097	2165.0	193.3	166568	0.097	2165.0	193.3	166568
	Total				6483.5		644739			6367.5		633230		6390.4		645248
746+20	Left Overbank	37.75	0.0015	0.100	2118.1	171.3	168268	6	0.093	2058.3	187.5	163303	0.088	2082.6	187.8	173939
	Channel			0.080	1496.5	57.9	242878		0.080	1496.5	57.9	242878	0.080	1496.5	57.9	242878
	Right Overbank			0.086	1654.2	180.5	125287		0.086	1654.2	180.5	125287	0.086	1654.2	180.5	125287
	Total				5268.8		536434			5209.0		531469		5233.4		542105
744+70	Left Overbank	37.94	0.0037	0.100	1940.7	169.1	146731	6	0.094	1886.8	181.1	142926	0.091	1907.2	181.6	150060
	Channel			0.080	1672.1	62.7	277353		0.080	1672.1	62.7	277353	0.080	1672.1	62.7	277353
	Right Overbank			0.100	131.8	32.1	5027		0.100	131.8	32.1	5027	0.100	131.8	32.1	5027
	Total				3744.7		429111			3690.7		425307		3711.2		432440
744+10	Left Overbank	38.21	0.0037	0.100	1756.2	170.0	123765	8	0.093	1682.3	182.2	118304	0.089	1730.4	182.7	128783
	Channel			0.080	1869.0	73.5	300248		0.080	1869.0	73.5	300248	0.080	1869.0	73.5	300248
	Right Overbank			0.100	195.7	33.6	9414		0.100	195.7	33.6	9414	0.100	195.7	33.6	9414
	Total				3820.9		433427			3747.0		427967		3795.1		438445
741+90	Left Overbank	39.09	0.0037	0.100	2536.6	208.3	199522	8	0.093	2437.2	225.1	190882	0.090	2465.8	225.9	199923
	Channel			0.080	2131.0	89.0	328849		0.080	2131.0	89.0	328849	0.080	2131.0	89.0	328849
	Right Overbank			0.100	691.1	60.5	52071		0.100	691.1	60.5	52071	0.100	691.1	60.5	52071
	Total				5358.7		580441			5259.4		571802		5287.9		580842
738+50	Left Overbank	40.20	0.0037	0.100	3585.1	298.9	279127	8	0.100	3585.1	298.9	279127	0.100	3585.1	298.9	279127
	Channel			0.080	2149.4	78.7	362137		0.080	2149.4	78.7	362137	0.080	2149.4	78.7	362137
	Right Overbank			0.100	866.3	72.1	67556		0.098	861.1	68.6	70484	0.098	861.1	68.6	70484
	Total				6600.9		708820			6595.6		711748		6595.6		711748

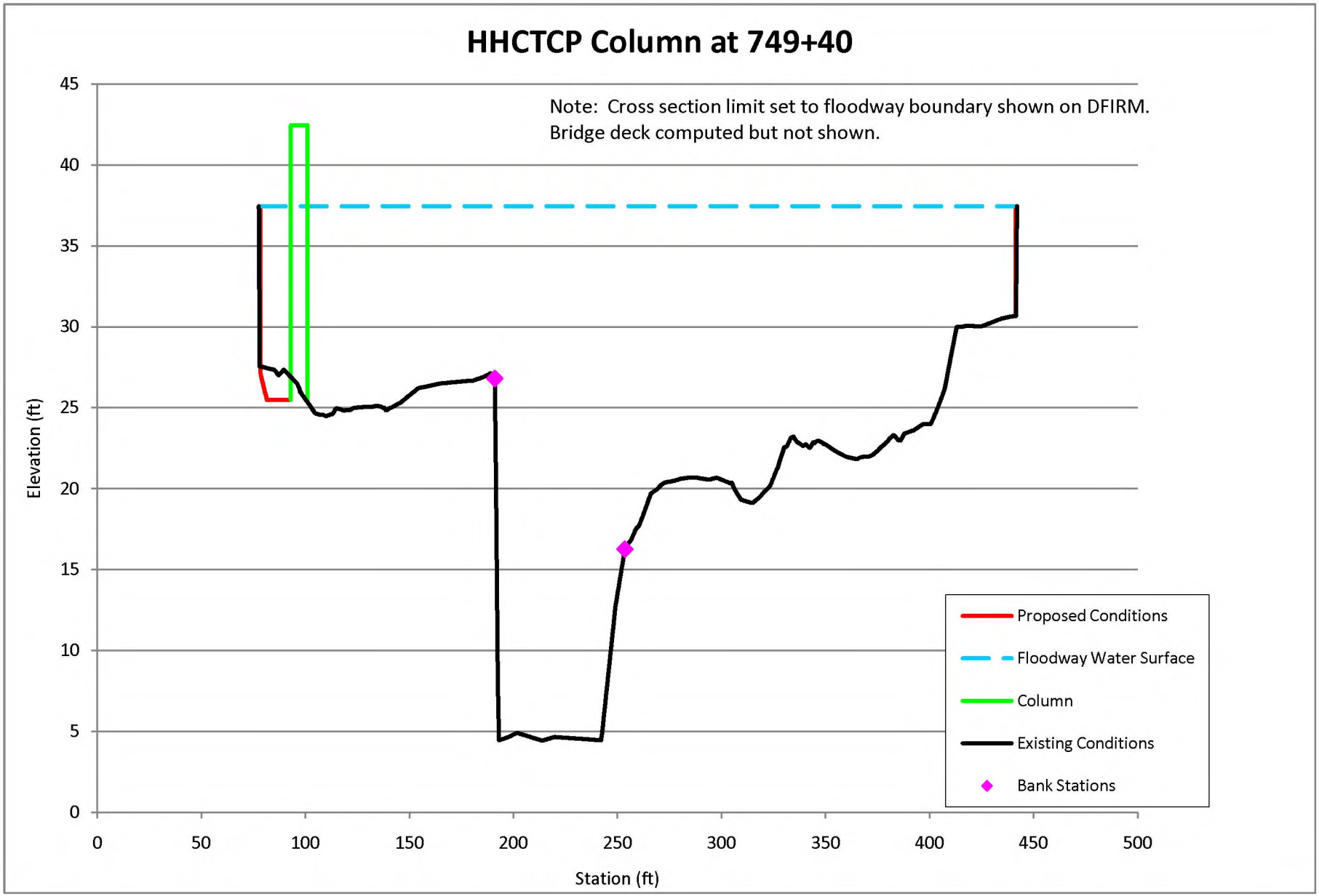
Note: 1) The following values were used for roughness based on the effective FIS HEC-2 model for Waiawa Stream. Overbank = 0.1; Channel = 0.08; Concrete = 0.013; Bare Earth = 0.07.

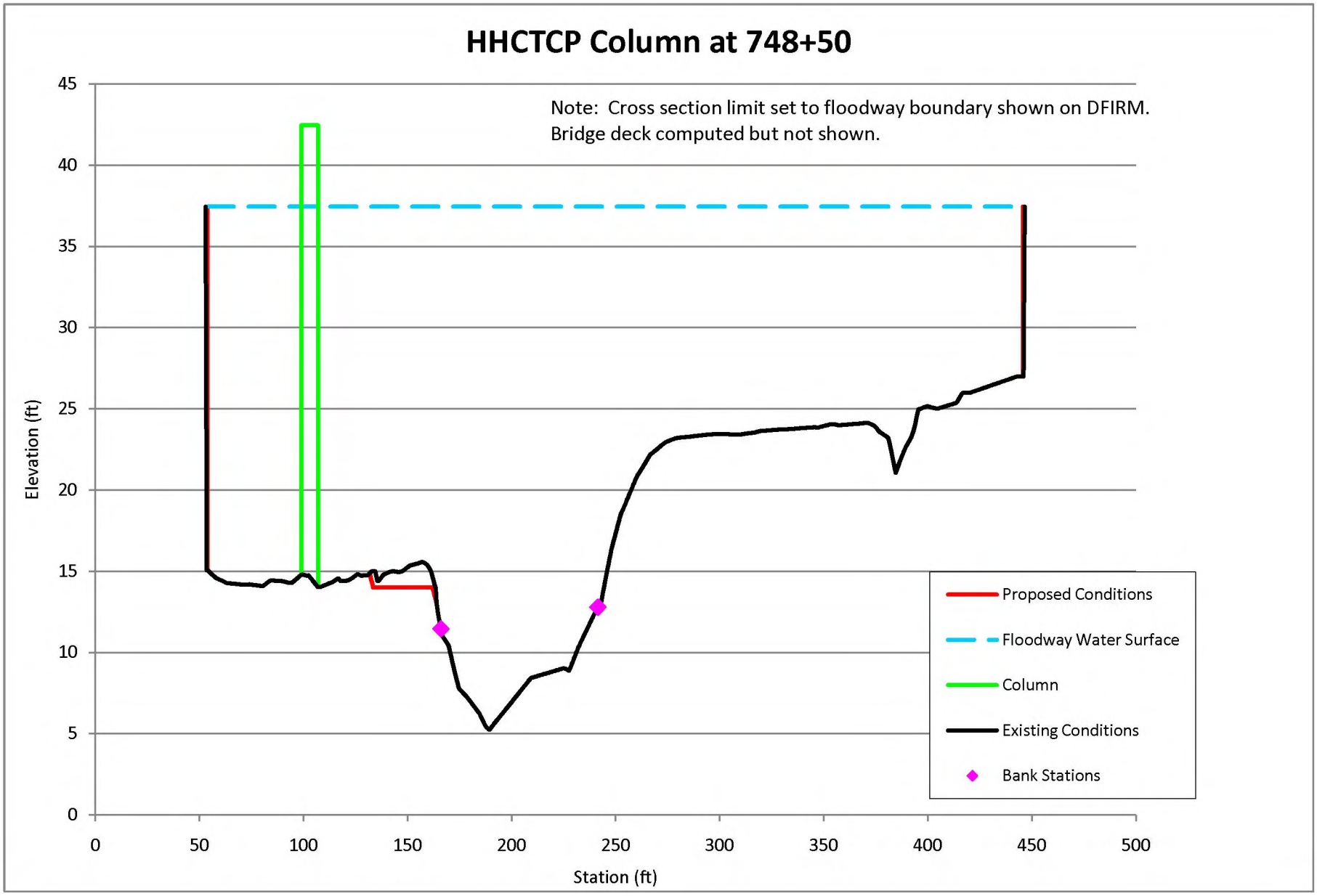
2) The water surface elevation and slope were obtained directly from the effective FIS HEC-2 model output table (wai-enc.out) provided by FEMA.

APPENDIX A

Plots of Cross-Sections Used in No-Rise Analysis
Existing and Proposed Conditions

AR00041508


HHCTCP Column at 749+40
Note: Cross section limit set to floodway boundary shown on DFIRM.
Bridge deck computed but not shown.
Elevation (ft)
Station (ft)
0
5
10
15
20
25
30
35
40
45
0
50
100
150
200
250
300
350
400
450
500
Proposed Conditions
Floodway Water Surface
Column
Existing Conditions
Bank Stations


HHCTCP Column at 748+50
Note: Cross section limit set to floodway boundary shown on DFIRM.
Bridge deck computed but not shown.
Elevation (ft)
Station (ft)
0
5
10
15
20
25
30
35
40
45
0
50
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150
200
250
300
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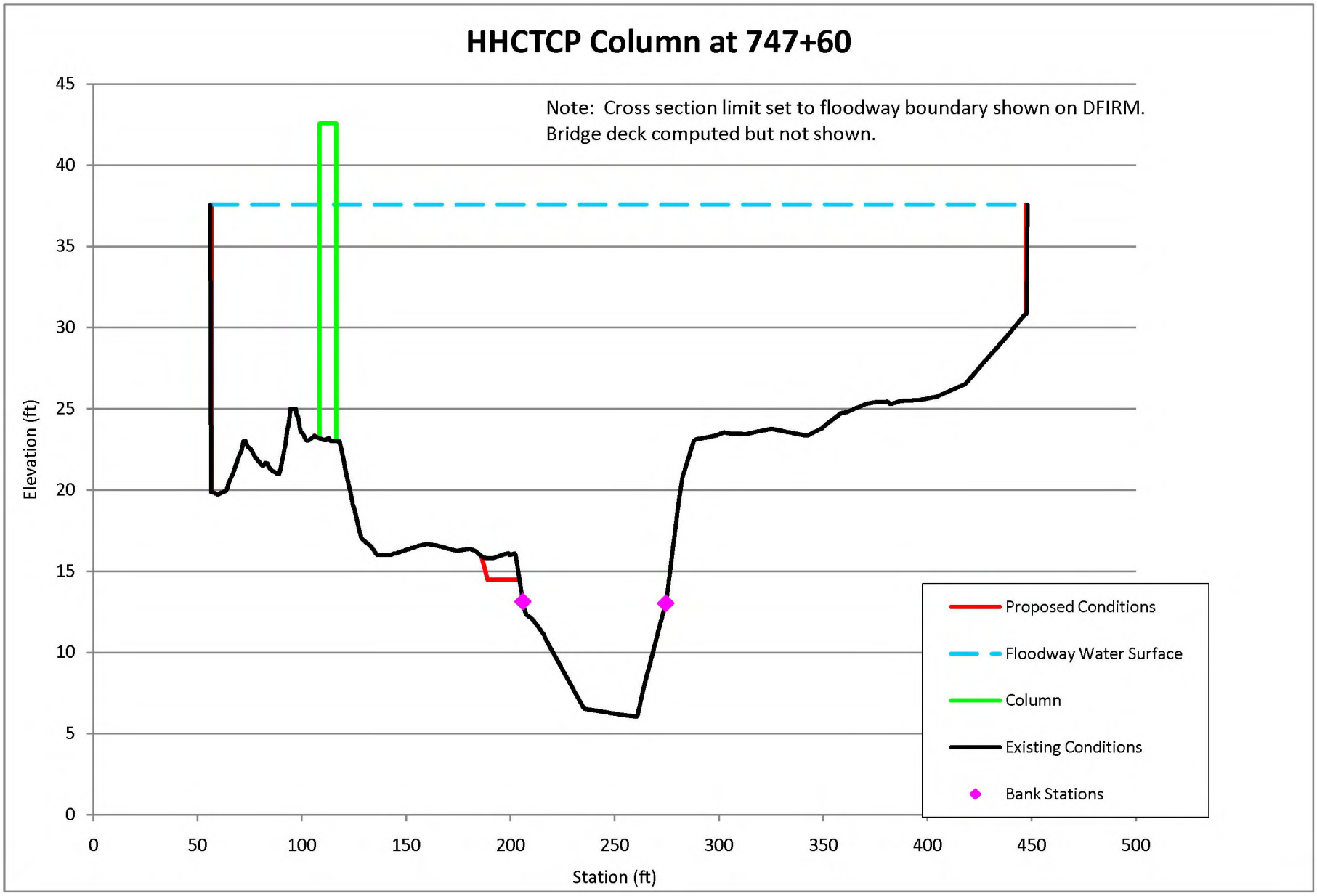
500
Proposed Conditions
Floodway Water Surface
Column
Existing Conditions
Bank Stations


HHCTCP Column at 747+60
Note: Cross section limit set to floodway boundary shown on DFIRM.
Bridge deck computed but not shown.
Elevation (ft)
Station (ft)
0
5
10
15
20
25
30
35
40
45
0
50
100
150
200
250
300
350
400
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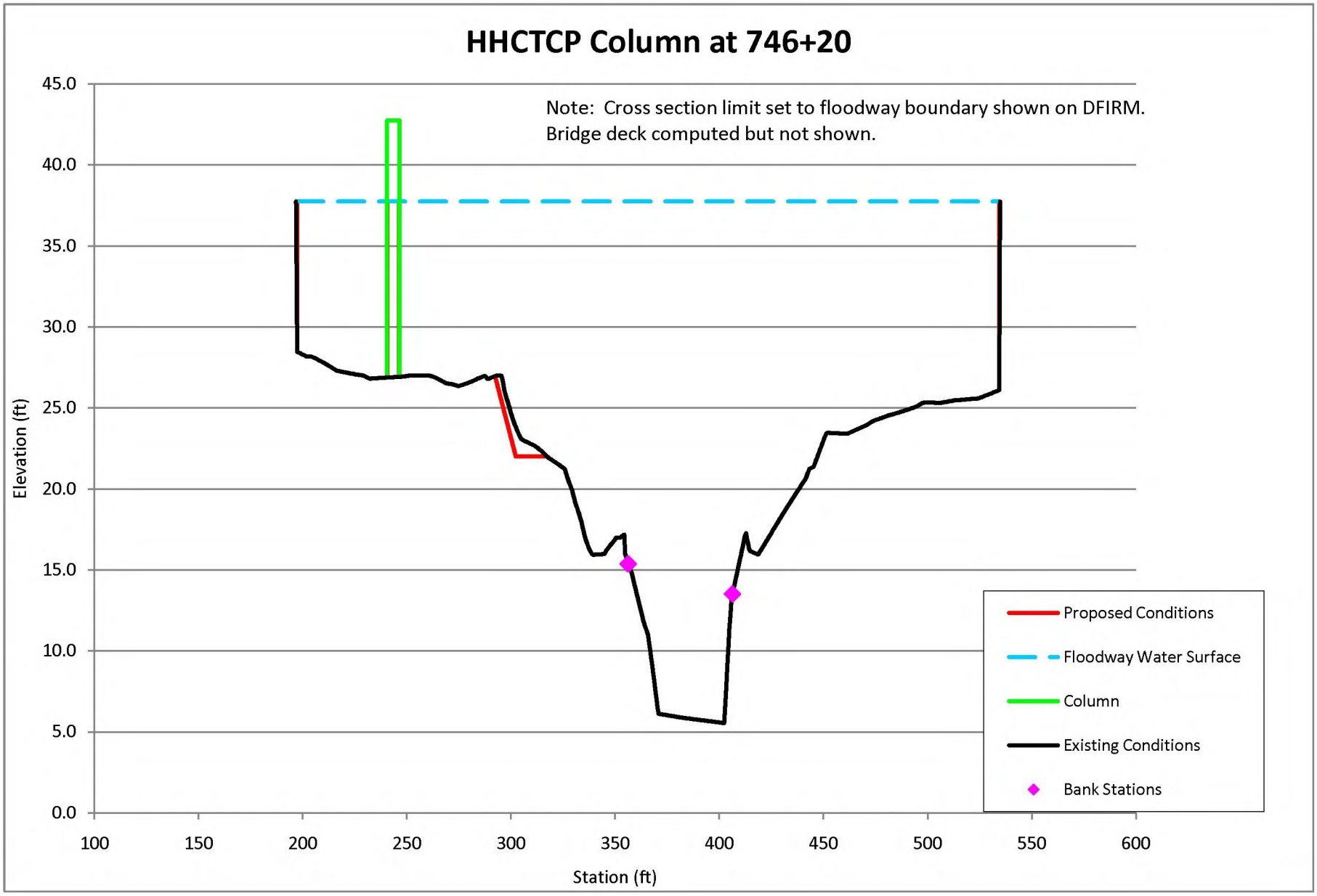
500
Proposed Conditions
Floodway Water Surface
Column
Existing Conditions
Bank Stations


HHCTCP Column at 746+20
Note: Cross section limit set to floodway boundary shown on DFIRM.
Bridge deck computed but not shown.
Elevation (ft)
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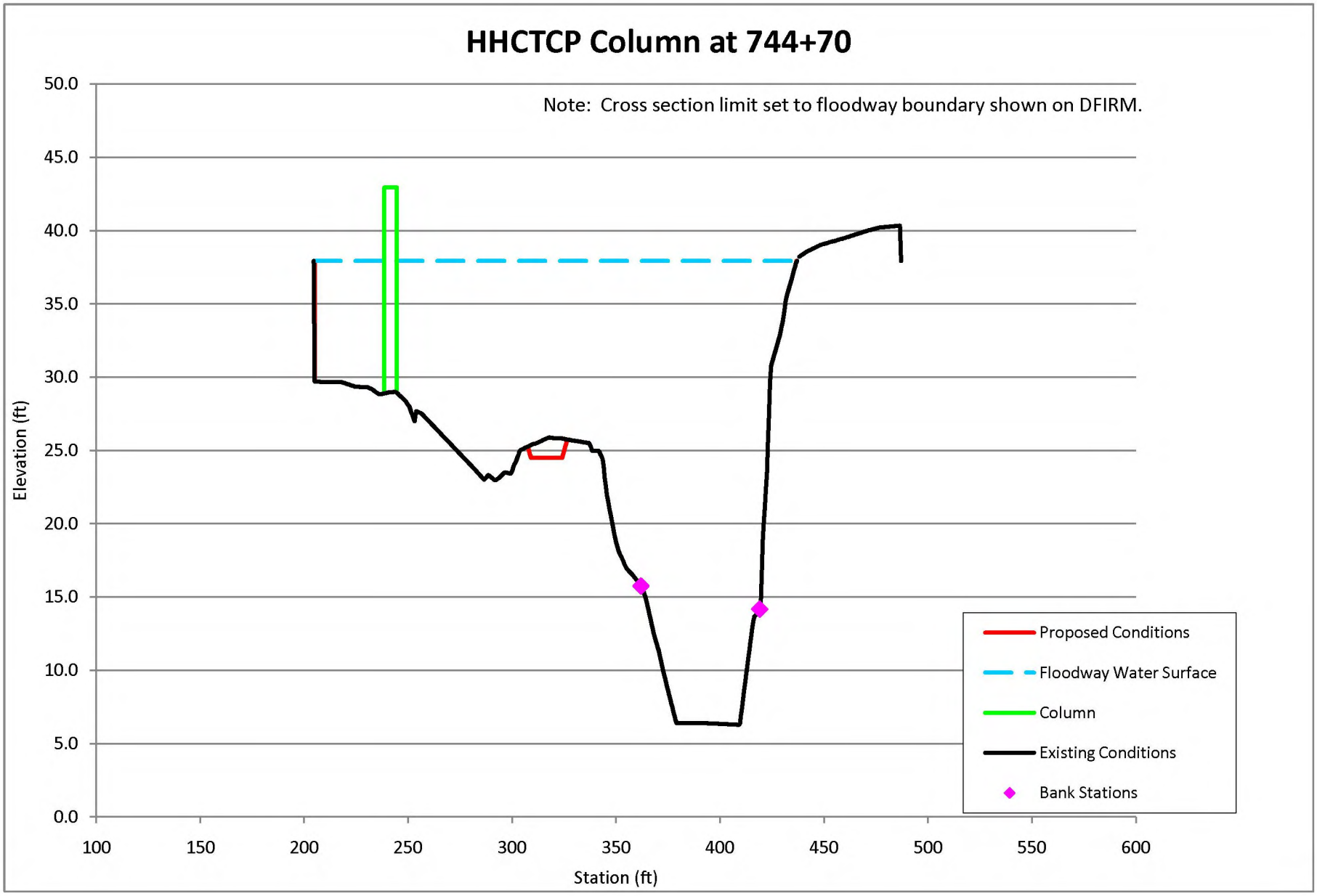
40.0
35.0
30.0
25.0
20.0
15.0
10.0
5.0
0.0
100
150
200
250
300
350
400
450
500
550
600
Station (ft)
Proposed Conditions
Floodway Water Surface
Column
Existing Conditions
Bank Stations


HHCTCP Column at 744+70
Note: Cross section limit set to floodway boundary shown on DFIRM.
Elevation (ft)
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45.0
40.0
35.0
30.0
25.0
20.0
15.0
10.0
5.0
0.0
100
150
200
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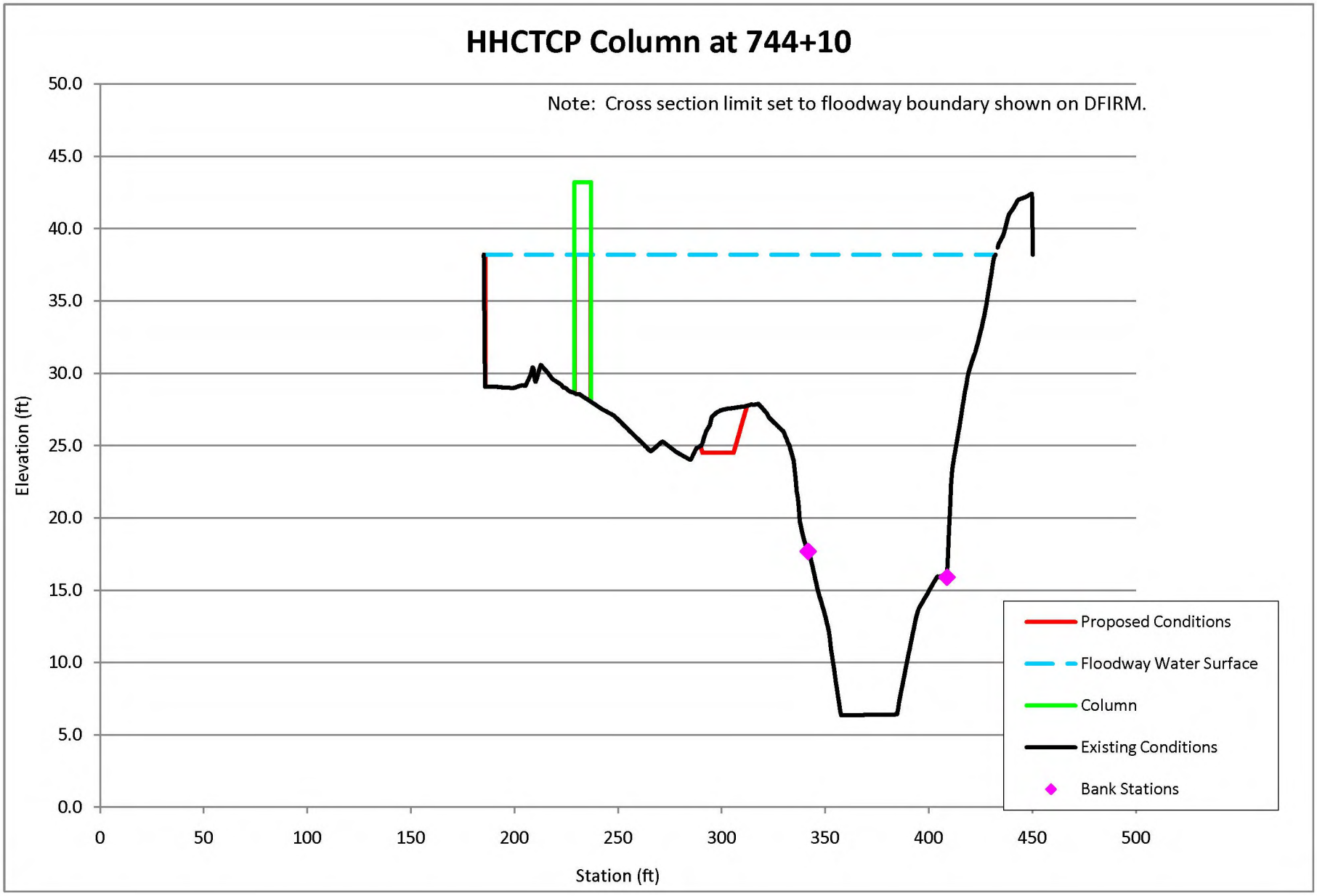
500
550
600
Station (ft)
Proposed Conditions
Floodway Water Surface
Column
Existing Conditions
Bank Stations


HHCTCP Column at 744+10
Note: Cross section limit set to floodway boundary shown on DFIRM.
Elevation (ft)
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45.0
40.0
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30.0
25.0
20.0
15.0
10.0
5.0
0.0
0
50
100
150
200
250
300
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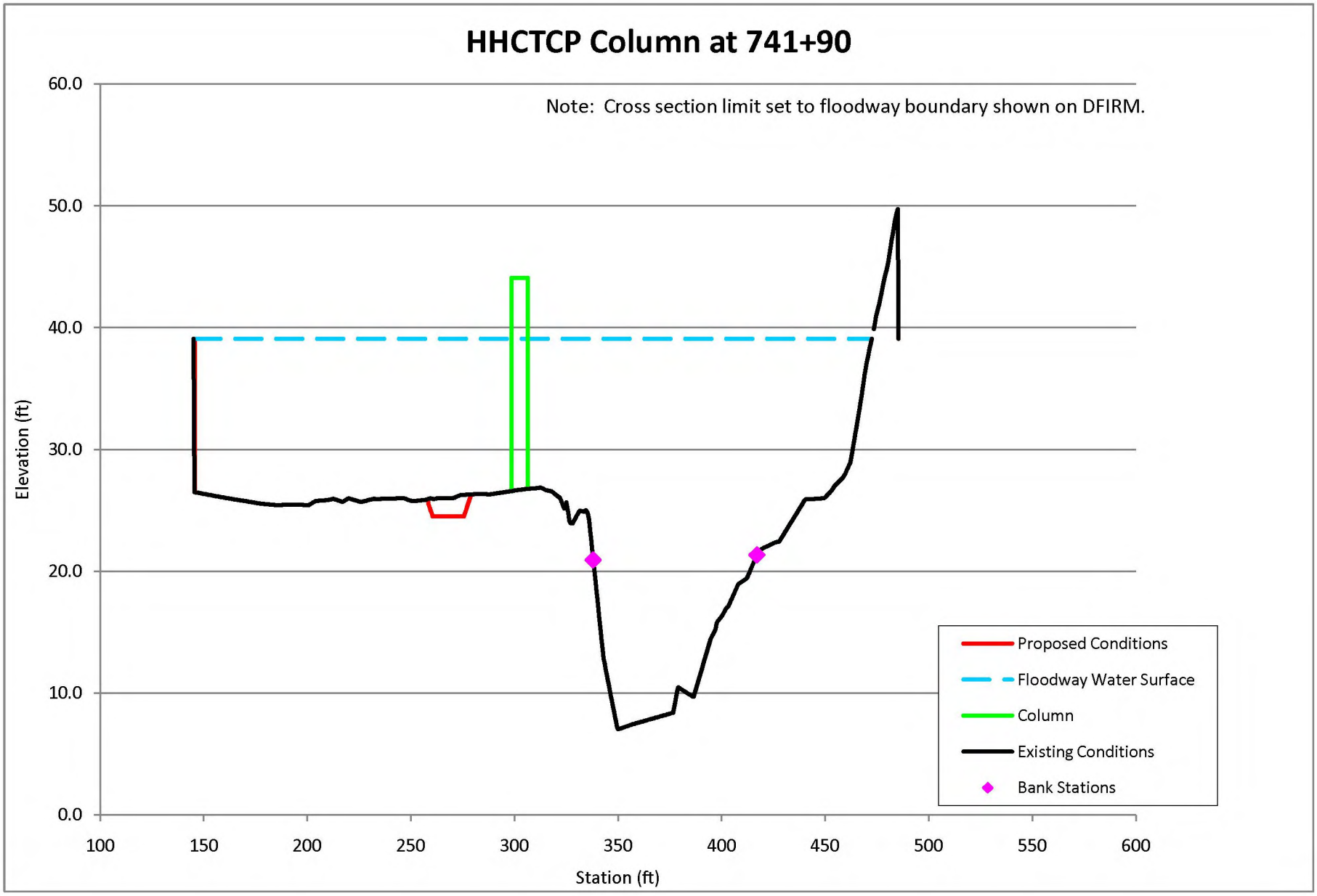
400
450
500
Station (ft)
Proposed Conditions
Floodway Water Surface
Column
Existing Conditions
Bank Stations


HHCTCP Column at 741+90
Note: Cross section limit set to floodway boundary shown on DFIRM.
Elevation (ft)
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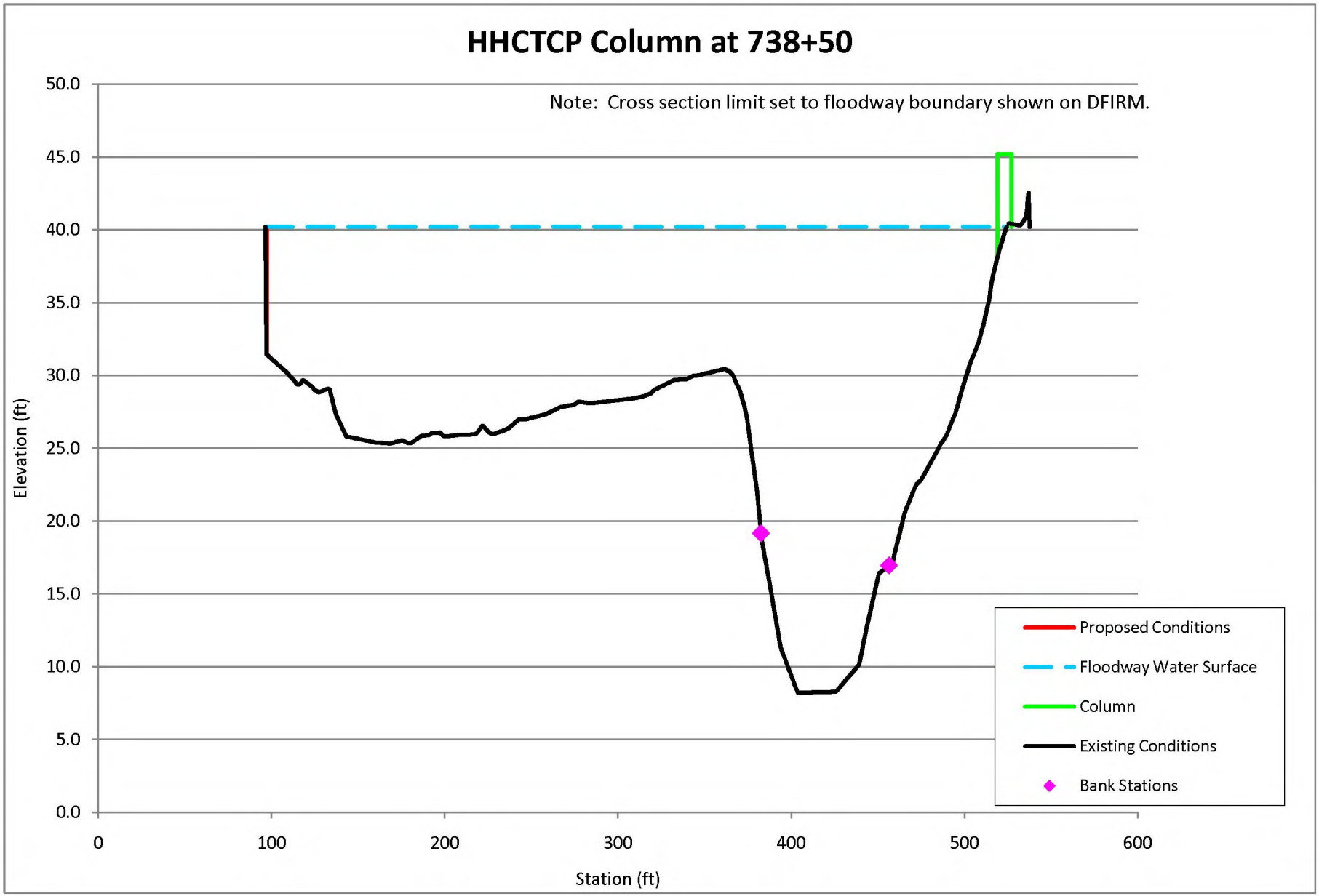
50.0
40.0
30.0
20.0
10.0
0.0
100
150
200
250
300
350
400
450
500
550
600
Station (ft)
Proposed Conditions
Floodway Water Surface
Column
Existing Conditions
Bank Stations


HHCTCP Column at 738+50
Note: Cross section limit set to floodway boundary shown on DFIRM.
Elevation (ft)
50.0
45.0
40.0
35.0
30.0
25.0
20.0
15.0
10.0
5.0
0.0
0
100
200
300
400
500
600
Station (ft)
Proposed Conditions
Floodway Water Surface
Column
Existing Conditions
Bank Stations

APPENDIX B

Waiawa Stream
Effective FIS HEC-2 Hydraulic Model Results
Floodway Run

Provided by FEMA Library
File: wai-enc.out

AR00041517

Source: Wai-enc.out

THIS RUN EXECUTED 23AUG99 09:37:34

****
HEC-2 WATER SURFACE PROFILES

Version 4.6.2; May 1991
****

NOTE- ASTERISK (*) AT LEFT OF CROSS-SECTION NUMBER INDICATES MESSAGE IN SUMMARY OF ERRORS LIST

WAIAWA STREAM: INEFFECT

SUMMARY PRINTOUT

profiles
100-yr Flooding Run

	SECNO	Q	CWSEL	DEPTH	TOPWID	SSTA	ENDST	STENCL	STENCR
	.000	18110.70	1.90	13.40	1000.99	.01	1000.99	.00	.00
	.000	19410.70	1.90	13.40	1000.99	.01	1000.99	.00	1001.00
*	500.000	18110.70	2.63	13.63	657.87	216.09	873.96	.00	.00
*	500.000	19410.70	2.75	13.75	400.00	470.00	870.00	470.00	870.00
	870.000	18110.70	5.36	16.88	462.98	1901.27	2364.25	1785.00	2560.00
A *	870.000	19410.70	6.33	17.85	400.00	1940.00	2340.00	1940.00	2340.00
	1270.000	18110.70	9.17	20.17	355.41	1738.94	2270.00	1716.00	2270.00
	1270.000	19410.70	9.68	20.68	270.00	1780.00	2050.00	1780.00	2050.00
	1590.000	16560.70	10.99	21.68	776.94	1230.00	2006.94	1230.00	2060.00
B	1590.000	19410.70	11.56	22.25	590.00	1400.00	1990.00	1400.00	1990.00
	1890.000	19410.70	12.30	22.50	1404.26	420.00	1867.49	420.00	1885.00
	1890.000	19410.70	12.98	23.18	780.00	1070.00	1850.00	1070.00	1850.00
	2190.000	19410.70	13.14	22.96	1347.85	433.00	1974.90	433.00	3113.00
	2190.000	19410.70	13.81	23.63	830.00	1110.00	1940.00	1110.00	1940.00
	2250.000	19410.70	13.23	27.19	1304.66	387.00	1899.41	387.00	2106.00
	2250.000	19410.70	13.90	27.86	820.00	1090.00	1910.00	1090.00	1910.00
*	2251.000	19410.70	13.50	27.46	1357.95	387.00	1913.57	387.00	2106.00
*	2251.000	19410.70	14.18	28.14	820.00	1090.00	1910.00	1090.00	1910.00
	2271.000	19410.70	13.62	27.58	1382.22	387.00	1920.03	387.00	2106.00
	2271.000	19410.70	14.34	28.30	820.00	1090.00	1910.00	1090.00	1910.00
C	2272.000	19410.70	13.52	13.52	1403.02	426.00	1941.73	426.00	3080.00
	2272.000	19410.70	14.16	14.16	800.00	1100.00	1900.00	1100.00	1900.00

AR00041518

	SECNO	Q	CWSEL	DEPTH	TOPWID	SSTA	ENDST	STENCL	STENCR
	2292.000	19410.70	13.62	13.62	1424.42	426.00	1946.63	426.00	3080.00
	2292.000	19410.70	14.30	14.30	800.00	1100.00	1900.00	1100.00	1900.00
	2293.000	19410.70	13.62	21.68	1383.20	400.00	1907.00	400.00	3040.00
	2293.000	19410.70	14.39	22.45	810.00	1060.00	1870.00	1060.00	1870.00
	2713.000	19410.70	14.52	26.25	1329.14	285.00	1633.67	285.00	2908.00
*	2713.000	19410.70	15.45	27.18	900.00	660.00	1560.00	660.00	1560.00
D	3043.000	19410.70	14.89	23.39	1192.53	306.00	1510.47	306.00	1640.00
	3043.000	19410.70	15.89	24.39	780.00	570.00	1350.00	570.00	1350.00
	3163.000	19410.70	15.05	22.55	1160.24	250.00	1445.03	250.00	1567.00
	3163.000	19410.70	16.07	23.57	760.00	515.00	1275.00	515.00	1275.00
*	3263.000	23034.70	15.17	21.81	1346.46	.00	1346.46	.00	1549.00
	3263.000	23034.70	16.17	22.81	830.00	355.00	1185.00	355.00	1185.00
	3335.000	23034.70	15.29	26.32	1366.32	.00	1366.32	.00	1550.00
	3335.000	23034.70	16.31	27.34	920.00	310.00	1230.00	310.00	1230.00
	3343.000	23034.70	15.30	26.33	1366.42	.00	1366.42	.00	1550.00
	3343.000	23034.70	16.32	27.35	920.00	310.00	1230.00	310.00	1230.00
E	3415.000	23034.70	15.36	22.89	1514.28	.00	1523.06	.00	1590.00
	3415.000	23034.70	16.39	23.92	920.00	370.00	1290.00	370.00	1290.00
	4163.000	23034.70	15.61	24.75	1562.89	124.18	1687.08	.00	.00
	4163.000	23034.70	16.63	25.77	1280.00	300.00	1580.00	300.00	1580.00
*	4243.000	23034.70	15.74	23.18	1577.74	63.01	1640.76	.00	.00
	4243.000	23034.70	16.73	24.17	1320.00	220.00	1540.00	220.00	1540.00
	4257.000	23034.70	15.76	23.20	1579.73	61.68	1641.41	.00	.00
	4257.000	23034.70	16.75	24.19	1340.00	220.00	1560.00	220.00	1560.00
F *	4293.000	23034.70	15.84	22.63	1572.46	251.74	1824.20	.00	.00
*	4293.000	23034.70	16.81	23.60	1350.00	360.00	1710.00	360.00	1710.00
G	5373.000	34000.00	16.30	16.30	1873.95	224.30	2098.25	.00	.00
	5373.000	34000.00	17.18	17.18	1230.00	340.00	1570.00	340.00	1570.00
H	5729.000	34000.00	16.65	18.63	1789.65	180.58	1986.77	.00	.00
	5729.000	34000.00	17.52	19.50	1369.64	320.00	1690.00	320.00	1690.00
I *	6689.000	34000.00	20.63	22.97	838.47	200.95	1039.42	.00	.00
*	6689.000	34000.00	20.58	22.92	640.00	300.00	940.00	300.00	940.00
*	7005.000	34000.00	24.03	25.11	1028.71	7.97	1036.68	.00	.00
*	7005.000	34000.00	24.35	25.43	590.00	390.00	980.00	390.00	980.00

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AR00041519

	SECNO	Q	CWSEL	DEPTH	TOPWID	SSTA	ENDST	STENCL	STENCR
*	7205.000	34000.00	24.13	25.28	829.19	195.60	1024.79	.00	.00
*	7205.000	34000.00	24.42	25.57	400.00	590.00	990.00	590.00	990.00
J	7581.000	34000.00	26.92	25.31	680.66	293.94	974.60	.00	.00
	7581.000	34000.00	27.50	25.89	260.00	690.00	950.00	690.00	950.00
	8053.000	34000.00	33.01	33.01	620.01	235.69	855.70	.00	.00
	8053.000	34000.00	34.00	34.00	480.00	290.00	770.00	290.00	770.00
	Farrington Bridge								
	8089.000	34000.00	33.44	33.44	654.27	347.17	1001.44	.00	.00
	8089.000	34000.00	34.48	34.48	480.00	395.00	875.00	395.00	875.00
	8509.000	34000.00	34.87	30.43	496.08	26.27	522.34	.00	.00
	8509.000	34000.00	35.72	31.28	325.00	80.00	405.00	80.00	405.00
	8765.000	34000.00	36.51	33.27	706.02	24.11	730.13	.00	.00
	8765.000	34000.00	37.37	34.13	530.00	70.00	600.00	70.00	600.00
	Farrington Bridge								
	8785.000	34000.00	36.58	33.34	707.37	23.98	731.34	.00	.00
	8785.000	34000.00	37.46	34.22	530.00	70.00	600.00	70.00	600.00
	8981.000	34000.00	36.92	30.78	554.18	20.93	608.14	.00	.00
K	8981.000	34000.00	37.87	31.73	330.00	115.00	445.00	115.00	445.00
	10461.000	34000.00	42.01	33.56	467.46	25.19	492.64	.00	.00
L	10461.000	34000.00	42.70	34.25	290.00	100.00	390.00	100.00	390.00
	10721.000	34000.00	43.04	32.87	805.61	33.91	839.52	.00	.00
*	10721.000	34000.00	43.95	33.78	610.00	150.00	760.00	150.00	760.00
	Kam Hwy Bridge								
	10741.000	34000.00	43.07	32.90	805.81	33.85	839.65	.00	.00
	10741.000	34000.00	43.98	33.81	610.00	150.00	760.00	150.00	760.00
	10765.000	34000.00	43.11	33.58	925.84	55.34	981.18	.00	.00
	10765.000	34000.00	44.01	34.48	610.00	170.00	780.00	170.00	780.00
	Kam Hwy Bridge								
	10785.000	34000.00	43.16	33.63	926.37	55.28	981.65	.00	.00
	10785.000	34000.00	44.06	34.53	610.00	170.00	780.00	170.00	780.00
*	10865.000	34000.00	43.25	32.52	785.34	50.80	836.14	.00	.00
*	10865.000	34000.00	44.20	33.47	580.00	130.00	710.00	130.00	710.00
	11045.000	34000.00	43.41	31.44	793.37	57.17	850.54	.00	.00
	11045.000	34000.00	44.36	32.39	620.00	140.00	760.00	140.00	760.00
M	11145.000	34000.00	43.49	30.19	833.31	40.16	873.48	.00	.00
	11145.000	34000.00	44.43	31.13	670.00	70.00	740.00	70.00	740.00
	11785.000	23000.00	43.96	36.99	1155.15	47.34	1202.49	.00	.00
	11785.000	23000.00	44.91	37.94	800.00	170.00	970.00	170.00	970.00

AR00041520

	SECNO	Q	CWSEL	DEPTH	TOPWID	SSTA	ENDST	STENCL	STENCR
N*	11995.000	23000.00	44.02	32.02	951.82	68.35	1020.17	.00	.00
*	11995.000	23000.00	44.97	32.97	620.00	180.00	800.00	180.00	800.00
	12195.000	23000.00	44.17	28.17	806.73	71.12	877.86	.00	.00
	12195.000	23000.00	45.11	29.11	510.00	190.00	700.00	190.00	700.00
	12355.000	23000.00	44.29	27.29	770.69	90.80	861.49	.00	.00
	12355.000	23000.00	45.22	28.22	400.00	320.00	720.00	320.00	720.00
O	12515.000	23000.00	44.48	26.10	478.52	365.37	843.89	.00	.00
	12515.000	23000.00	45.37	26.99	300.00	400.00	700.00	400.00	700.00
	12615.000	23000.00	44.70	25.70	740.56	183.84	924.39	.00	.00
	12615.000	23000.00	45.59	26.59	340.00	410.00	750.00	410.00	750.00
	12675.000	23000.00	44.99	25.49	777.72	155.99	933.71	.00	.00
	12675.000	23000.00	45.89	26.39	370.00	390.00	760.00	390.00	760.00
	12705.000	23000.00	45.17	25.42	765.56	161.57	927.12	.00	.00
	12705.000	23000.00	46.08	26.33	380.00	320.00	700.00	320.00	700.00
	12709.000	23000.00	45.21	21.21	797.44	203.36	1000.80	.00	.00
	12709.000	23000.00	46.10	22.10	380.00	380.00	760.00	380.00	760.00
	12729.000	23000.00	45.21	21.41	755.75	265.73	1021.48	.00	.00
	12729.000	23000.00	46.12	22.32	400.00	390.00	790.00	390.00	790.00
	12779.000	23000.00	45.17	23.28	709.32	274.37	983.69	.00	.00
*	12779.000	23000.00	46.07	24.18	410.00	380.00	790.00	380.00	790.00
	12873.000	23000.00	45.38	23.43	691.21	198.85	890.06	.00	.00
	12873.000	23000.00	46.31	24.36	510.00	220.00	730.00	220.00	730.00
P	13283.000	23000.00	46.23	23.73	721.59	160.33	924.52	.00	.00
	13283.000	23000.00	47.17	24.67	491.08	270.00	800.00	270.00	800.00
	13403.000	23000.00	46.44	23.44	796.62	129.53	985.77	.00	.00
	13403.000	23000.00	47.36	24.36	473.23	300.00	830.00	300.00	830.00
	13493.000	23000.00	46.64	23.42	832.60	152.12	1069.85	.00	.00
	13493.000	23000.00	47.53	24.31	565.22	240.00	880.00	240.00	880.00
Q	13813.000	23000.00	48.58	23.58	722.40	100.56	918.95	.00	.00
	13813.000	23000.00	49.55	24.55	548.58	180.00	820.00	180.00	820.00
	14093.000	23000.00	49.97	22.91	494.30	379.49	873.80	.00	.00
	14093.000	23000.00	50.72	23.66	480.00	390.00	870.00	390.00	870.00
*	14533.000	23000.00	53.35	23.74	454.19	720.00	1174.19	720.00	1259.50
*	14533.000	23000.00	53.32	23.71	450.00	720.00	1170.00	720.00	1170.00

1

AR00041521

	SECNO	Q	CWSEL	DEPTH	TOPWID	SSTA	ENDST	STENCL	STENCR
*	14823.000	23000.00	57.32	25.32	553.02	450.00	1003.02	450.00	1055.00
*	14823.000	23000.00	57.35	25.35	500.00	500.00	1000.00	500.00	1000.00
	14973.000	23000.00	57.84	23.84	499.60	500.00	999.60	500.00	1005.00
	14973.000	23000.00	57.99	23.99	460.00	500.00	960.00	500.00	960.00
	15053.000	23000.00	57.70	22.70	444.25	530.00	974.25	530.00	980.00
	15053.000	23000.00	57.86	22.86	420.00	530.00	950.00	530.00	950.00
	15203.000	23000.00	59.42	21.50	222.73	92.58	440.93	.00	.00
	15203.000	23000.00	59.49	21.57	223.94	92.08	441.27	92.00	459.00
*	15299.000	23000.00	62.67	25.03	331.88	225.33	557.20	.00	.00
*	15299.000	23000.00	62.68	25.04	331.00	226.00	557.00	226.00	557.00
*	15399.000	23000.00	62.78	19.94	319.99	166.56	486.55	.00	.00
*	15399.000	23000.00	62.79	19.95	319.00	167.00	486.00	167.00	486.00

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AR00041522

APPENDIX C

Conveyance and Volume Calculations

AR00041523

Exhibit C1 - Conveyance Hand Calculation Methodology

HHCTCP
Proposed Guideway Columns - Waiawa Stream
No-Rise Conveyance Compensation Calculations
by Pat Flanagan, nhc
April 2010

Conveyance Calculations based on Example provided by FEMA.

EXAMPLE of calculations at Cross-section (XS) 741+90.

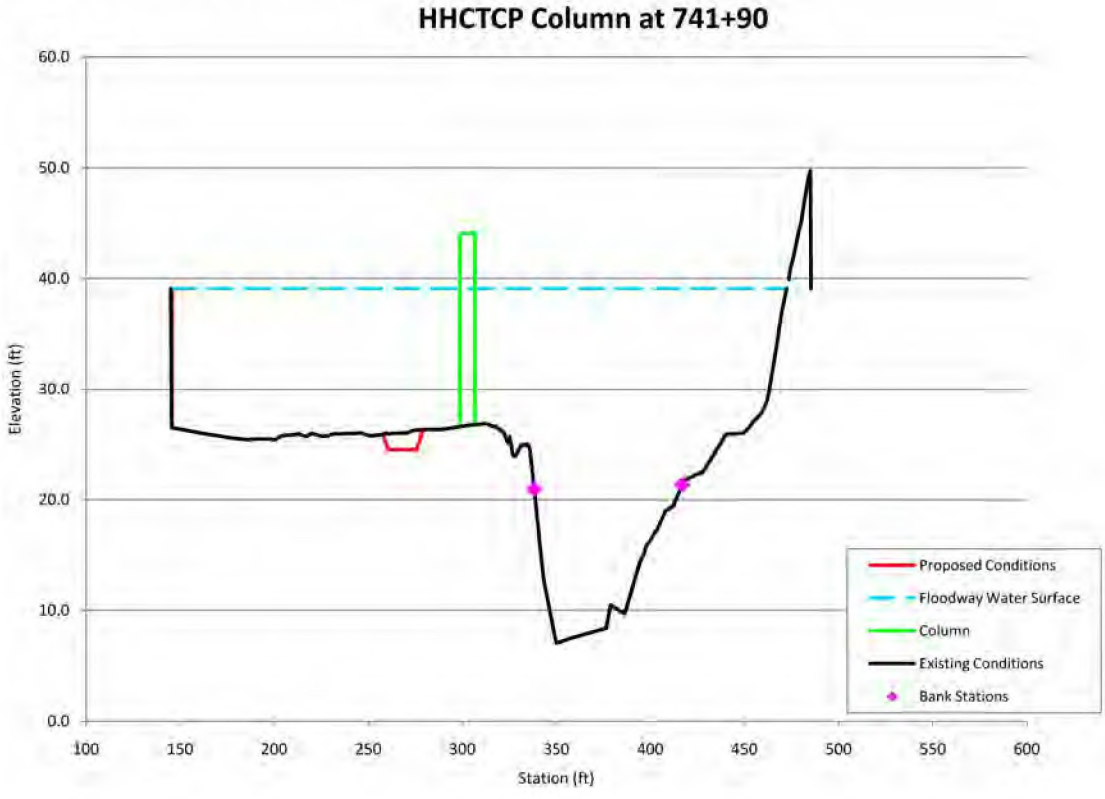


Figure A. Existing Cross-section geometry, overlayed with proposed column and left bank excavation. Effective FEMA Floodway boundary is shown as the limits of the XS lines.

Assumptions:
1) Conveyance for the cross-section will be calculated for the floodway run. Floodway elevation obtained from the effective FIS model output provided by FEMA (HEC-2) and interpolated to XS.
2) The cross-section will be divided into three sections to calculate conveyance: Left overbank, channel, and right overbank.
3) The floodway encroachment and wetted portions of the columns will be included in the calculation of wetted perimeter and area.
4) Manning's n is weighted based on wetted perimeter to the 2/3 power. (Chow "Open Channel Hydraulics")

Equation for Conveyance (K) :

$$K = \frac{1.486}{n} \cdot A \cdot R^{\frac{2}{3}}$$

where: K = Conveyance
n = manning's n
A = Area (ft^2)
R = A/P (ft)
P = Wetted Perimeter (ft)

Existing Conveyance:

(Note: LB= left overbank, CH= channel, RB= right overbank)

$$n\_units := \frac{sec}{ft^{\frac{1}{3}}}$$

$$cfs := \frac{ft^3}{sec}$$

Area:

Calculated as the area above the ground surface but below the floodway water surface.

$A_{LB} := 2536.5866ft^2$ $A_{CH} := 2131.0463 \cdot ft^2$ $A_{RB} := 691.1122 \cdot ft^2$

Wetted Perimeter:

Calculated as the distance of wetted surface, includes ground surface, columns, floodway edge, bridges, etc.

$P_{LB} := 208.2886 \cdot ft$ $P_{CH} := 88.9982 \cdot ft$ $P_{RB} := 60.5357ft$

Weighted Manning's n:

Manning's n has been assigned to surfaces using the effective FIS model values. The XS was tabulated using station/elevation data spaced every 0.5-ft. The weighted n for each section of the XS (overbanks, channel) was calculated using the equation:

$$n_{weighted} = \left[\frac{\sum_i \left(P_i \cdot n_i^{1.5} \right)}{\sum_i P_i} \right]^{\frac{2}{3}}$$

$n_{LB} := 0.10000 \cdot n\_units$ $n_{CH} := 0.080000 \cdot n\_units$ $n_{RB} := 0.10000 \cdot n\_units$

AR00041525

$$K_{LB} := \frac{1.486}{n_{LB}} \cdot A_{LB} \cdot \left(\frac{A_{LB}}{P_{LB}}\right)^{\frac{2}{3}} \qquad K_{LBpre} := K_{LB}$$

$$K_{CH} := \frac{1.486}{n_{CH}} \cdot A_{CH} \cdot \left(\frac{A_{CH}}{P_{CH}}\right)^{\frac{2}{3}} \qquad K_{CHpre} := K_{CH}$$

$$K_{RB} := \frac{1.486}{n_{RB}} \cdot A_{RB} \cdot \left(\frac{A_{RB}}{P_{RB}}\right)^{\frac{2}{3}} \qquad K_{RBpre} := K_{RB}$$

$$K_{pre} := K_{LB} + K_{CH} + K_{RB}$$

Conveyance for the existing cross-section geometry:

$K_{LBpre} = 199522.2\,cfs$ $K_{CHpre} = 328848.7\,cfs$ $K_{RBpre} = 52070.5\,cfs$

$K_{pre} = 580441\,cfs$ Total Conveyance for the XS

The proposed column is then added to the XS and conveyance is calculated.

Existing Channel with Proposed Column Conveyance:

Updated Area and Perimeter in wetted cross-section:

Area:

$A_{LB} := 2437.2387 \cdot ft^2$ $A_{CH} := 2131.0463 \cdot ft^2$ $A_{RB} := 691.1122 \cdot ft^2$

Wetted Perimeter:

$P_{LB} := 225.0935 \cdot ft$ $P_{CH} := 88.9982 \cdot ft$ $P_{RB} := 60.5357 ft$

Weighted Manning's n: (updated with n=.013 for columns)

$n_{LB} := 0.0928627 \cdot n\_units$ $n_{CH} := 0.0800 \cdot n\_units$ $n_{RB} := 0.10000 \cdot n\_units$

$$K_{LB} := \frac{1.486}{n_{LB}} \cdot A_{LB} \cdot \left(\frac{A_{LB}}{P_{LB}}\right)^{\frac{2}{3}} \quad K_{CH} := \frac{1.486}{n_{CH}} \cdot A_{CH} \cdot \left(\frac{A_{CH}}{P_{CH}}\right)^{\frac{2}{3}} \quad K_{RB} := \frac{1.486}{n_{RB}} \cdot A_{RB} \cdot \left(\frac{A_{RB}}{P_{RB}}\right)^{\frac{2}{3}}$$

$K_{LBcol} := K_{LB}$ $K_{CHcol} := K_{CH}$ $K_{RBcol} := K_{RB}$ $K_{col} := K_{LB} + K_{CH} + K_{RB}$

$K_{LBcol} = 190882.3\,cfs$ $K_{CHcol} = 328848.7\,cfs$ $K_{RBcol} = 52070.5\,cfs$

$dK_{LB} := K_{LBcol} - K_{LBpre}$ $dK_{CH} := K_{CHcol} - K_{CHpre}$ $dK_{RB} := K_{RBcol} - K_{RBpre}$

Change in Conveyance due to the column:

$\boxed{dK_{LB} = -8639.9\,cfs}$ $\boxed{dK_{CH} = 0\,cfs}$ $\boxed{dK_{RB} = 0\,cfs}$

$dK_{col} := K_{col} - K_{pre}$ $\boxed{dK_{col} = -8640\,cfs}$

These calculations show that the column reduces total conveyance, therefore compensation for this lost conveyance must be made to keep existing floodway elevations unchanged.

The compensation features are then added to the XS to offset the lost conveyance due to the column.

Proposed Mitigated Geometry:
Excavate swale (or bench) on left bank, typically near the column (see Figure A) to compensate for lost conveyance.

Area and Wetted Perimeter of proposed XS, including column:

$A_{LB} := 2465.7842 \cdot ft^2$ $A_{CH} := 2131.0463 \cdot ft^2$ $A_{RB} := 691.1122 \cdot ft^2$

$P_{LB} := 225.9122 \cdot ft$ $P_{CH} := 88.9982 ft$ $P_{RB} := 60.5357 \cdot ft$

Weighted Manning's n: (updated with n=.013 for column and n=.07 for grass-lined excavation)

$n_{LB} := 0.0901823 \cdot n\_units$ $n_{CH} := 0.080 \cdot n\_units$ $n_{RB} := 0.1000 \cdot n\_units$

$$K_{LB} := \frac{1.486}{n_{LB}} \cdot A_{LB} \cdot \left(\frac{A_{LB}}{P_{LB}}\right)^{\frac{2}{3}} \quad K_{CH} := \frac{1.486}{n_{CH}} \cdot A_{CH} \cdot \left(\frac{A_{CH}}{P_{CH}}\right)^{\frac{2}{3}} \quad K_{RB} := \frac{1.486}{n_{RB}} \cdot A_{RB} \cdot \left(\frac{A_{RB}}{P_{RB}}\right)^{\frac{2}{3}}$$

$K_{LBprop} := K_{LB}$ $K_{CHprop} := K_{CH}$ $K_{RBprop} := K_{RB}$ $K_{prop} := K_{LB} + K_{CH} + K_{RB}$

$K_{LBprop} = 199923\,cfs$ $K_{CHprop} = 328848.7\,cfs$ $K_{RBprop} = 52070.5\,cfs$

$dK_{LB} := K_{LBprop} - K_{LBpre}$ $dK_{CH} := K_{CHprop} - K_{CHpre}$ $dK_{RB} := K_{RBprop} - K_{RBpre}$

Change in Conveyance due to the column and mitigation:

$\boxed{dK_{LB} = 401\,cfs}$ $\boxed{dK_{CH} = 0\,cfs}$ $\boxed{dK_{RB} = 0\,cfs}$

$dK_{prop} := K_{prop} - K_{pre}$ $\boxed{dK_{prop} = 401\,cfs}$

These calculations show that the proposed modifications to excavate a swale on the left bank will provide adequate conveyance compensation for the column for the FEMA 100-year floodway.

Table Appendix C. Detailed Volume Calculations for the Proposed Guideway Columns and Compensation Measures in Waiawa Stream Floodway

HHCTCP Column Station		Existing Conditions	With Proposed Guideway Columns				With Proposed Columns and Compensation Measures			
		Area	Column Width	Area	Difference from Existing Conditions Area	Wetted Volume in Floodway Blocked by Column	Approximate Length of Proposed Excavation	Area	Increase in XS Area Due to Mitigation	Approximate Volume Gained by Mitigation
		sq. ft	ft	sq. ft	sq. ft	cu. yds	ft	sq. ft	sq. ft	cu. yds
749+40	Left Overbank	1298.3	8	1207.8	-90.5	-26.8	52	1231.5	23.8	45.8
	Channel	1961.7		1961.7	0.0	0	0	1961.7	0.0	0
	Right Overbank	2722.6		2722.6	0.0	0	0	2722.6	0.0	0
748+50	Left Overbank	2568.2	8	2384.7	-183.5	-54.4	48	2418.2	33.5	59.5
	Channel	1678.5		1678.5	0.0	0	0	1678.5	0.0	0
	Right Overbank	2858.9		2858.9	0.0	0	0	2858.9	0.0	0
747+60	Left Overbank	2779.3	8	2663.3	-116.0	-34.4	126	2686.2	22.9	106.8
	Channel	1539.1		1539.1	0.0	0	0	1539.1	0.0	0
	Right Overbank	2165.0		2165.0	0.0	0	0	2165.0	0.0	0
746+20	Left Overbank	2118.1	6	2058.3	-59.8	-13.3	45	2082.6	24.3	40.1
	Channel	1496.5		1496.5	0.0	0	0	1496.5	0.0	0
	Right Overbank	1654.2		1654.2	0.0	0	0	1654.2	0.0	0
744+70	Left Overbank	1940.7	6	1886.8	-54.0	-12.0	138	1907.2	20.5	104.7
	Channel	1672.1		1672.1	0.0	0	0	1672.1	0.0	0
	Right Overbank	131.8		131.8	0.0	0	0	131.8	0.0	0
744+10	Left Overbank	1756.2	8	1682.3	-73.8	-21.9	141	1730.4	48.1	251.3
	Channel	1869.0		1869.0	0.0	0	0	1869.0	0.0	0
	Right Overbank	195.7		195.7	0.0	0	0	195.7	0.0	0
741+90	Left Overbank	2536.6	8	2437.2	-99.3	-29.4	225	2465.8	28.5	237.9
	Channel	2131.0		2131.0	0.0	0	0	2131.0	0.0	0
	Right Overbank	691.1		691.1	0.0	0	0	691.1	0.0	0
738+50	Left Overbank	3585.1	8	3585.1	0.0	0	0	3585.1	0.0	0
	Channel	2149.4		2149.4	0.0	0	0	2149.4	0.0	0
	Right Overbank	866.3		861.1	-5.2	-1.5	0	861.1	0.0	0
			Sum of Volume Lost due to Columns =			**-194**	Total Volume Excavated from Floodway =			**846**

Note: Differences in area are calculated such that positive values are an increase in area over existing conditions, while negative values are a loss in area from existing conditions.

APPENDIX D

FEMA Certification of a No-Rise Determination

AR00041530

Federal Emergency Management Agency

Washington, D.C. 20472

CERTIFICATION OF A "NO-RISE" DETERMINATION

FOR A PROPOSED FLOODWAY DEVELOPMENT

City and County of Honolulu, Hawaii
Community Name

HHCTCP Guideway and Support Columns in Waiawa Stream
Development Name

Lot/Property Designation

Property Owner

I hereby certify that the proposed remedial measures, in combination with the property development designated above, will result in no loss of flow conveyance during the occurrence of the 1 percent annual chance of exceedence (100-year flood) discharge.

I further certify that the data submitted herewith in support of this request are accurate to the best of my knowledge, that the analyses have been performed correctly and in accordance with sound engineering practice, and that the proposed structural works are designed in accordance with sound engineering practice.

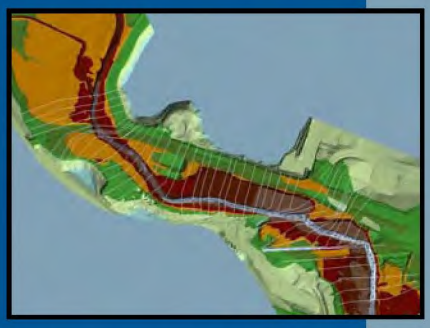
April 21, 2010
Date

Robert C. MacArthur
Registered Professional Engineer

Seal

NO-RISE ANALYSIS FOR PEARL HIGHLANDS STATION AND PARK-AND-RIDE FACILITY – WAIAWA STREAM

DRAFT



Prepared for:

Parsons Brinckerhoff Inc.
1001 Bishop Street, Suite 2400
Honolulu, HI 96813
808-531-7094

Prepared by:

Northwest Hydraulic Consultants Inc.
16300 Christensen Road, Suite 350
Seattle, WA 98188
206-241-6000

April 30, 2010

EXECUTIVE SUMMARY

The proposed Honolulu High-Capacity Transit Corridor Project (HHCTCP) is an elevated rail line that will provide rapid public transportation service in the City and County of Honolulu on the island of Oahu, Hawaii. Northwest Hydraulic Consultants (NHC) was retained to perform hydraulic evaluations with respect to and as required for planning and design of the proposed Pearl Highlands Station and Park-and-Ride Facility (PHSPF), part of the greater HHCTCP.

Portions of the proposed PHSPF are located within the floodway of Waiawa Stream. The City and County of Honolulu Department of Planning and Permitting (DPP) is the lead agency responsible for Flood Hazard District compliance within the City and County of Honolulu. As mandated by FEMA, DPP requires that all segments of the HHCTCP comply with a "No-Rise" policy, i.e. have no impact on 100-year flood water levels within floodway zones.

After inspecting the project site, and evaluating the hydrology of Waiawa Stream, NHC worked closely with the HHCTCP design team to develop a design that will not only meet the DPP / FEMA "No-Rise" requirement, but will minimize scour and erosion risks to both the facility and the stream. To properly consider the hydraulic issues at the site, NHC created two different hydraulic models of Waiawa stream. The first model utilized the U.S. Army Corps of Engineers HEC-RAS software program to create a one-dimensional model of the stream and project site. This model was used to provide a preliminary assessment of project impacts and to evaluate potential mitigation measures to offset those impacts. It was also used to demonstrate that the final project configuration will meet the DPP / FEMA "No-Rise" requirement. The second model was created using a software code referred to as FESWMS, which is a two-dimensional hydraulic model that was developed for the Federal Highway Administration. This model provides much more detailed hydraulic information than the HEC-RAS model and was used not only to refine the PHSPF and mitigation design to minimize impacts, but also to provide the design team with an understanding of how flow will move through and interact with the project features. This model will be used in future project phases to develop detailed designs for scour and erosion protection.

This report herein is explicitly written to provide DPP with the information that staff will need to confirm that the proposed project meets their own and FEMA's "No-Rise" requirements. FEMA provides guidelines describing the tasks that need to be completed to demonstrate "No-Rise" (these tasks are presented in Chapter 6 of this report). The guidelines state that the effective FEMA hydraulic model is the primary tool that should be used to demonstrate "No-Rise", which for this site is the Army Corps of Engineers' backwater programs HEC-RAS and its predecessor HEC-2. NHC followed the FEMA guidelines and modified the effective model to create an HEC-RAS model that contains sufficient detail to evaluate the final project configuration. The model results demonstrate that the project will not increase 100-year floodway water levels and therefore satisfies the "No-Rise" requirement.

DPP may decide that this No-rise analysis requires FEMA's review through the CLOMR/LOMR process due to the revised modeling, updated site topography, and numerous modifications proposed inside the effective floodway. This analysis and report herein would serve as the supporting documentation for any CLOMR or LOMR submittal.

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The two-dimensional FESWMS model that was used to assist with detailed design of the project was not used to confirm the "No-Rise" condition. A separate design report that contains results from the two-dimensional modeling analyses will be submitted along with this report to RTD. The two-dimensional modeling report is titled "Advanced Floodplain Evaluations for Pearl Highlands Station and Park-and-Ride Facility – Waiawa Stream" (April 30, 2010). It contains much of the same information included here.

NHC, working with the HHCTCP design team, together have developed a design for this complex facility that successfully mitigates impacts the project will have on 100-year flood levels. This document herein provides the data and results required to demonstrate that the DPP / FEMA "No-Rise" requirement has been satisfied.

The No-Rise analysis described above is for the entire project, which includes the elevated guideway as well as the Station / Park-and-Ride Facility. The current plan is to construct the guideway portion of the project as part of the West Oahu / Farrington Design-Build (WOF D-B) Contract, prior to the Station and Park-and-Ride Facility. The No-Rise analysis for the WOF D-B Contract only is presented in a separate report titled "No-Rise Analysis for HHCTCP Guideway Columns – Waiawa Stream" (April 30, 2010).

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No-Rise Analysis for Pearl Highlands Station and Park-and-Ride Facility – Waiawa Stream

1. INTRODUCTION

The proposed Honolulu High-Capacity Transit Corridor Project (HHCTCP) is an elevated rail line that will provide rapid public transportation service in the City and County of Honolulu on the island of Oahu, Hawaii. Parsons Brinckerhoff (PB), the General Engineering Consultant (GEC) for the project, retained Lyon Associates (LAI) who contracted with Northwest Hydraulic Consultants (NHC) to perform hydraulic evaluations with respect to and as required for planning and design of the proposed Pearl Highlands Station and Park-and-Ride Facility (PHSPF), part of the greater HHCTCP.

According to the existing FEMA flood maps, the proposed PHSPF is in part located within the floodway of Waiawa Stream. The City and County of Honolulu Department of Planning and Permitting (DPP) is the lead agency responsible for Flood Hazard District compliance within the City and County of Honolulu. As mandated by FEMA, DPP requires that all segments of the HHCTCP comply with "No-rise" policy, i.e. have no impact on water levels within floodway zones. The existing FEMA effective model of Waiawa Stream is generally too coarse to be able to accurately simulate the complex details of the proposed project, and to reliably assess the potential impacts of the proposed station and guideway support columns on area water levels. Furthermore, newer, more refined survey and LiDAR data are available to better represent the present geometric conditions of the channel and floodplain. It was therefore decided to develop new and more up-to-date hydraulic computer modeling to re-evaluate present hydraulic conditions and future with-project conditions.

Two modeling programs were utilized for NHC's evaluation of the project. In most flood insurance studies and for the purposes of completing a No-rise analysis for adherence to DPP and FEMA regulations, one-dimensional analysis is typically applied. FEMA's guidelines were developed in a one-dimensional framework and are not suited for two-dimensional analysis. The review process is designed for one-dimensional modeling, in particular by using the Army Corps of Engineers' HEC-RAS program, which is the current accepted standard tool. Therefore, a HEC-RAS model was developed following FEMA guidelines to demonstrate that the final project configuration will not impact 100-year flood levels. Because of the complex hydraulics at the site, a two-dimensional model was also developed to aid in the detailed design of the project and to help identify mitigation features that will eliminate increases in flood levels caused by the proposed PHSPF structure. This model is described in a separate report prepared by NHC and submitted to RTD, titled "Advanced Floodplain Evaluations for Pearl Highlands Station and Park-and-Ride Facility – Waiawa Stream" (April 30, 2010).

The purpose of this report, as described in the following sections, is to provide the data needed to demonstrate that the proposed project will not increase flood levels and therefore, satisfies the DPP and FEMA No-rise requirements. Project background information is provided first, followed by a description of the HEC-RAS model, and then a discussion of No-rise compliance.

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2. STUDY AREA

This report uses the standard cardinal or ordinal directions in its description and orientation of locations and features (i.e. the North, South, East and West compass points). Along Waiawa Stream, North is generally the upstream direction and analogous to Mauka, South is downstream or Makai, East is the left bank (facing downstream) and analogous to Diamondhead or Honolulu, and West is the right bank or Ewa. At the project site itself, the stream follows a slightly different alignment, whereby the left bank becomes somewhat more Mauka than Diamondhead and the right bank more Makai than Ewa. To avoid confusion, we simply adhere to the cardinal directions but also make use of upstream/downstream and left/right terminologies.

The proposed PHSPF transit station structures will be located alongside Waiawa Stream, approximately 1.5 miles upstream of Middle Loch at the north end of Pearl Harbor, and above the influence of tidal fluctuations. While the proposed elevated rail line (guideway) alignment for the HHCTCP will generally follow the Farrington Highway alignment, the PHSPF facility itself will be located alongside the Kamehameha Highway in Pearl City, near its junction with Farrington Highway. The location is approximately 10 miles northwest of Honolulu on the Hawaiian island of Oahu (Figure 2.1). It will consist of an elevated fixed guideway transit system across Waiawa Stream, a transit center station, parking garage, and commuter drop-off areas. Structural components of the PHSPF, including guideway support columns (piers), retaining and building walls, and fill will be constructed within the 100-year regulatory floodplain and a portion of the floodway of the Waiawa Stream, downstream of the Kamehameha Highway bridge and upstream of Farrington Highway. Figure 2.2 depicts the features of the original design for the proposed PHSPF, including the project footprint and a three-dimensional rendering.

The climate of the island of Oahu is generally mild, with fairly uniform temperatures. The wet season extends from October through April, the dry season from May through September. During heavy winter rainfalls, the Pearl City area is subject to frequent flooding from the local streams. The major source of overland flooding is Waiawa Stream and its tributaries.

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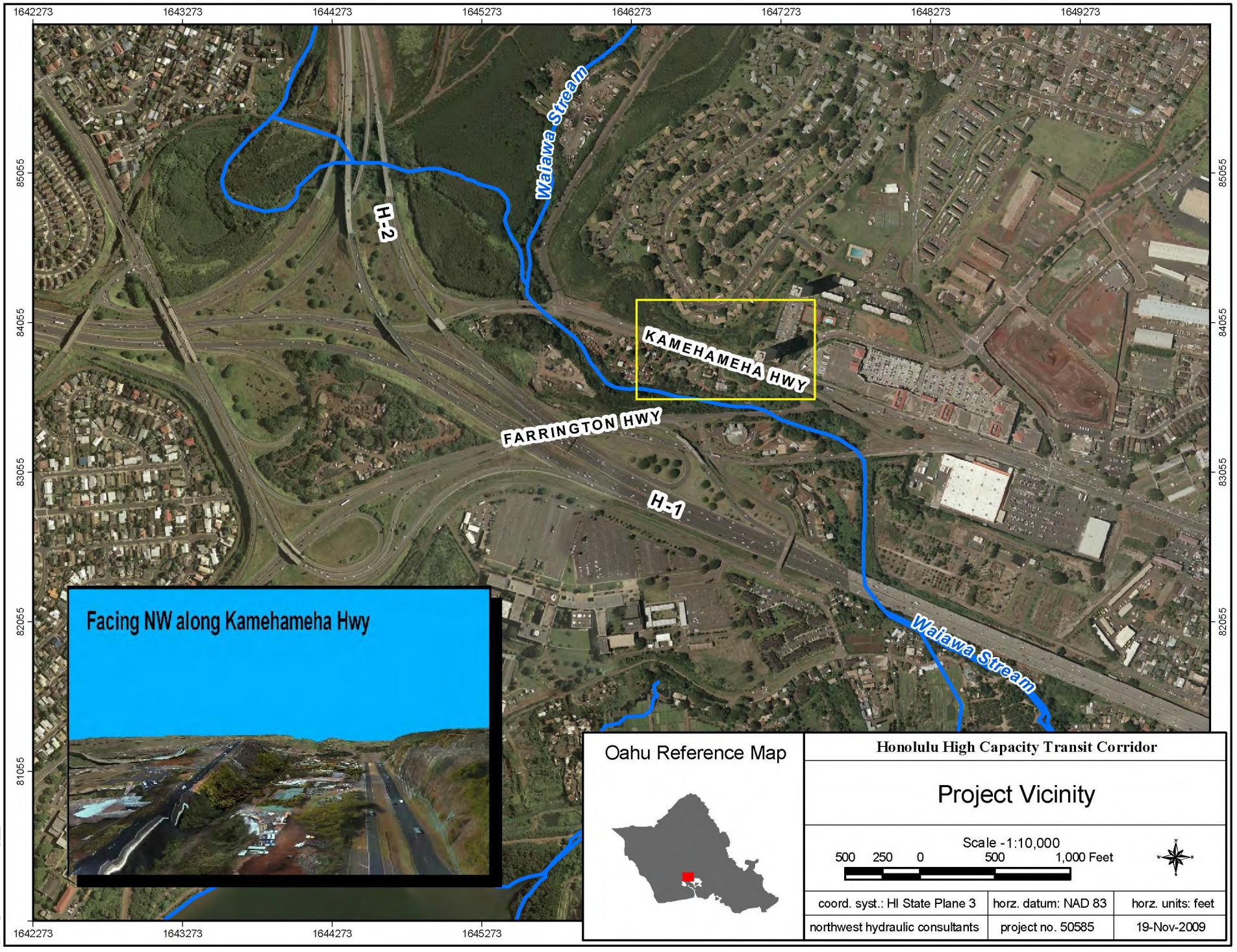
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1646273
1647273
1648273
1649273
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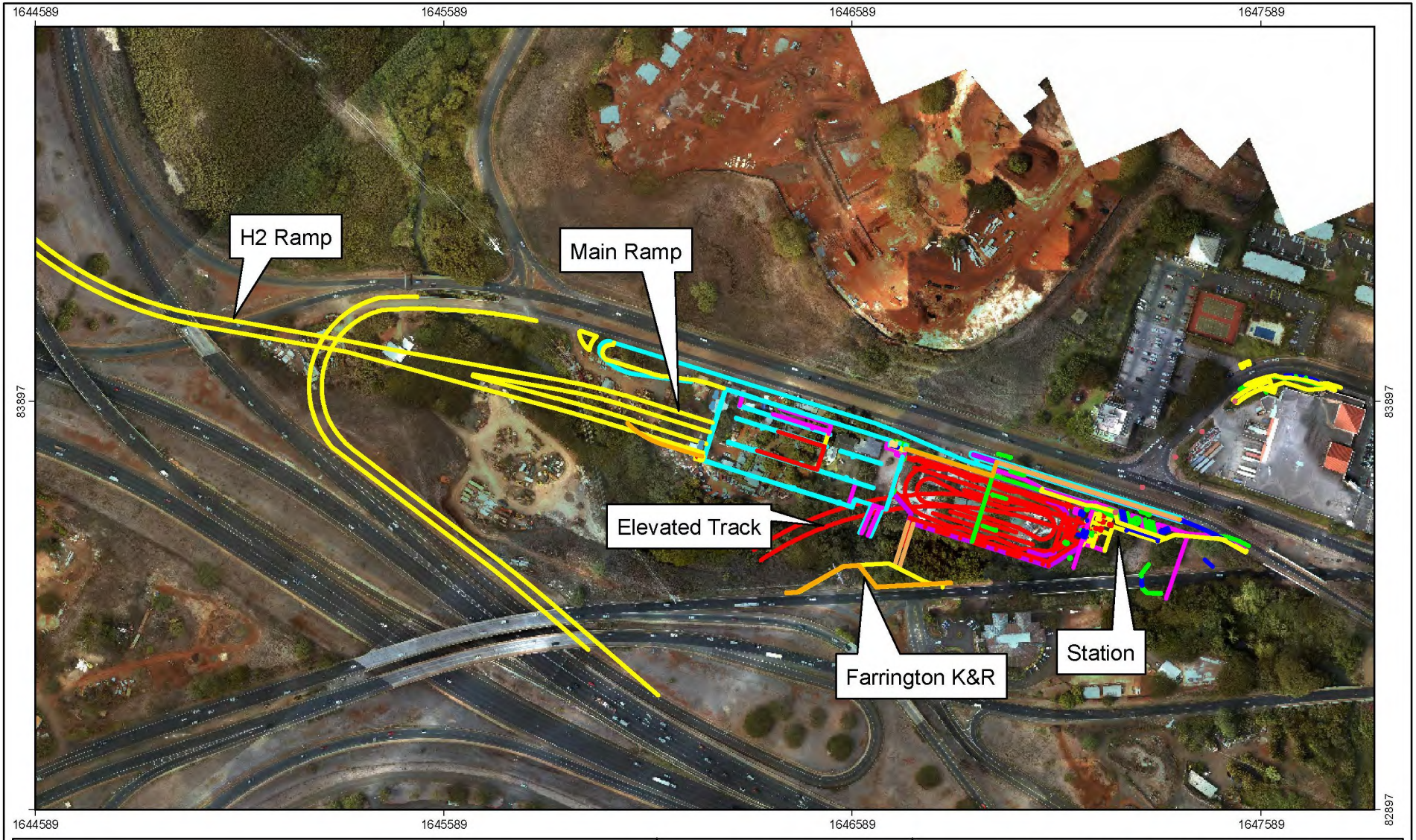
84055
83055
82055
81055
Waiawa Stream
H-2
KAMEHAMEHA HWY
FARRINGTON HWY
H-1
Waiawa Stream
Facing NW along Kamehameha Hwy
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Project Vicinity
Scale -1:10,000
500 250 0 500 1,000 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
19-Nov-2009

Figure 2.1



Legend

- Elevated Transit Facility
- Elevated Parking Garage
- Ramps

Oahu Reference Map

Honolulu High Capacity Transit Corridor

HHCTCP Pearl Highlands Transit Station

Scale -1:4,200

200 100 0 200 400 Feet

coord. syst.: HI State Plane 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50585	19-Nov-2009

Figure 2.2

AR00041540

3. FIELD INVESTIGATIONS AND DATA COLLECTION

This chapter presents results from a field inspection of the study area conducted jointly by PB, NHC, and LAI on January 15, 2009 and data collection activities undertaken to support development of the models of the study stream floodplain area.

3.1 Site Inspection

A field inspection of the study area was conducted on January 15, 2009. The purpose of the site visit was to document channel and floodplain conditions of the stream, pertinent hydraulic features, locations where ponding and ineffective flow are likely, and high water mark observations from past events. Results of the field inspection are discussed herein, with photographs presented in Appendix A.

Waiawa Stream drains a portion of the wet Koolau mountains, and generates the largest 100-year instantaneous peak streamflow on Oahu. In general, the upper watershed has narrow floodplains with steep mountain gradients that quickly convey flow to the project reach. The PHSPF reach of Waiawa is bounded by the two Kamehameha Highway bridges upstream and the two Farrington Highway bridges downstream (see Figure 3.1). The floodplain upstream of Kamehameha Highway consists of old cane fields and agriculture land. Panakauahi Gulch is a major tributary that joins Waiawa less than 500 feet upstream of the Kamehameha Highway bridges. The two Kamehameha Highway bridges were built in 1949 and 1951 and provide a significant constriction to Waiawa Stream. Based on residents' comments and the site visit, these bridges collect large amounts of debris on the bents and overtop during large floods (see Photos 1 and 2).

After crossing Kamehameha Highway, Waiawa Stream changes from a generally southerly to a southeasterly downstream flow direction, through the area known as the "Banana Patch". Through most of this reach the channel is located at the toe of a steep tall slope on the right bank (facing downstream); Farrington Highway and Interstate H-2 are located at the top of this slope. The left bank floodplain has been filled over the years with rocks and debris (see Photos 3 and 4) and developed with numerous residential and accessory buildings between the channel and Kamehameha Highway which parallels the stream. There is, however, a section of more natural land that has not been significantly filled located on the left bank just upstream of the Farrington Highway bridge (see Photo 5). The channel banks within the Banana Patch reach have been protected with various rocks, sticks, concrete pieces and car bodies (see Photos 6 and 7) and there are large trees on both banks that provide a relatively continuous canopy. Based on interviews with Banana Patch residents, debris from the channel is routinely cleaned following flood events (see Photos 8 and 9). Floodplain areas adjacent to the stream are occupied by dwellings, outbuildings, heavy equipment, scrap piles, etc.

Two Farrington Highway bridges cross Waiawa Stream immediately downstream of the Banana Patch (see Figure 3.1). The upstream bridge carries the westbound lanes of Farrington Highway and was constructed in 1933. This bridge is a significant constriction to Waiawa Stream due to

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its small opening, heavy debris collection, presence of fill downstream of the bridge, and large piers which are misaligned with the direction of flow through the bridge, resulting in a general skew of more than 50 degrees (see Photos 10 to 13). Thus, the upstream Farrington Highway bridge significantly controls water surface elevations in the PHSPF reach upstream. The flow in Waiawa channel is directed nearly parallel to the bridge on the upstream side until the flow hits the left bank abutment and is forced to turn through the bridge. These flow patterns have caused local erosion along the left bank upstream of the bridge and scour at the base of the two piers located nearest the left bank (Photos 13 and 14). On the downstream side of the bridge the right bank has been filled, which significantly blocks four of the six bridge spans from conveying much flow and has led to deposition under the bridge (Photos 15 and 16).

The reach between the two Farrington Highway bridges is approximately 600 ft long. The left overbank is mostly natural area and heavily vegetated (Photo 17), while the right bank has been filled and includes numerous buildings on both the Hawaii Laborers' Union Training Program and Naval properties. There is a submerged concrete weir that was used by the USGS for Waiawa Stream flow gauging from 1952 to 2004, located about 50 ft upstream of the downstream Farrington Highway bridge (Photo 17). The downstream eastbound Farrington bridge (Photo 18) was built in 1951 and also constricts flow in Waiawa Stream. The bridge deck partially interacts with large floods and the right bank approach fill blocks over 250 ft of floodplain until the road is overtopped. Downstream of this bridge is a relatively straight and natural reach (see Photo 19) for approximately 800 ft until Interstate H-1 crosses the stream.

During the site visit several residents were interviewed about historic flooding along Waiawa Stream. Figure 3.2 shows the locations of several observed anecdotal high water marks from the March 21, 1991 and December 11, 2008 flood events.

3.2 Channel Survey

Existing channel survey information along Waiawa Stream was needed to support model development and hydraulic analyses for the HHCTCP. Following the site inspection of the study area, NHC developed a detailed channel survey program. The survey was conducted by ControlPoint Surveying, Inc. (ControlPoint) in the spring of 2009. The survey data included channel cross sections at various locations along the study reach, beginning upstream of Kamehameha Highway and extending downstream to the Interstate H-1 crossing. Altogether, approximately 20 cross sections were surveyed along Waiawa Stream. They also surveyed key feature elevations at the Kamehameha and Farrington Highway bridge crossings, elevations for the high water marks identified during the site visit, and miscellaneous floodplain feature elevations. The survey points are shown on a topographic map of the study reach in Figure 3.3.

3.3 Topographic Data

Bare ground surface topography was needed to support development of the hydraulic models of the study area, and for mapping with-project inundation limits. Topographic data used in this study included 1 ft contour interval Light Detection and Ranging (LiDAR) based on topography

AR00041542

data within the HHCTCP corridor obtained by John Chance Land Surveys, Inc. (JCLS) in 2007 (official data for the project) and 5 ft by 5 ft grid FEMA LiDAR data developed in 2006 for FEMA Hurricane Study and covering the entire island coast including Pearl Highlands and Waiawa Stream area. These topographic data sets were provided to the project team by the City and County of Honolulu.

The 2007 project corridor topography data and 2006 FEMA LiDAR data were compared in ArcGIS to ensure their consistency and compatibility. The comparison indicated a close agreement (within one foot) of ground elevations between these topographic data sets for most overlapping areas. A comparison was also made between the ground survey data along Waiawa Stream and the project topography. The difference in ground elevations in these datasets was small in overbank regions, typically within 0.5 ft. The ground survey data was used for the channels and around bridge structures, with the project topography used for overbank areas. The only location the FEMA LiDAR dataset was used for the Waiawa Stream analysis was downstream of the Farrington Highway bridges beyond the extents of the project topographic dataset. Shaded ground contours of the final merged digital elevation model (DEM) developed and used by NHC are shown in Figure 3.3.

3.4 Other Data Sources

Bridge plans were provided by PB staff for the four Kamehameha and Farrington Highway crossings of Waiawa Stream. FEMA effective FIS, DFIRM mapping, and HEC-2 hydraulic models were obtained directly from FEMA by NHC. Oceanit identified (flagged) and ControlPoint surveyed in the ordinary high water marks along the Waiawa Stream channel between Farrington and Kamehameha Highways. These marks were then connected by NHC to estimate the ordinary high water mark boundary along the entire study reach.

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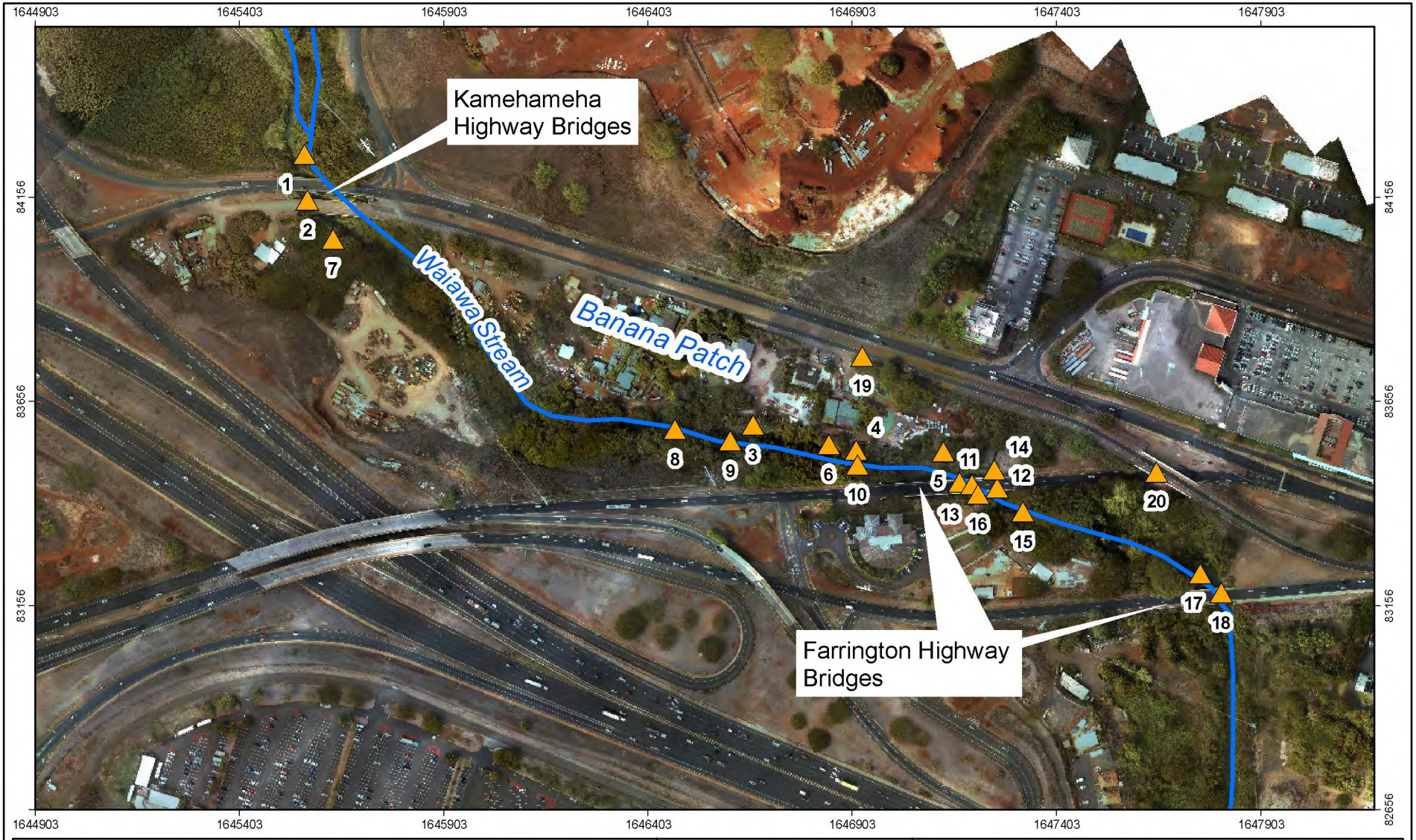

Legend

▲ Photograph Location

Oahu Reference Map

Honolulu High Capacity Transit Corridor

Map of Approximate Photographed Locations During Site Visit

Scale -1:4,200

200 100 0 200 400 Feet

coord. syst.: HI State Plane 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50585	19-Nov-2009

Figure 3.1

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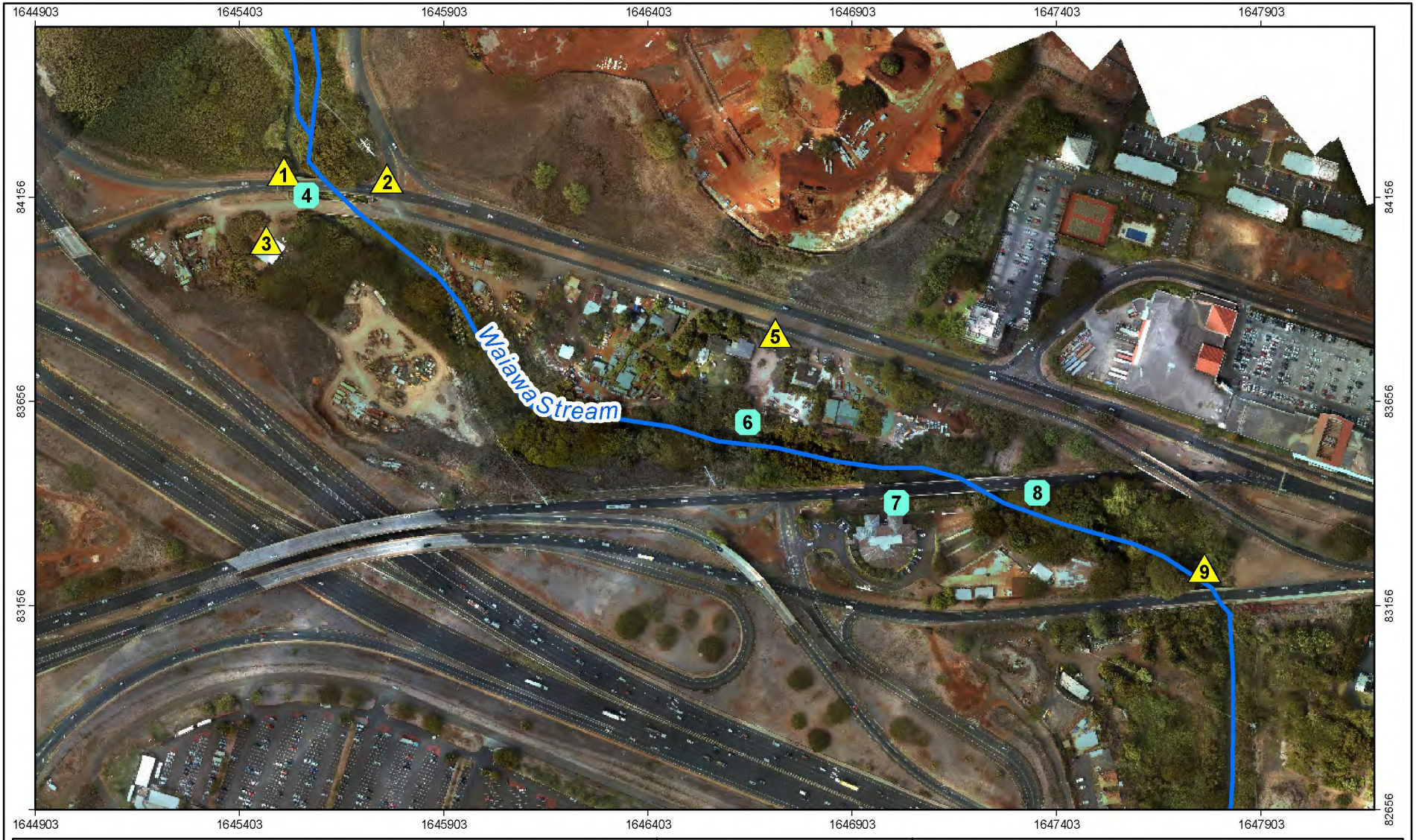


Legend

High Water Marks

1991 Event

2008 Event

Oahu Reference Map

Honolulu High Capacity Transit Corridor

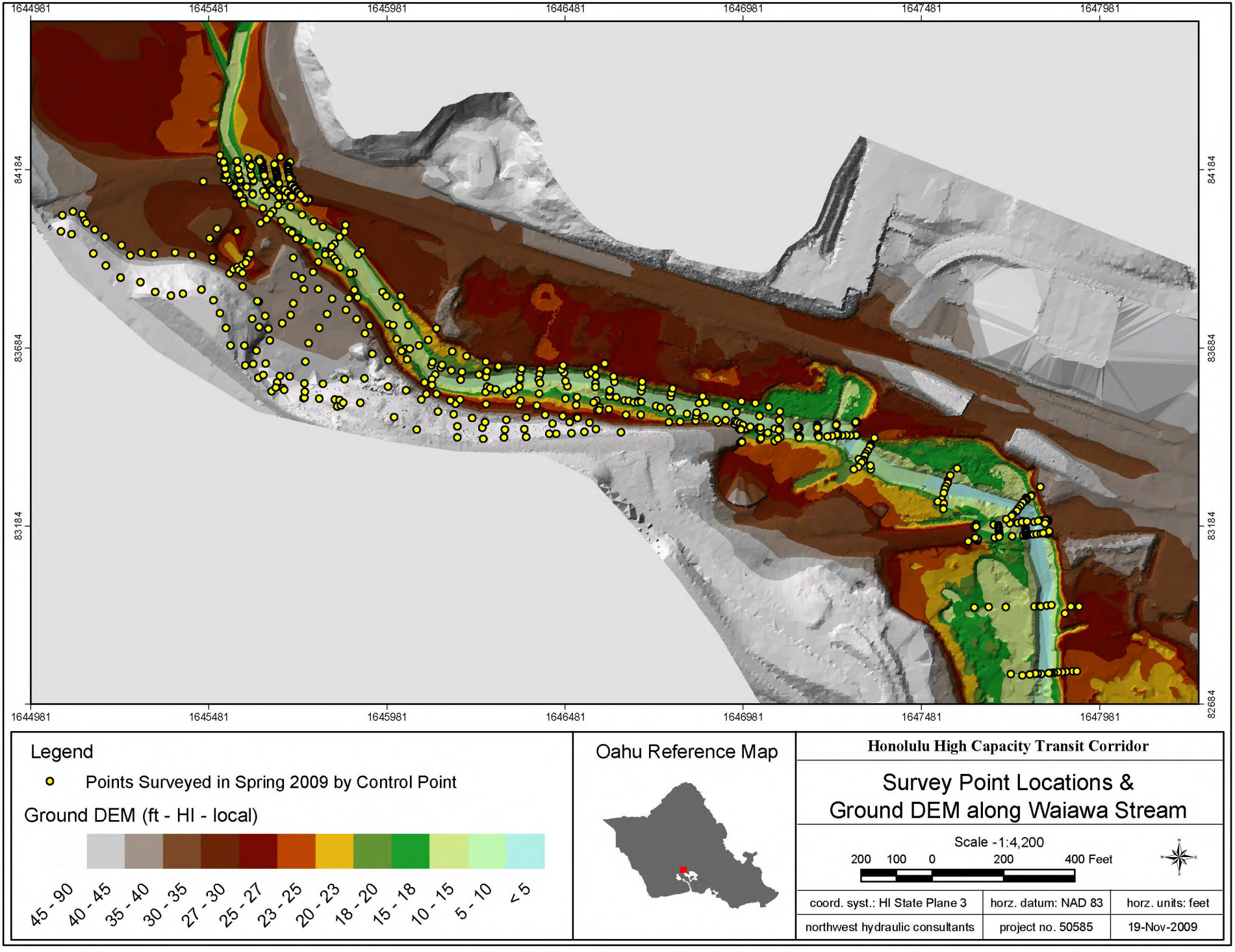
Locations of High Water Marks Obtained During Site Visit

Scale -1:4,200

200 100 0 200 400 Feet

coord. syst.: HI State Plane 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50585	19-Nov-2009

Figure 3.2


1644981
1645481
1645981
1646481
1646981
1647481
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84184
83684
83184
82684
Legend
Points Surveyed in Spring 2009 by Control Point
Ground DEM (ft - HI - local)
45 - 90
40 - 45
35 - 40
30 - 35
27 - 30
25 - 27
23 - 25
20 - 23
18 - 20
15 - 18
10 - 15
5 - 10
< 5
Oahu Reference Map
Honolulu High Capacity Transit Corridor
Survey Point Locations &
Ground DEM along Waiawa Stream
Scale - 1:4,200
200 100 0 200 400 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50585
19-Nov-2009

Figure 3.3

4. HYDROLOGY

The purpose of the hydrologic analysis was to review and analyze existing discharge data from prior studies and stream gages, and to update these data as necessary to provide inputs to support the hydraulic analysis and modeling.

4.1 Review of FEMA Effective FIS Hydrologic Analysis

The effective Flood Insurance Study (FIS) for the City and County of Honolulu (FEMA, 2004) was originally published on November 20, 2000 and subsequently revised and re-issued on September 30, 2004. The FIS documents hydrologic and hydraulic analyses and provides maps for dozens of streams on the island of Oahu. These analyses utilized a variety of methods and data sources to compute flood quantiles and map the Base Flood Elevations (BFEs).

The effective FIS utilized a nomograph shown on Plate 6 of the City and County of Honolulu Storm Drainage Standards (DPP 2000), which estimates 100-year peak discharge for streams and conduits as a function of drainage area. Three lines represent peak discharge as a power function of drainage area for 3 distinct geographic regions, windward Oahu (highest peak flows per unit area), central Oahu (2nd highest peak flows per unit area) and west Oahu (lowest peak flows per unit area). According to a label, the nomograph is based on USGS data as of 1988. The effective FIS states that 10-year and 50-year peak flow values were not computed; however, the 500-year peak is described as having been estimated using the ratio of 500-year to 100-year peak flow derived from data from USGS Gage 16216000.

4.2 Re-Analysis of Waiawa Stream Frequency Curve

NHC used the entire available stream record of peak annual flows (52 years, water years spanning 1953-2004) from USGS gage 16216000 to fit a Log-Pearson III frequency curve. The USGS gage was located on the upstream side of the eastbound lanes of Farrington Highway (lower bridge) in very close proximity to proposed HHCTCP facilities.

The USGS data were fit according to Bulletin 17b (WRC 1981) procedures using the HEC-SSP computer program. Detailed output from the HEC-SSP frequency analysis is provided in Appendix B. Summary results are shown in Figure 4.1 and in Table 4.1. 100-year and 500-year quantile estimates from the re-analysis are within 5% of the values reported in the effective FIS. This suggests that the Waiawa record may have been used to formulate the City's Plate 6 flood nomograph for the Central Oahu region.

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4.3 Application of Hydrologic Analysis to Hydraulic Modeling of Waiawa Stream for HHCTCP Facilities

The hydraulic analysis required for the PHSPF examined flow, water level, and velocity pattern conditions for a range of flood discharges from the 2-year to the 100-year events. The flood quantiles shown in Table 4.1 (both the FIS and revised HHCTCP values) are suitable discharge inputs for hydraulic modeling of the entire reach of Waiawa Stream that is expected to be affected by the proposed PHSPF. Because the FIS 100-year discharge is very close to the newly computed value (within 5%), the subsequent hydraulic analysis for No-rise compliance was therefore based upon the presently adopted FIS (Plate 6) value of 34,000 cfs.

Table 4.1 Waiawa Stream Flood Quantile Estimates

Study	Location	Drainage Area (sq mi)	2-yr	5-yr	10-yr	50-yr	100-yr	500-yr	Source or Method
Effective FIS	"Mouth"	27.34					34,000	43,800	Plate 6, City and County of Honolulu Storm Drainage Standards, 1988 plus ratio from LP-III, Bulletin 17b.
HHCTCP (NHC), Effective FIS verification	USGS 16216000 (upstream of E.bound Farrington Hwy)	26.40	8,477	15,032	19,740	30,507	35,128	45,838	Bulletin 17B Frequency Analysis via HEC-SSP, 52 years of record. (WY 1953-2004)
Change from Effective FIS			N/A	N/A	N/A	N/A	3.3%	4.7%	

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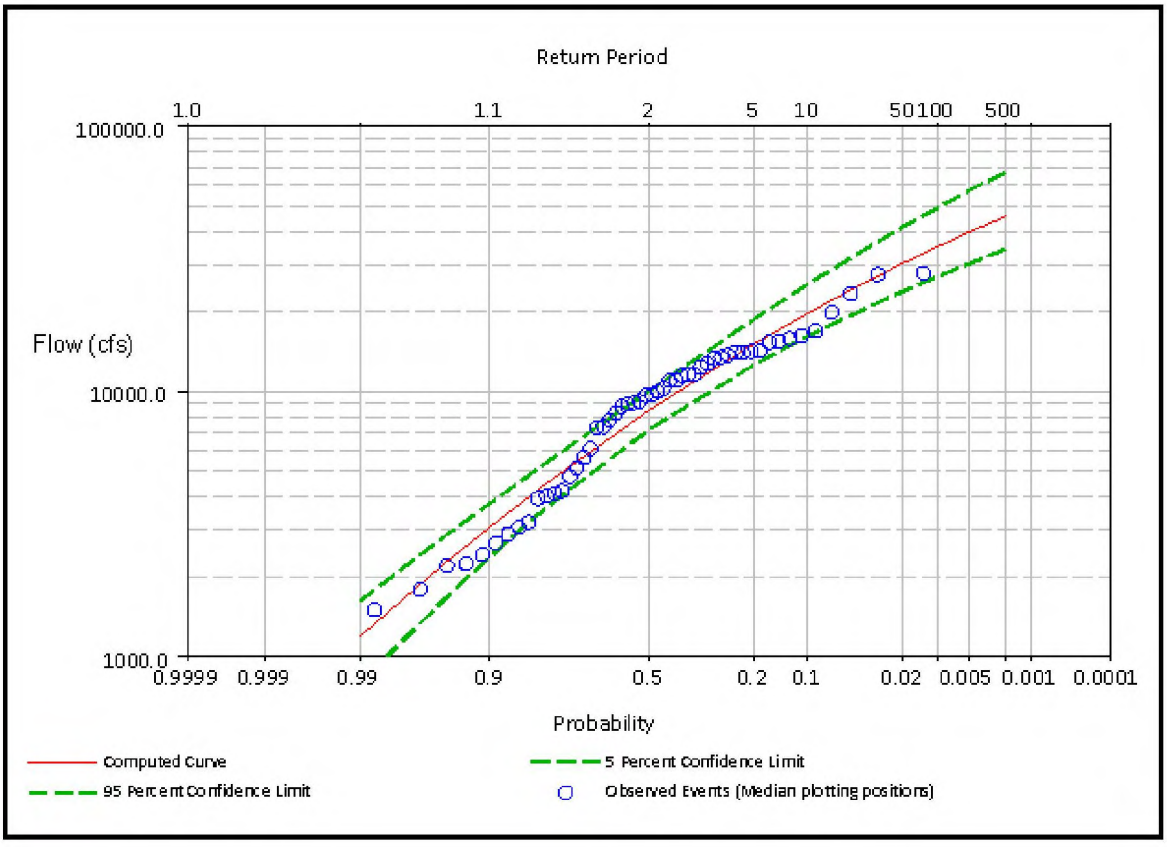
Figure 4.1 Flood Frequency Curve for Waiawa Stream near Pearl City, USGS Gage 16216000.



5. HEC-RAS MODEL DEVELOPMENT

5.1. FEMA Effective, Duplicate Effective, and Existing Conditions Models

The effective FEMA HEC-2 model for Waiawa Stream was obtained from FEMA and reviewed. The HEC-2 model was imported into and converted to HEC-RAS (USACE 2009), initially retaining the original cross sections and bridge geometries from the HEC-2 model. This HEC-RAS model is the "duplicate effective" model for this study. It was determined however that the model is too coarse to be a reliable and useful tool for assisting the design or evaluating impacts of the complex details of the PHSPF project. Furthermore, as detailed in Chapter 3, new topographic data are available for the channel and floodplain, which would be more up-to-date than the effective FEMA cross sections. Because of these reasons, it was determined that a more refined model was needed to provide a defensible tool for evaluating realistic impacts of project alternatives and demonstrating to agencies No-rise of the final project configuration. Therefore, using the new project topography along with the Waiawa channel cross-sectional surveys that were obtained, a revised model was developed between XS J and N in the effective FEMA FIS to create a much more detailed "existing conditions model" of the project reach. Figure 5.1 shows the location of the cross sections added to the HEC-RAS model in the project reach.

5.2 Calibration and Verification

Hydraulic model calibration is a process by which model input parameters, most typically coefficients such as roughness and other empirical inputs including coefficients for weir flow, bridge and contraction/expansion losses, are adjusted (within reasonable limits) in order to more closely match observed high water levels from past floods. This requires knowledge, or at least a reasonable estimation, of the actual discharge from the historic event. Ideally, high water and discharge data are available for more than one past event, in which case the adjusted coefficients from the calibration event can be tested against a second (or third, etc.) event. If the additional event(s) perform well, without requiring re-adjustment of the input parameters, the model is said to be verified.

Measured and anecdotal high water data are available from the 1991 flood, which measured 27,600 cfs at the USGS Waiawa gage 16216000 and is approximately a 25-year event and only slightly smaller than the 27,950 cfs event of record in 1982 (no high water marks are available for the 1982 flood). A more recent, but substantially smaller event occurred in 2008 after the USGS gage ceased operation. The discharge for this event has been roughly estimated to be on the order of a 5-year event, or approximately 12,000 cfs. Table 5.1 lists the location and estimated elevations of the various measured high water marks for these two flood events. The high water mark locations are identified on Figure 3.2 (included with the figures in Chapter 3).

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Table 5.1 High Water Mark Locations Used in Calibration			
High Water Mark ID	Event	Location	Estimated Elevation (ft)
1	Mar 21, 1991	Overtopped guardrail on upstream Kamehameha Highway bridge	36.6
2	Mar 21, 1991	Light pole at intersection was submerged	34.8
3	Mar 21, 1991	Water above the third step of Irwin Kawano's staircase	33.1
4	Dec 11, 2008	Debris on low chord of upstream Kamehameha Highway bridge	29.5
5	Mar 21, 1991	Kamehameha Highway was not significantly overtopped	32.0 to 34.0
6	Dec 11, 2008	High bank was not overtopped	24.5
7	Dec 11, 2008	Debris downstream of bridge	20.4
8	Dec 11, 2008	Debris downstream of bridge	20.8
9	Mar 21, 1991	USGS Gage at concrete weir	24.2

Variables adjusted in the HEC-RAS model during the calibration included bridge routine selection, roughness values, cross section orientation, and contraction and expansion losses. Table 5.2 lists typical roughness values used in the calibrated model, while Table 5.3 lists the contraction and expansion losses by cross section. Simulated water surface profiles and observed high water marks for the calibration events are shown in Figure 5.2. One will notice that the flood profiles are slightly higher than a number of the high water marks. There is significant uncertainty in the accuracy of many of the marks because they are based upon the very approximate anecdotal information provided by local residents (the 1991 event occurred many years ago). Therefore, it was determined that it would be best if the HEC-RAS model computes flood profiles that are slightly conservative, or in the case of calibration, reproduce the highest water marks.

Table 5.2 Calibrated Manning's Roughness Values in HEC-RAS Model	
Land Use	n Value
Road or Paved	0.015
Channel	0.025
Shade	0.035
Mostly Open	0.040
Light Vegetation	0.060
Heavy Vegetation	0.070
Dense Vegetation	0.080

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Table 5.3 Calibrated Contraction and Expansion Coefficients in HEC-RAS Model					
HEC-RAS Cross Section	Contraction	Expansion	HEC-RAS Cross Section	Contraction	Expansion
7581	0.10	0.30	9419	0.10	0.30
8400	0.10	0.30	9471	0.10	0.30
8450	0.10	0.30	9558	0.10	0.30
8500	0.10	0.30	9626	0.10	0.30
8550	0.10	0.30	9682	0.10	0.30
8600	0.10	0.30	9780	0.10	0.30
8650	0.10	0.30	9924	0.10	0.30
8690	0.10	0.30	10028	0.10	0.30
8720	0.30	0.50	10148	0.10	0.30
8750	0.30	0.50	10242	0.10	0.30
8780	0.30	0.50	10328	0.10	0.30
8810	0.30	0.50	10404	0.10	0.30
8840	0.30	0.50	10500	0.10	0.30
8870	0.30	0.50	10600	0.10	0.30
8900	0.15	0.40	10720	0.15	0.50
8950	0.15	0.40	10740	0.15	0.50
8980	0.15	0.40	10760	0.15	0.50
9000	0.15	0.40	10780	0.15	0.50
9011	0.15	0.40	10850	0.10	0.30
9050	0.15	0.40	10900	0.10	0.30
9090	0.15	0.40	10950	0.10	0.30
9130	0.15	0.40	11045	0.10	0.30
9171	0.15	0.40	11145	0.10	0.30
9253	0.15	0.40	11785	0.10	0.30
9338	0.15	0.40	11995	0.10	0.30

5.3 HEC-RAS FEMA Floodway Model

Once the "existing conditions" HEC-RAS model was complete, NHC modified the channel cross sections to confine flows within the limits of the effective FEMA floodway. This was done for two reasons. First, it was to ensure that the PHSPF was designed to accommodate future development within the flood fringe (i.e. on the floodplain landward of the floodway boundary). Under current floodplain development rules, the portion of the floodplain landward of the floodway boundary could theoretically be completely filled which would eliminate flood conveyance in the flood fringe. This in-turn would elevate flood levels within the stream

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corridor which needs to be accounted for in the facility design. Second, when a structure will be built within the FEMA floodway, one is required to show that the features will have no impact on FEMA floodway or 100-year floodplain water surface elevations.

To ensure that the cross sections in the refined model include the correct floodway boundaries, NHC identified the boundary limits using data from the existing FEMA Digital Flood Insurance Rate Map (DFIRM) for the Waiawa Stream, Flood Insurance Study (FIS), and the effective HEC-2 floodway model.

To demonstrate that the floodway encroachments in the new "existing conditions" model do not cause flood levels to increase more than one foot, water surface profiles were computed for both the with and without floodway boundaries. These profiles are compared in Figure 5.3, and the actual computed water surface elevations at each cross section are compared in Table 5.4.

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Table 5.4 HEC-RAS Water Surface Elevation Comparison For Existing Condition Simulations With and Without the FEMA Floodway								
HEC-RAS Cross Section	100-year WSE	Floodway WSE	WS Increase Caused by Floodway		HEC-RAS Cross Section	100-year WSE	Floodway WSE	WS Increase Caused by Floodway
7581	26.920	27.500	0.580		9419	36.635	37.183	0.548
8400	27.327	27.693	0.366		9471	36.474	37.061	0.587
8450	27.718	28.427	0.709		9558	36.813	37.370	0.557
8500	28.631	29.261	0.630		9626	37.065	37.592	0.527
8550	28.051	28.754	0.703		9682	37.040	37.571	0.531
8600	26.417	27.414	0.997		9780	36.949	37.465	0.516
8650	28.671	29.570	0.899		9924	37.390	37.819	0.429
8690	29.312	29.889	0.577		10028	37.029	37.364	0.335
8720	29.820	30.043	0.223		10148	36.893	37.166	0.273
8750	29.832	30.180	0.348		10242	36.664	36.923	0.259
8780	29.405	29.840	0.435		10328	36.659	36.993	0.334
8810	31.942	32.304	0.362		10404	36.751	37.218	0.467
8840	31.898	32.276	0.378		10500	38.080	38.107	0.027
8870	31.916	32.395	0.479		10600	38.343	38.934	0.591
8900	32.657	33.101	0.444		10720	38.940	39.517	0.577
8950	35.432	36.093	0.661		10740	39.108	39.672	0.564
8980	35.120	35.811	0.691		10760	39.156	39.710	0.554
9000	35.533	36.179	0.646		10780	39.363	39.981	0.618
9011	35.307	35.888	0.581		10850	39.063	39.729	0.666
9050	35.128	35.709	0.581		10900	39.036	39.660	0.624
9090	34.537	35.147	0.610		10950	39.187	39.782	0.595
9130	34.492	35.168	0.676		11045	39.286	39.819	0.533
9171	34.515	35.221	0.706		11145	38.220	38.955	0.735
9253	35.151	35.828	0.677		11785	40.607	41.119	0.512
9338	36.453	36.993	0.540		11995	40.333	40.874	0.541

5.4 Evaluation of Project Alternatives

NHC used the "existing condition" floodplain and floodway HEC-RAS models to evaluate the original PHSPF facility layout. The model revealed that the project, as initially proposed, would have caused an unacceptable impact in the 100-year water surface and flood inundation. NHC then used the model to make some very rough estimates of the amount of excavation that would need to occur along the project site to eliminate the impact of the proposed facility. At this point

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NHC set the HEC-RAS model aside and developed a two-dimensional model of the project area in order to further refine the project and mitigation measures. Development and use of the two-dimensional model is presented in a separate design report to PB titled "Advanced Floodplain Evaluations for Pearl Highlands Station and Park-and-Ride Facility – Waiawa Stream" (April 30, 2010). Following final evaluation of the project alternatives, NHC came back and modified the "existing condition" HEC-RAS model to create a "post-project" HEC-RAS model. The model was modified to include all of the final PHSPF features as well as the mitigation which was required to offset increases in the water surface elevation due to the project (without mitigation). The HEC-RAS modeling that was completed to demonstrate that the final proposed project satisfies the "No-rise" requirement is described in the next chapter.

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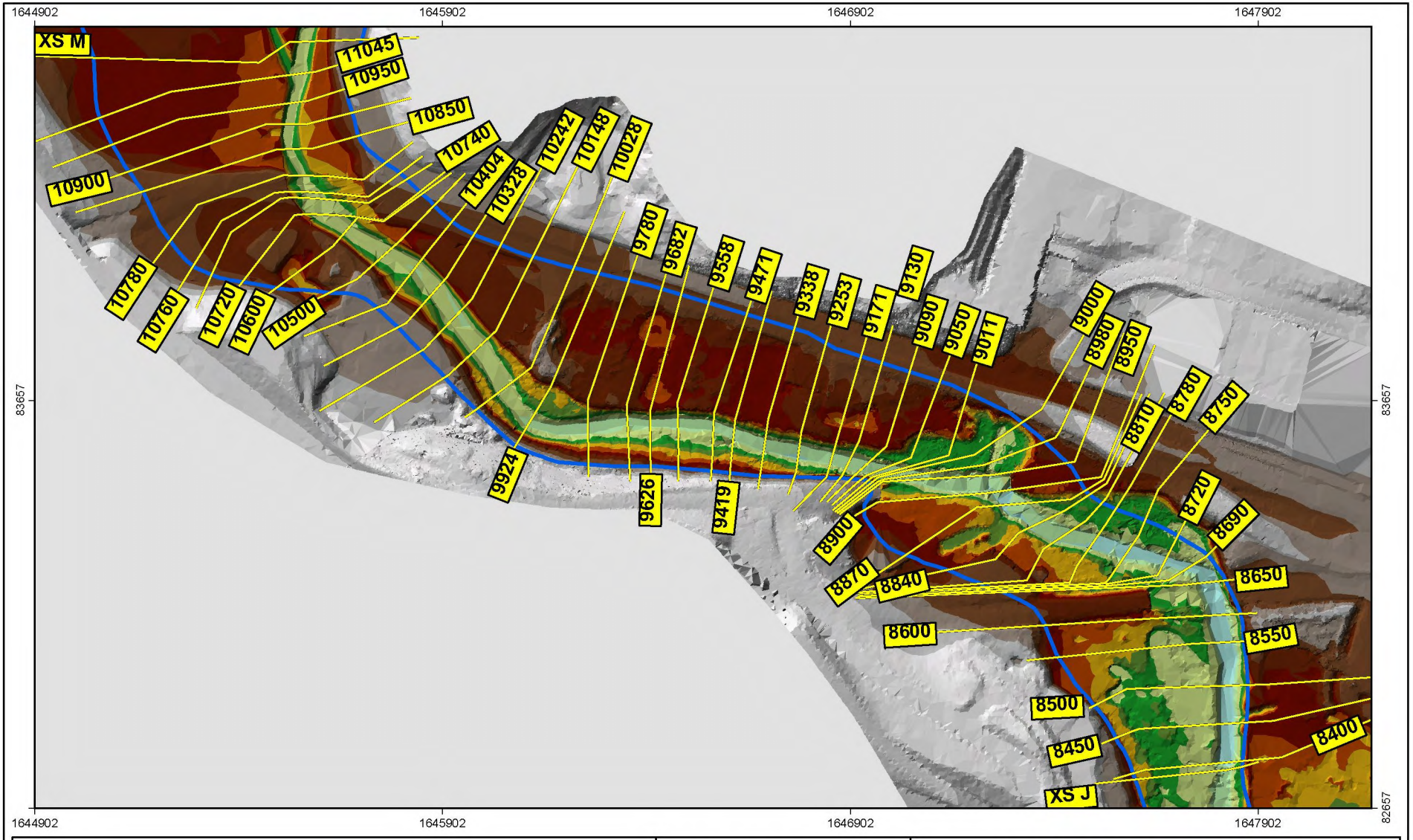
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Legend
Model Cross-Sections
FEMA Floodway
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Oahu Reference Map
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No-Rise HEC-RAS Model
Cross-Section Locations
Scale -1:4,200
200 100 0 200 400 Feet
coord. syst.: HI State Plane 3
horz. datum: NAD 83
horz. units: feet
northwest hydraulic consultants
project no. 50530
30-Apr-2010

Figure 5.1

Honolulu High Capacity Transit Corridor		
HEC-RAS Model Water Surface Profiles		
Calibration Events		
northwest hydraulic consultants	project no. 50585	19-Nov-2009

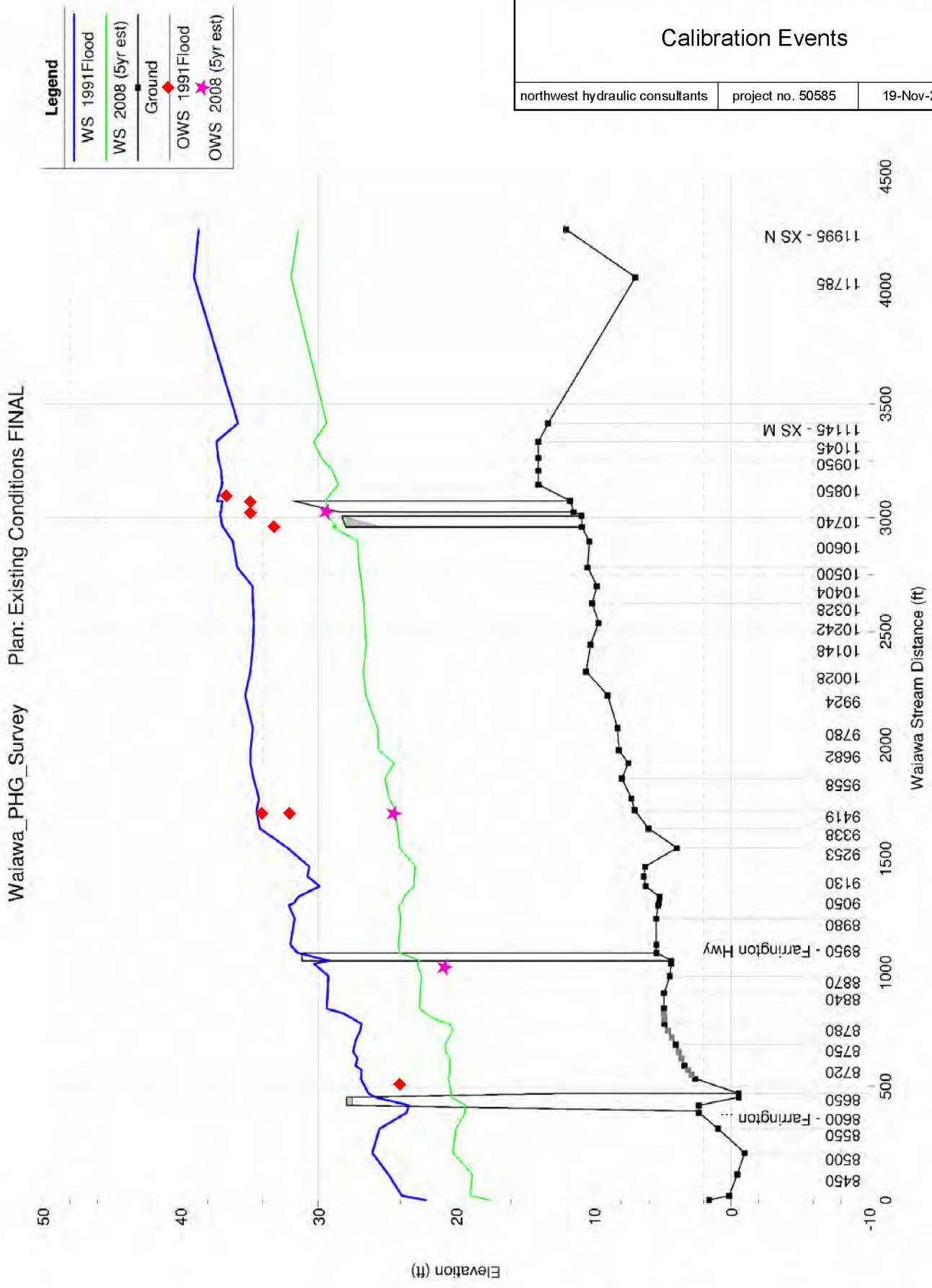

Figure 5.2

Honolulu High Capacity Transit Corridor		
HEC-RAS Model Water Surface Profiles		
Existing Conditions 100-year Floodplain and Floodway		
northwest hydraulic consultants	project no. 50585	19-Nov-2009

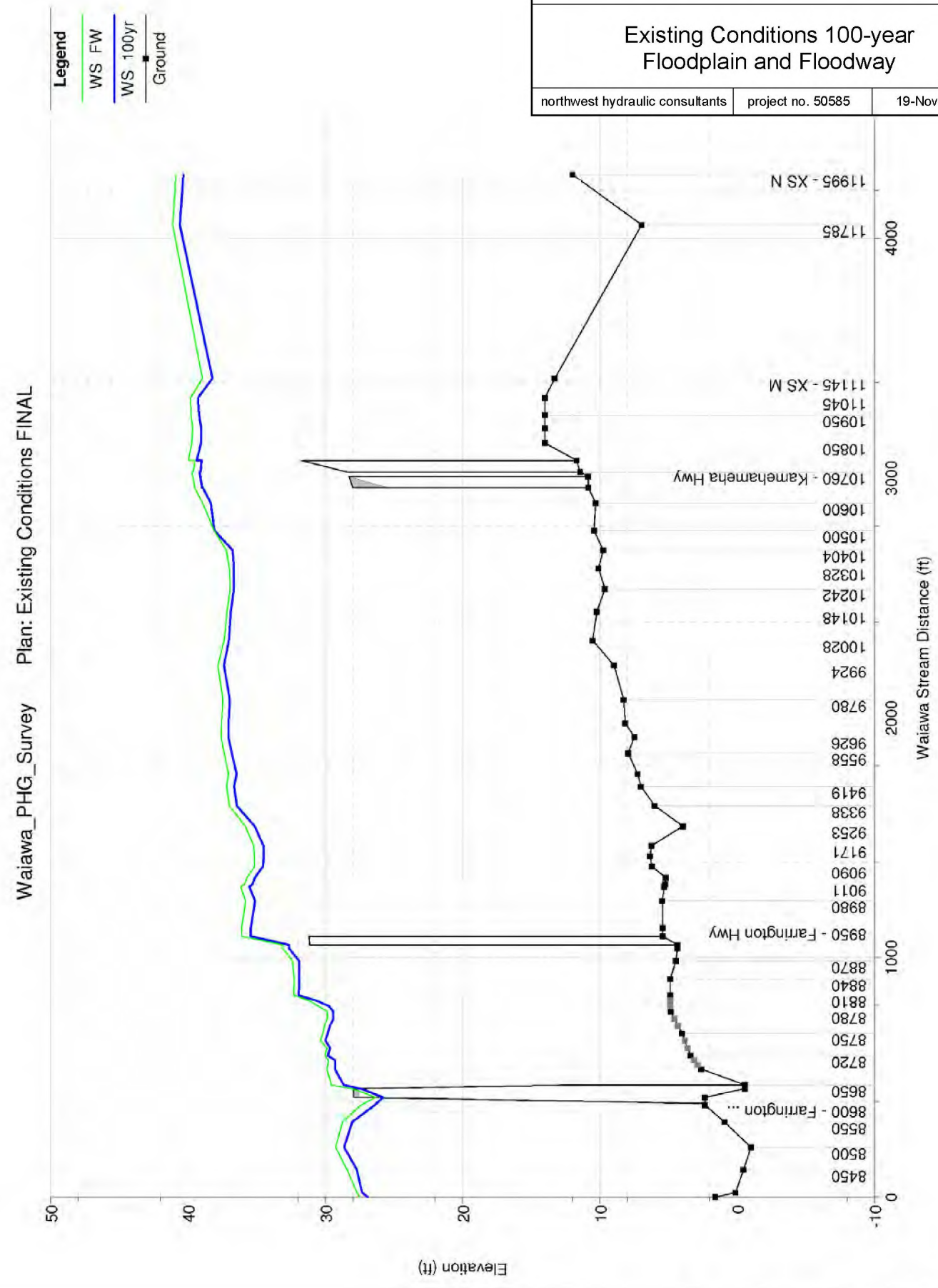

Figure 5.3

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6. "NO-RISE" CERTIFICATION AND DOCUMENTATION

City and County of Honolulu flood hazard district regulation 21-9.10.5(b) states that construction of features within a FEMA designated floodway are:

"…prohibited unless certification and supporting data including hydrologic and hydraulic analyses performed in accordance with standard engineering practice are provided by a licensed engineer demonstrating that the proposed encroachment will not cause any increase in regulatory flood elevations during the occurrence of the regulatory flood."

To demonstrate compliance with this condition, the City and County requires the project proponent to fill out a document called "Certification of a "No-Rise" Determination for a Proposed Floodway Development", a form that was created by FEMA (Appendix C). Also appended to this is a document titled "Certification Requirement for Simple Floodway Encroachments", also prepared by FEMA (Appendix D). It spells out the data that need to be prepared and submitted to demonstrate compliance with the "No-Rise" condition. For the PHSPF project the following hydraulic data are required:

1. Hydraulic backwater model of the 100-year flood and floodway water-surface profiles for the following:
    a. Duplicate of the effective Flood Insurance Study (FIS) model.
    b. Existing-conditions (effective FIS) model modified to include cross sections through the project site. Cross sections must reflect condition prior to construction of the project.
    c. Post-project conditions model. This model must include cross section through the project site reflecting floodplain conditions after construction for the project.
2. A copy of the appropriate NFIP map showing the existing floodway and indicating the project area.
3. Topographic mapping of the entire project area indicating the locations of all cross sections used in the modified hydraulic model and a plan view of all project elements.
4. Construction plans, certified by a registered professional engineer, for all project elements, including those measures employed to provide additional effective conveyance.

Data demonstrating that the proposed PHSPF project complies with the "No-Rise" requirement are presented below.

6.1 No-Rise Certification

A signed and stamped copy of the FEMA form "Certification of a "No-Rise" Determination for a Proposed Floodway Development" is presented in Appendix C.

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6.2 Hydraulic Backwater Model

As stated in Chapter 5, the FEMA floodplain maps for the Waiawa stream were created using HEC-2, a simple one-dimensional steady state water surface profile computer model. The first step in developing the data required to demonstrate No-rise is to convert the HEC-2 model to HEC-RAS to create a "duplicate effective model". This new model must then be modified to create both an "existing conditions model" and a "post-project conditions model" which together are used to demonstrate No-rise. NHC has completed these steps as explained below.

6.2.1 Duplicate Effective Model

As described in Chapter 5, NHC obtained the original FEMA HEC-2 effective model from FEMA and converted it to HEC-RAS to create a "duplicate effective model".

6.2.2 Existing Conditions Model

As described in Chapter 5, NHC modified and refined the "duplicate effective model" to contain sufficient detail to appropriately evaluate the impact of the proposed PHSPF facilities. Modifications included a revised cross section layout, updated geometry and modeling of the four Kamehameha and Farrington Highway bridges, and calibration of ineffective areas, roughness, and contraction and expansion losses. The updated model reach ties into the effective FIS at XS J downstream and XS M upstream. Tables 5.2 and 5.3 list the final calibrated values used for roughness, and parameters for contraction and expansion losses. The final cross section layout and model reach of the existing conditions HEC-RAS model are illustrated in Figure 5.1.

6.2.3 Post-Project Conditions Model

Once NHC and the PB team settled on a final design for the PHSPF as described in "Advanced Floodplain Evaluations for Pearl Highlands Station and Park-and-Ride Facility – Waiawa Stream" (April 30, 2010) prepared by NHC for PB, NHC then modified the "existing condition model" to include the final project configuration (with mitigation, see Figure 6.1), and thus created a "post-project conditions model". Cross section locations in this model are the same as in the "existing conditions model" (Figure 5.1).

6.2.4 Flood Profiles and No-Rise Confirmation

The "existing condition" versus proposed "post-project" 100-year without floodway and with floodway water surface profiles are compared in Figures 6.2 and 6.3, respectively. The actual computed water surface elevations at each cross section are compared in Tables 6.1 and 6.2. The figures and tables demonstrate that in no place does the post-project profile extend above the existing condition profile, for both the 100-year with and without floodway case; demonstrating that the project meets the "No-rise" requirement.

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Table 6.1 HEC-RAS Water Surface Elevation Comparison of 100-year Floodplain (without Floodway) Simulations								
HEC-RAS Cross Section	Existing Conditions	Proposed Conditions	WS Increase Caused by Proposed Conditions		HEC-RAS Cross Section	Existing Conditions	Proposed Conditions	WS Increase Caused by Proposed Conditions
7581	26.920	26.920	0.000		9419	36.635	34.199	-2.436
8400	27.327	27.327	0.000		9471	36.474	34.604	-1.870
8450	27.718	27.718	0.000		9558	36.813	34.813	-2.000
8500	28.631	28.631	0.000		9626	37.065	34.713	-2.352
8550	28.051	28.051	0.000		9682	37.040	34.671	-2.369
8600	26.417	26.417	0.000		9780	36.949	34.443	-2.506
8650	28.671	28.671	0.000		9924	37.390	34.125	-3.265
8690	29.312	29.287	-0.025		10028	37.029	34.653	-2.376
8720	29.820	29.439	-0.381		10148	36.893	33.212	-3.681
8750	29.832	29.522	-0.310		10242	36.664	35.074	-1.590
8780	29.405	29.357	-0.048		10328	36.659	35.009	-1.650
8810	31.942	30.762	-1.180		10404	36.751	35.000	-1.751
8840	31.898	30.609	-1.289		10500	38.080	37.419	-0.661
8870	31.916	30.857	-1.059		10600	38.343	37.256	-1.087
8900	32.657	31.979	-0.678		10720	38.940	38.310	-0.630
8950	35.432	33.816	-1.616		10740	39.108	38.468	-0.640
8980	35.120	33.550	-1.570		10760	39.156	38.506	-0.650
9000	35.533	33.712	-1.821		10780	39.363	38.775	-0.588
9011	35.307	33.256	-2.051		10850	39.063	38.415	-0.648
9050	35.128	33.635	-1.493		10900	39.036	38.417	-0.619
9090	34.537	34.181	-0.356		10950	39.187	38.621	-0.566
9130	34.492	34.328	-0.164		11045	39.286	38.738	-0.548
9171	34.515	34.224	-0.291		11145	38.220	37.242	-0.978
9253	35.151	34.155	-0.996		11785	40.607	40.425	-0.182
9338	36.453	34.185	-2.268		11995	40.333	40.129	-0.204

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Table 6.2 HEC-RAS Water Surface Elevation Comparison of Floodway Simulations							
HEC-RAS Cross Section	Existing Conditions	Proposed Conditions	WS Increase Caused by Proposed Conditions	HEC-RAS Cross Section	Existing Conditions	Proposed Conditions	WS Increase Caused by Proposed Conditions
7581	27.500	27.500	0.000	9419	37.183	35.003	-2.180
8400	27.693	27.693	0.000	9471	37.061	35.368	-1.693
8450	28.427	28.427	0.000	9558	37.370	35.559	-1.811
8500	29.261	29.261	0.000	9626	37.592	35.472	-2.120
8550	28.754	28.754	0.000	9682	37.571	35.433	-2.138
8600	27.414	27.414	0.000	9780	37.465	35.245	-2.220
8650	29.570	29.570	0.000	9924	37.819	34.941	-2.878
8690	29.889	29.842	-0.047	10028	37.364	35.285	-2.079
8720	30.043	29.742	-0.301	10148	37.166	34.069	-3.097
8750	30.180	29.755	-0.425	10242	36.923	35.384	-1.539
8780	29.840	29.689	-0.151	10328	36.993	35.265	-1.728
8810	32.304	31.540	-0.764	10404	37.218	35.578	-1.640
8840	32.276	31.432	-0.844	10500	38.107	37.268	-0.839
8870	32.395	31.797	-0.598	10600	38.934	37.465	-1.469
8900	33.101	32.792	-0.309	10720	39.517	38.520	-0.997
8950	36.093	34.669	-1.424	10740	39.672	38.672	-1.000
8980	35.811	34.423	-1.388	10760	39.710	38.702	-1.008
9000	36.179	34.566	-1.613	10780	39.981	39.083	-0.898
9011	35.888	34.149	-1.739	10850	39.729	38.764	-0.965
9050	35.709	34.376	-1.333	10900	39.660	38.718	-0.942
9090	35.147	34.922	-0.225	10950	39.782	38.901	-0.881
9130	35.168	35.087	-0.081	11045	39.819	38.937	-0.882
9171	35.221	34.994	-0.227	11145	38.955	37.625	-1.330
9253	35.828	34.955	-0.873	11785	41.119	40.680	-0.439
9338	36.993	34.999	-1.994	11995	40.874	40.393	-0.481

6.3 NFIP Existing Floodway Map

A copy of the existing FEMA DFIRM Floodway Map for the project area is attached as Figure 6.4.

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6.4 Existing Topographic Map with Cross Section Locations

A current topographic map of the project site is included with the HEC-RAS cross section locations shown on Figure 5.1. The proposed project configuration is also illustrated in Figure 6.1.

6.5 Construction Plans

Construction plans for the PHSPF will be submitted by PB as a separate document.

6.6 Compensatory Volume Requirement

When evaluating the impact a structure may have on flood levels there are two issues that need to be addressed – loss of the ability for the stream to convey flow downstream and loss of floodplain storage. In this project, conveyance would be reduced by the obstructions created by the columns and fills. However, the loss of conveyance will be mitigated by excavating and restoring the floodplain as illustrated in Figure 6.1. The HEC-RAS modeling described in Section 6.2 appropriately evaluated this conveyance issue and demonstrated that there will be no net loss of conveyance.

Loss of storage volume in the floodway is not addressed by the steady-state HEC-RAS modeling that was completed for this investigation. When obstructions are placed within a floodway one must demonstrate that the proposed project will not reduce existing storage volume. If storage is lost, flood levels downstream may rise. The easiest way to prevent downstream impacts is to remove sufficient fill to compensate for the obstruction. This has been carefully considered in the design of the PHSPF facility and the proposed excavation volume will significantly exceed the volume lost due to the columns and fill that will be placed when the facility is constructed. Proposed facility features will displace approximately 13200 cu yards of water within the floodway during a 100-year flood, while the excavation will remove approximately 79700 cu yards of existing material inside the floodway. This more than compensates for the storage volume that will be eliminated by the proposed facility features.

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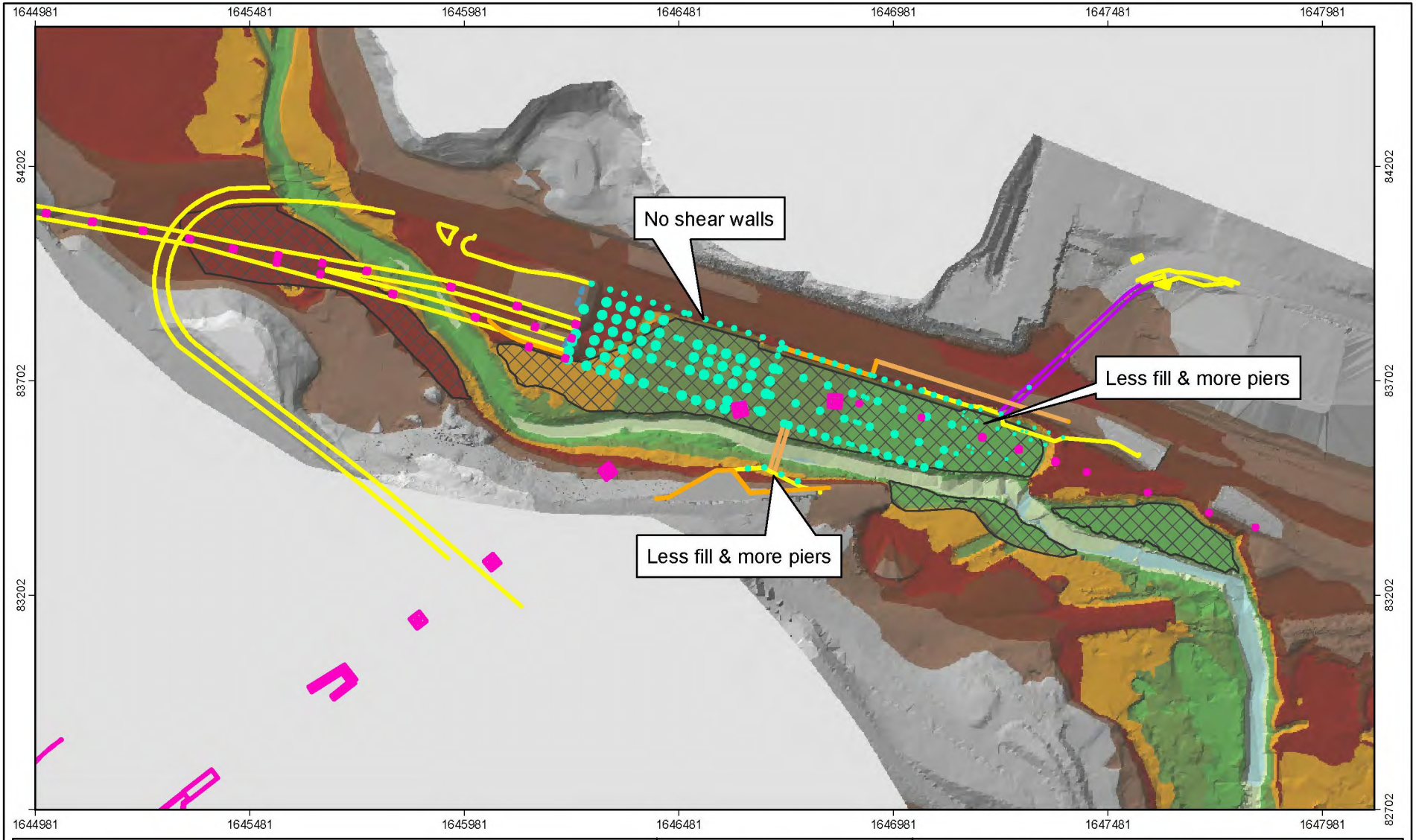

Legend

Restore Floodplain Bench

Proposed Ground DEM (ft - HI - local)

80 - 90 | 45 - 80 | 35 - 45 | 30 - 35 | 25 - 30 | 20 - 25 | 15 - 20 | 10 - 15 | 5 - 10 | -0.988 - 5

Oahu Reference Map

Honolulu High Capacity Transit Corridor

Map of Project Mitigation Features

Scale - 1:4,000

200 100 0 200 400 Feet

coord. syst.: HI State Plane 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50585	19-Nov-2009

Figure 6.1

Honolulu High Capacity Transit Corridor		
HEC-RAS Model Water Surface Profiles		
100-year Floodplain - Existing and Proposed Conditions		
northwest hydraulic consultants	project no. 50585	19-Nov-2009

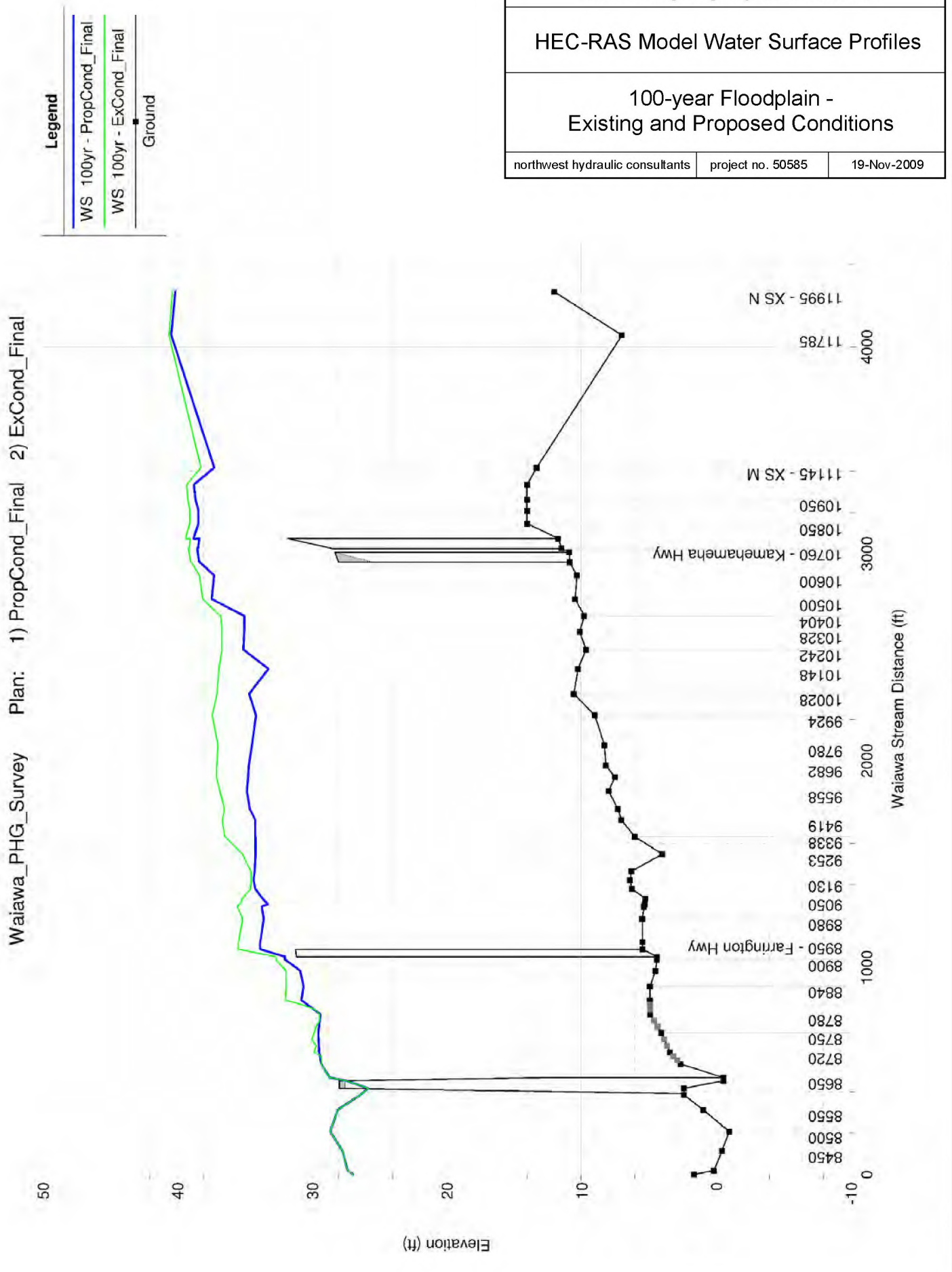

Figure 6.2

AR00041565

Honolulu High Capacity Transit Corridor		
HEC-RAS Model Water Surface Profiles		
Floodway Run - Existing and Proposed Conditions		
northwest hydraulic consultants	project no. 50585	19-Nov-2009

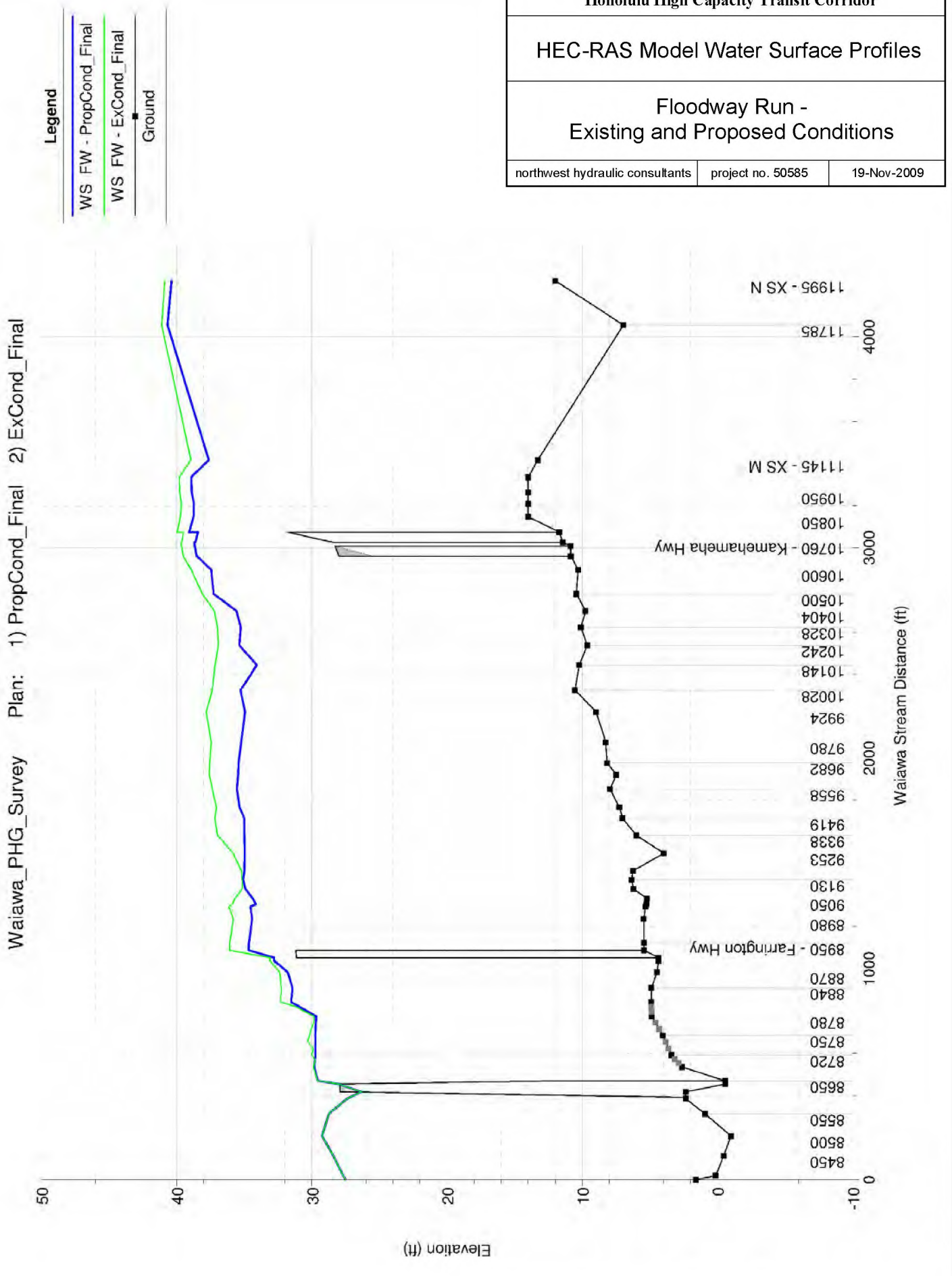

Figure 6.3

AR00041566

Honolulu High Capacity Transit Cooridor

Effective FEMA DFIRM Map

FIRM Panel 239 of 395 - Map # 15003C0239F
Map Revised September 30, 2004

Scale - 1:7,000

coord. syst.: HI State Plane 3	horz. datum: NAD 83	horz. units: feet	
northwest hydraulic consultants	project no. 50585	19-Nov-2009	

Oahu Reference Map

LEGEND

SPECIAL FLOOD HAZARD AREAS SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD

The 1% annual chance flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V, and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.

ZONE A	No Base Flood Elevations determined.
ZONE AE	Base Flood Elevations determined.
ZONE AH	Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.
ZONE AO	Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
ZONE AR	Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
ZONE A99	Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.
ZONE V	Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.
ZONE VE	Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE

Figure 6.4

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7. REVISED FLOOD HAZARD MAPS

The modifications to the floodplain within the Pearl Highlands project site will be so significant that it is likely that FEMA will eventually want to update the existing FIRM to reflect "with project" base flood elevations (BFEs), refined floodplain and floodway boundaries, and flood profiles. The "with project" floodplain and floodway profiles have already been presented in Tables 6.1 and 6.2 and Figures 6.2 and 6.3. The new BFEs, floodplain and floodway boundaries are presented in Figures 7.1 and 7.2 for existing and "with project" conditions, respectively.

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Legend

38 Base Flood Elevation (ft - HI local)

Floodway

100-Year Floodplain Inundation

500-Year Floodplain Inundation

Oahu Reference Map

Honolulu High Capacity Transit Corridor

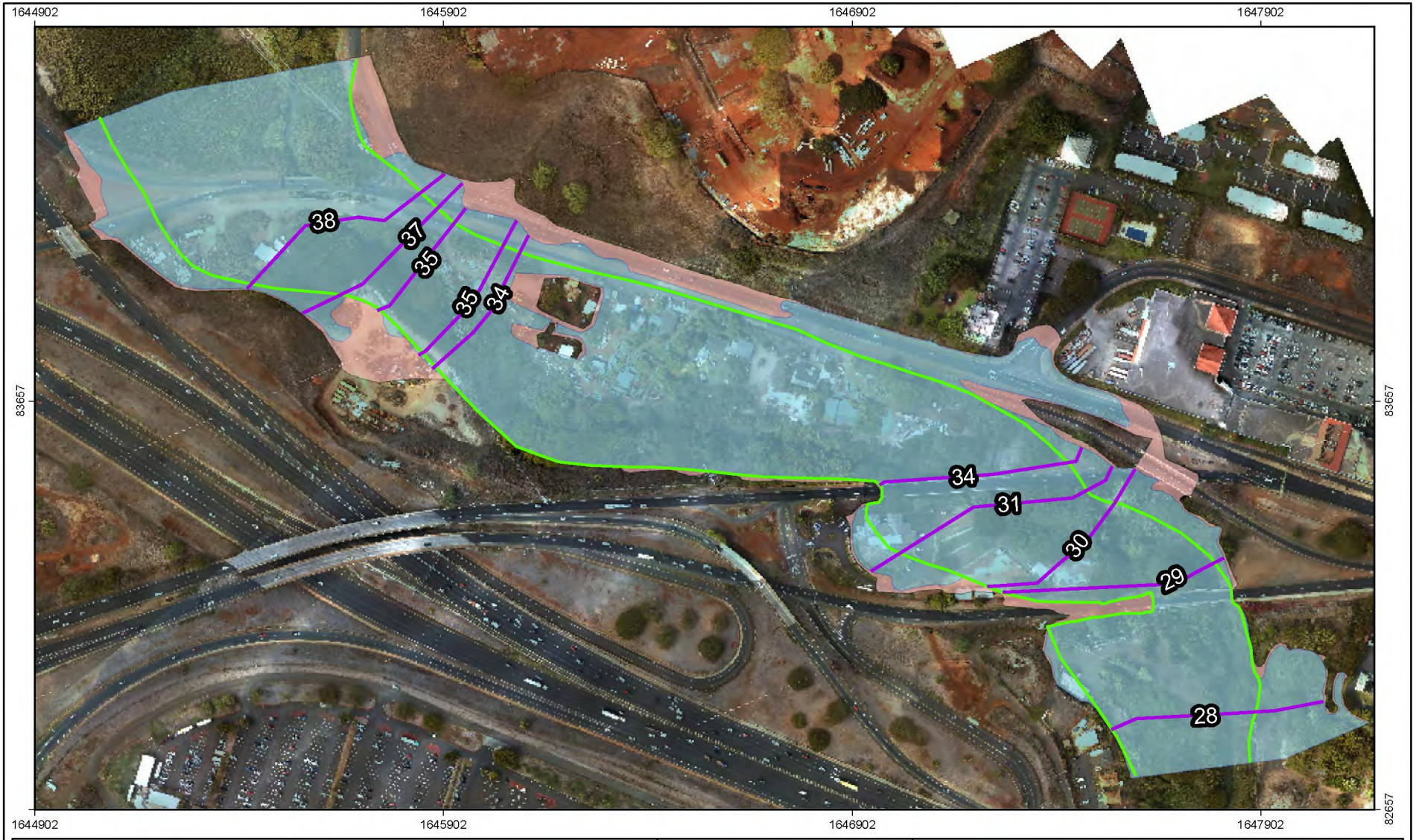
Existing Conditions Workmap

Scale -1:4,200

200 100 0 200 400 Feet

coord. syst.: HI State Plane 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	22-Apr-2010

Figure 7.1



Legend

38 Base Flood Elevation (ft - HI local)

Floodway

100-Year Floodplain Inundation

500-Year Floodplain Inundation

Oahu Reference Map

Honolulu High Capacity Transit Corridor

Proposed Conditions Workmap

Scale -1:4,200

200 100 0 200 400 Feet

coord. syst.: HI State Plane 3	horz. datum: NAD 83	horz. units: feet
northwest hydraulic consultants	project no. 50530	22-Apr-2010

Figure 7.2

8. CONCLUSION

The proposed PHSPF is in part located within the floodway of Waiawa Stream, a system that generates the largest peak discharges on the island of Oahu. NHC, working with the HHCTCP design team, together have developed a design for the facility that not only successfully mitigates impacts the project may have on 100-year flood levels, but also minimizes the risk imposed on the facility by the stream. This document provides the data required to demonstrate that the FEMA and DPP "No-rise" requirement has been satisfied.

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NHC (2010), "Advanced Floodplain Evaluations for Pearl Highlands Station and Park-and Ride Facility – Waiawa Stream", report prepared for Parsons Brinckerhoff, April 30, 2010.

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APPENDIX A

Photos

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Photo 1. Viewing downstream to the Kamehameha Highway bridges.

Photo 2. Debris on Kamehameha Highway bridge piers and deck.

Photo 3. Viewing the across the channel at the left bank fill in Banana Patch reach.

Photo 4. Viewing the across the channel at the left bank fill in Banana Patch reach.

Photo 5. Viewing vegetated "natural" area of left bank floodplain upstream of west-bound Farrington highway bridge; fill in background.

Photo 6. Viewing left bank fill and old cars used as bank protection.

Photo 7. Rocks have been placed along the banks in many locations in the project reach.

Photo 8. Viewing upstream at the Waiawa Stream channel in the Banana Patch reach.

Photo 9. Viewing downstream at the Waiawa Stream channel in the Banana Patch reach.

Photo 10. Viewing downstream to the west-bound Farrington Highway bridge.

Photo 11. Viewing upstream along Waiawa Stream from underneath the west-bound Farrington Highway bridge; debris caught on pier in foreground.

Photo 12. Viewing downstream from underneath the west-bound Farrington Highway bridge.

Photo 13. Viewing towards the left bank under the west-bound Farrington Highway bridge; note the deposition bars and scouring of the banks.

Photo 14. Local left bank erosion upstream of west-bound Farrington Highway bridge.

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Photo 15. Viewing upstream to the west-bound Farrington highway bridge; note the fill downstream of the bridge (left in photo) blocking all but two bridge spans.

Photo 16. Viewing the fill on the right bank under and downstream of the west-bound Farrington Highway bridge.

Photo 17. Viewing upstream from the east-bound Farrington Highway bridge. The USGS gage was located at the concrete weir in the foreground.

Photo 18. Viewing the upstream right bank of the east-bound Farrington Highway bridge.

Photo 19. Viewing downstream along Waiawa Stream towards Interstate H-1 from underneath the east-bound Farrington Highway bridge.

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APPENDIX B

HEC-SSP Output from Re-analysis of Waiawa Stream Record

AR00041584

Appendix B. HEC-SSP Output from Re-Analysis of Waiawa Stream Record

```
------------------------------
Bulletin 17B Frequency Analysis
    06 Oct 2008   08:43 AM
------------------------------


--- Input Data ---

Analysis Name: test
Description:

Data Set Name: waiawa
DSS File Name: C:\ssptest\test\test.dss
DSS Pathname: ///FLOW-PEAK/01jan1900/IR-CENTURY//

Report File Name: C:\ssptest\test\Bulletin17bResults\test\test.rpt
XML File Name: C:\ssptest\test\Bulletin17bResults\test\test.xml

Start Date:
End Date:

Skew Option: Use Weighted Skew
Regional Skew: -0.05
Regional Skew MSE: 0.302

Plotting Position Type: Median

Upper Confidence Level: 0.05
Lower Confidence Level: 0.95

Display ordinate values using 0 digits in fraction part of value

--- End of Input Data ---



----------------------
<< Low Outlier Test >>
----------------------
 Based on 52 events, 10 percent outlier test value K(N) = 2.783

       0 low outlier(s) identified below test value of 1,063.85



----------------------
<< High Outlier Test >>
----------------------
 Based on 52 events, 10 percent outlier test value K(N) = 2.783

     0 high outlier(s) identified above test value of 61,156.18
```

--- Final Results ---

<< Plotting Positions >>
waiawa

Events Analyzed			Ordered Events			
Day	Mon	Year	FLOW cfs	Rank	Water Year	FLOW cfs	Median Plot Pos
19	Nov	1952	1,790	1	1982	27,950	1.34
01	Jan	1954	1,500	2	1991	27,600	3.24
28	Nov	1954	16,900	3	1968	23,400	5.15
25	Feb	1956	13,500	4	1980	19,900	7.06
21	Jan	1957	6,080	5	1955	16,900	8.97
05	Mar	1958	7,810	6	1988	16,200	10.88
23	Oct	1958	7,320	7	1966	15,800	12.79
14	May	1960	11,500	8	1963	15,500	14.69
13	Feb	1961	3,220	9	1970	15,400	16.60
12	Mar	1962	2,240	10	1994	14,270	18.51
14	May	1963	15,500	11	1992	14,200	20.42
22	Mar	1964	2,690	12	1996	14,000	22.33
02	May	1965	14,000	13	1965	14,000	24.24
14	Nov	1965	15,800	14	1974	13,700	26.15
11	Oct	1966	11,000	15	1956	13,500	28.05
05	Jan	1968	23,400	16	1969	13,300	29.96
03	Jan	1969	13,300	17	1989	12,700	31.87
25	Jul	1970	15,400	18	1972	12,400	33.78
26	Nov	1970	9,080	19	2000	11,600	35.69
15	Apr	1972	12,400	20	1987	11,600	37.60
09	Jul	1973	2,900	21	1960	11,500	39.50
19	Apr	1974	13,700	22	1979	11,000	41.41
21	Nov	1974	8,760	23	1967	11,000	43.32
27	Nov	1975	8,300	24	1997	10,300	45.23
09	Jun	1977	5,640	25	2004	10,000	47.14
28	Jun	1978	4,780	26	1990	9,720	49.05
10	Feb	1979	11,000	27	2002	9,680	50.95
18	Mar	1980	19,900	28	1993	9,180	52.86
04	Aug	1981	7,380	29	1971	9,080	54.77
28	Oct	1981	27,950	30	1986	9,000	56.68
28	Oct	1982	5,170	31	1975	8,760	58.59
19	Apr	1984	3,950	32	1976	8,300	60.50
27	Nov	1984	4,130	33	1958	7,810	62.40
20	Oct	1985	9,000	34	1981	7,380	64.31
12	Jun	1987	11,600	35	1959	7,320	66.22
31	Dec	1987	16,200	36	1957	6,080	68.13
21	Jul	1989	12,700	37	1977	5,640	70.04
03	Oct	1989	9,720	38	1983	5,170	71.95
21	Mar	1991	27,600	39	1978	4,780	73.85
03	Sep	1992	14,200	40	2003	4,220	75.76
26	Dec	1992	9,180	41	1985	4,130	77.67
18	Sep	1994	14,270	42	1995	4,050	79.58
23	Aug	1995	4,050	43	1984	3,950	81.49
25	Jan	1996	14,000	44	1961	3,220	83.40
03	Jan	1997	10,300	45	1999	3,080	85.31
03	Oct	1997	2,200	46	1973	2,900	87.21
07	Jan	1999	3,080	47	1964	2,690	89.12
02	Dec	1999	11,600	48	2001	2,430	91.03
29	Oct	2000	2,430	49	1962	2,240	92.94
06	May	2002	9,680	50	1998	2,200	94.85
10	Apr	2003	4,220	51	1953	1,790	96.76
04	Aug	2004	10,000	52	1954	1,500	98.66

<< Skew Weighting >>

Based on 52 events, mean-square error of station skew = 0.142
Mean-square error of regional skew = 0.302

<< Frequency Curve >>
waiawa

Computed Curve FLOW, cfs	Expected Probability	Percent Chance Exceedance	Confidence Limits 0.05 FLOW, cfs	0.95
45,838	---	0.2	67,245	34,349
39,745	---	0.5	56,905	30,280
35,128	---	1.0	49,267	27,137
30,507	---	2.0	41,813	23,931
24,389	---	5.0	32,280	19,572
19,740	---	10.0	25,338	16,148
15,032	---	20.0	18,635	12,544
8,477	---	50.0	10,046	7,172
4,455	---	80.0	5,330	3,606
3,092	---	90.0	3,800	2,387
2,252	---	95.0	2,854	1,657
1,195	---	99.0	1,628	791

<< Systematic Statistics >>
waiawa

Log Transform: FLOW, cfs		Number of Events	
Mean	3.9067	Historic Events	0
Standard Dev	0.3161	High Outliers	0
Station Skew	-0.5800	Low Outliers	0
Regional Skew	-0.0500	Zero Events	0
Weighted Skew	-0.4107	Missing Events	0
Adopted Skew	-0.4107	Systematic Events	52

APPENDIX C

FEMA Certification of a No-Rise Determination

AR00041588

Federal Emergency Management Agency

Washington, D.C. 20472

CERTIFICATION OF A "NO-RISE" DETERMINATION

FOR A PROPOSED FLOODWAY DEVELOPMENT

City and County of Honolulu, Hawaii
Community Name

Pearl Highlands Station and Parking Facility
Development Name

Lot/Property Designation

Property Owner

I hereby certify that the proposed remedial measures, in combination with the property development designated above, will result in no loss of flow conveyance during the occurrence of the 1 percent annual chance of exceedence (100-year flood) discharge.

I further certify that the data submitted herewith in support of this request are accurate to the best of my knowledge, that the analyses have been performed correctly and in accordance with sound engineering practice, and that the proposed structural works are designed in accordance with sound engineering practice.

April 21, 2010
Date

Robert C. MacArthur
Registered Professional Engineer

Seal

AR00041589

APPENDIX D

FEMA Certification Requirements for Simple Floodway Encroachments

AR00041590

Federal Emergency Management Agency

Washington, D.C. 20472

CERTIFICATION REQUIREMENTS FOR SIMPLE FLOODWAY ENCROACHMENTS

Introduction

This document is intended to provide guidance to local floodplain administrators in evaluating requests for the placement of fill at a single location, a building, or another simple encroachment within an adopted regulatory floodway. The procedure contained in this document is not intended to evaluate complex encroachments, such as extensive fills, multiple structures, bridges, or levees, where flow expansion and contraction losses may be significant. In such cases, full hydraulic analyses by computer backwater models should be employed. The minimum floodplain management requirements for communities participating in the National Flood Insurance Program (NFIP) in which a regulatory floodway has been designated prohibit any floodway development that would result in an increase in flood levels within the community during the occurrence of the base (100-year) flood discharge. This requirement is outlined in Paragraph 60.3(d)(3) of the NFIP regulations.

Because floodway development is contradictory to the tenets of sound floodplain management, such development is discouraged by the Federal Emergency Management Agency (FEMA). Therefore, these certification requirements assume that all practical alternatives to floodway development have been investigated thoroughly and have been deemed not feasible.

In accordance with the NFIP regulations, it is ultimately the responsibility of each community participating in the NFIP to prohibit floodway development that would result in increases in 100-year flood levels. Communities must make determinations of this type and maintain backup calculations and certifications in their files for review by FEMA personnel upon request.

This document also does not address the many cases and situations requiring the actual revision of the floodway via redelineation of the floodway boundary, the criteria for which are presented in Section 65.7 of the NFIP regulations. A FEMA document, entitled "Conditions and Criteria for Floodway Revisions," and dated August 27, 1984, addresses these issues.

Definition of "No-Rise"

It is important that the concept of "no rise" be clarified and understood. The actual wording of Paragraph 60.3(d)(3) of the NFIP regulations is that the community shall "prohibit ... any increase in flood levels during the occurrence of the base flood discharge." If an adopted regulatory floodway has been computed and is displayed on the effective NFIP map, all areas within the floodway are considered to be effective in conveying the 100-year flood discharge. Therefore, no obstruction, regardless of size, can be

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placed within the floodway without obstructing flow and causing some increase in the base flood elevation (BFE). Such increases may be localized and may seem insignificant; if modeled, they may yield changes on the order of hundredths or thousandths of a foot.

There has been a tendency to misinterpret the "no-rise" criterion to include rounding allowances and also to conclude that an increase in computed water-surface elevation of 0.01 foot or less, as determined by a backwater computer model, is sufficient evidence to support the acceptability of development in a floodway.

Although the backwater computer model output may show little change in water-surface elevation, closer examination will, in all likelihood, reveal changes in other variables (e.g., topwidth, flow area, velocity). These changes can be translated into increases in water-surface elevation that may not be considered significant by themselves. However, the long-term cumulative effects of such increases will eventually result in significant changes. Therefore, no development in the floodway should occur without proper compensation for the lost conveyance. This is the intent of Paragraph 60.3(d)(3) of the NFIP regulations.

It is FEMA's position that this regulation is to be interpreted exactly, and strictly, as written; that is, "no" rise above the BFE will be permitted. Therefore, nothing that offers any resistance to the flow of floodwaters may be placed within a regulatory floodway unless compensatory action is taken to restore the lost conveyance.

Loss of Conveyance

In the case of a simple floodway encroachment, as discussed previously, a "no-rise" determination can usually be made based on consideration of conveyance only. In such a situation, it is the difference in the conveyance before and after encroachment, or the aforementioned loss of conveyance, that must be addressed if the effects of development are to be compared against the "no-rise" criterion. The computation of loss of conveyance is most appropriately accomplished on a micro scale by isolating a portion of a cross section, separate from the backwater computer model, and performing hand computations. Examples of typical hand computations for proposed fill and bridge pier construction are attached for reference. However, it is appropriate to incorporate one or more new cross sections at the site of the proposed construction that reflect existing conditions into the unencroached and encroached backwater computer models. This is done to establish the base flood conditions at that location, which are to be used in the hand computations. The formula used to determine conveyance (K) is

$$K = 1.49/n\ AR^{2/3}$$

where n = Manning's roughness coefficient
A = Flow area
R = Hydraulic radius

The loss of conveyance is computed using the "n" value and hydraulic radius at the site of the encroachment as applied in the computer model.

Compensation

Once a determination has been made as to the amount of conveyance lost as a result of the proposed development, the designer or engineer is required to adequately compensate for this loss. This compensation is accomplished by including some means or measures within the proposed floodway development for providing an increase in effective conveyance, at some point on the cross section, equal to or greater than that lost. Equal area exchanges are only valid if the "n" value and hydraulic radius remain unchanged between the encroachment site and the compensation site. It is also important that the flow area provided be truly effective; that is, open to inflow and outflow and not just an isolated low spot or depression. This increased effective conveyance could be computed by hand in a manner similar to that used to compute the loss of conveyance.

The means or measures used to provide this effective conveyance (e.g., excavation, roughness coefficient reduction) would be at the discretion of the designer or engineer but must be approved by the community. Where these means and measures require some form of maintenance, the community must assume ultimate responsibility for their maintenance.

Data Requirements

The items identified below are necessary to document and demonstrate compliance with the "no-rise" criterion for simple floodway encroachments.

1. Hydraulic backwater models of the 100-year flood and floodway water-surface profiles for the following:

   a. Duplicate of the effective Flood Insurance Study (FIS) model.

   b. Existing-conditions (effective FIS) model modified to include cross sections through the project site. Cross sections must reflect conditions prior to construction of the project.

   c. Post-project conditions model. This model must include cross sections through the project site reflecting floodplain conditions after construction of the project. The 100-year flood (without floodway) and floodway elevations for this model must not be greater than those in the existing-conditions model described at letter "b" above. This hydraulic backwater model is necessary to ensure that any changes in transition losses, which are based on velocity heads rather than conveyance, do not cause increases in water-surface elevations. Also, a hydraulic backwater model provides a means of evaluating effective flow areas upstream and downstream of the encroachment and compensation sites.

2. A copy of the appropriate NFIP map showing the existing floodway and indicating the project area

3. Topographic mapping of the entire project area indicating the locations of all cross sections used in the modified hydraulic model and a plan view of all project elements

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4. Construction plans, certified by a registered professional engineer, for all project elements, including those measures employed to provide additional effective conveyance

5. The following information, to be obtained by hand computation using the cross section and 100-year encroached hydraulic data in the modified existing-conditions computer model output provided under Item 1b:

   a. Calculation of the reduction in conveyance (K) caused by the proposed obstruction, assuming no change in floodway water-surface elevation, and using the "n" value appropriate for the site of the proposed obstruction

   b. Calculation of the increase in conveyance (K) obtained by the proposed offsetting measure, using the "n" value appropriate for the site of this measure

   c. Comparison showing that the conveyance increase computed in 5b equals or exceeds the loss computed in 5a

6. Evidence that the increase in effective conveyance provided for in Item 5b will be maintained perpetually. This should be in the form of a self-maintaining measure or certified maintenance plans for the measure provided.

7. An executed copy of the attached certification statement signed and sealed by a registered professional engineer

A copy of the hydraulic computer model for the effective FIS for the specified stream and community may be obtained by written request to the following address:

Federal Emergency Management Agency
Federal Insurance Administration
Risk Studies Division
500 C Street, SW
Washington, D.C. 20472

AR00041594

NO RISE ANALYSIS FOR WAIPAHU TRANSIT CENTER STATION AND HHCTCP GUIDEWAY COLUMNS – WAIKELE, KAPAKAHI AND WAILANI STREAMS

DRAFT

Prepared for:

Parsons Brinckerhoff Inc.
1001 Bishop Street, Suite 2400
Honolulu, HI 96813
808-531-7094

Prepared by:

Northwest Hydraulic Consultants Inc.
3950 Industrial Boulevard, Suite 100C
West Sacramento, CA 95691
916-371-7400

April 30, 2010

No Rise Analysis for Waipahu Transit Center Station and HHCTCP Guideway Columns – Waikele, Kapakahi and Wailani Streams

Draft - April 30, 2010

EXECUTIVE SUMMARY

The Honolulu High-Capacity Transit Corridor Project (HHCTCP) is an elevated rail line that will provide rapid public transportation service in the City and County of Honolulu on the island of Oahu, Hawaii. Northwest Hydraulic Consultants (NHC) was retained to perform hydraulic analyses to address flood-related design considerations and floodplain regulation compliance for the proposed Waipahu Transit Center Station Facility, which is part of the greater HHCTCP.

The elevated guideway located along the Waipahu portion of Segment C will be placed on concrete support columns located in the center median along Farrington Highway through the Town of Waipahu. Also located in Waipahu are two proposed entrance buildings for the Waipahu Transit Centre Station, one on the north (mauka) side of Farrington Highway and one on the south (makai) side, located between Waipahu Depot Street and Mokuola Street. Many of the support columns and both entrance buildings are located in a mapped Federal Emergency Management Agency (FEMA) floodway zone. FEMA and the City and County of Honolulu Department of Planning and Permitting (DPP) require that all segments of the HHCTCP comply with their "no rise" policies. On behalf of Lyon Associates, Inc (LAI), subconsultant to Parsons Brinckerhoff (PB), the General Engineering Consultant (GEC) for the HHCTCP, Northwest Hydraulic Consultants (NHC) conducted an hydraulic assessment of the proposed HHCTCP "development" features within the FEMA floodway zone. NHC along with engineering staff from PB and LAI conducted a thorough site inspection of the study area. NHC then evaluated the hydrology for the Waikele, Kapakahi and Wailani streams, performed hydraulic analyses and worked closely with the HHCTCP design staff to develop a project design alternative for the Waipahu Transit Center Station facilities and elevated guideway that meets DPP and FEMA "no rise" requirements.

NHC reviewed FEMA's effective model to assess possible effects that proposed project features could have on water levels in Waipahu, as per FEMA guidelines. After considerable effort and testing, it was determined that the proposed structures for the Waipahu Station facilities and the guideway constitute simple encroachments and that a "no rise" determination could be made based on the consideration of floodplain storage volume and conveyance only instead of using FEMA's effective model. NHC's preliminary findings were discussed with staff from DPP and the City and County's Department of Transportation Services, Rapid Transit Division (RTD) during a meeting held on September 23, 2009 in Honolulu. During the meeting it was agreed that while detailed hydraulic models would be used to understand local flow conditions and to refine designs for specific project features, volume and conveyance analyses would be performed using an alternative method to show how proposed project features may affect FEMA's 100-year water

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levels and floodplain storage volumes. The goals of these analyses were to demonstrate that the proposed structural designs together with proposed mitigation measures will result in no impacts to existing floodplain storage volume or conveyance characteristics and therefore, will not create a rise in water level during a 100-year flood.

This "no rise" assessment report is explicitly written for DPP and it describes the simple volume and conveyance analyses that were performed and provides information that explains how HHCTCP designs for the Waipahu Transit Center Station and guideway will meet DPP's and FEMA's "no rise" and "no net floodplain storage volume loss" requirements. This report describes (1) analyses performed to determine volume displacement due to project features, (2) methods and analyses that show how displaced volumes will be compensated for, and (3) analyses regarding possible changes in flow conveyance area due to project features and how those changes will be compensated for.

Results from the volume displacement and conveyance calculations show that the preferred project alternative (described as Alternative 3 in NHC 2010) for the two entrance buildings will not obstruct flow or change present day conveyance conditions at either entrance building site. This preferred alternative includes an elevated (above the 100-year flood level) plaza and a new north entrance building that fully fills the gap between the existing adjacent buildings. The preferred alternative also includes a new south entrance building to be located in the same approximate location with the same maximum external ground floor dimensions (approximately 20 by 40 feet) as the existing building that it replaces. Results also show that all of the storage volume losses caused by support columns and buildings (3,284 cubic feet) and the very small amount of conveyance loss (267 square feet) caused by the 25 support columns located in the floodway are easily compensated for with simple modifications to a portion of the center median along Farrington Highway. Therefore, the preferred project alternative (Alternative 3) will cause no adverse effects to flood conveyance, FEMA's 100-year flood water levels or floodplain storage volumes.

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1. BACKGROUND

The Waipahu portion of Segment C of the elevated guideway for the proposed HHCTCP is planned to be placed on concrete support columns located in the center median along Farrington Highway through the Town of Waipahu. Also located in Waipahu are two proposed entrance buildings for the Waipahu Transit Center Station, one on the north (mauka) side of Farrington Highway and one on the south (makai) side, located between Waipahu Depot Street and Mokuola Street (see Figure 1). Most of the project features planned for Waipahu will be located in a mapped Federal Emergency Management Agency (FEMA) AE flood zone and the two entrance buildings and several guideway support columns will be located in a mapped FEMA floodway zone. FEMA and the City and County of Honolulu Department of Planning and Permitting (DPP) both require that any new "development" (man-made buildings, structures, paving, grading or filling) in a mapped FEMA flood zone or DPP Flood Hazard District "will not cause any increase in regulatory flood elevations, obstruct the regulatory flood or affect the capacity of the floodway." Northwest Hydraulic Consultants (NHC) conducted a hydraulic assessment of the proposed HHCTCP "development" features on behalf of Lyon Associates, Inc (LAI), subconsultant to Parsons Brinckerhoff (PB), the General Engineering Consultant (GEC) for the HHCTCP. Following is a summary of results and recommendations developed by NHC.

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Figure 1. Location of the Waipahu Transit Center Station entrance buildings and guideway support columns within FEMA's AE flood zone. Also shown are four transects used for conveyance area analyses.

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2. ANALYSES

PB provided NHC with detailed topographic data, architectural drawings for the support columns, guideway structures and entrance buildings that are planned for the Waipahu segment of the HHCTCP. Also obtained was a copy of FEMA's effective one-dimensional HEC-RAS model that was used to prepare the Flood Insurance Rate Map (FIRM) for Waipahu. NHC attempted to use FEMA's effective model to assess possible effects that proposed project features could have on water levels in Waipahu. After considerable effort and testing, it was determined that the structures proposed for the project constitute simple encroachments so FEMA's effective model would not be used for the 100-year flood "no rise" analyses. Therefore, an alternative method was applied to determine project impacts on conveyance, floodway storage volume, FEMA's 100-year water levels and to develop means for mitigating project impacts. The approach and results are described below.

2.1. Volume Displacement and Conveyance Calculations

As part of the permitting requirements for the HHCTCP in Waipahu, DPP asked NHC to perform a series of volume and conveyance analyses to show how proposed project features may affect local floodplain volume and conveyance conditions. The goal of these analyses is to demonstrate that the proposed structural designs, together with proposed mitigation measures, will result in no impacts to existing volume and conveyance characteristics and will not result in any water level rise during a regulatory 100-year flood. The following sections describe (1) analyses performed to determine volume displacement due to project features, (2) methods and analyses that show how displaced volumes will be compensated for, and (3) analyses regarding possible changes in conveyance area due to project features and how those changes will be compensated for.

2.2. Floodplain Volume Displaced by Project Features

NHC assisted PB to evaluate several alternative configurations for the two entrance buildings (north and south) and for the support columns that carry the elevated guideway (see report NHC 2010). A preferred alternative (project Alternative 3 in NHC 2010) that does not affect water levels was selected by the PB design team. This preferred alternative includes an elevated (above the 100-year flood level) plaza and north entrance building that fully fills the gap between the existing adjacent buildings. It also includes a new south entrance building in the same approximate location with a ground floor footprint that matches the maximum external dimensions (approximately 20 by 40 feet) of the existing building that it will replace. In addition to meeting the "no rise" criteria the project must not generate a net loss of floodplain storage. To address this requirement, NHC calculated the total volume that is displaced by proposed HHCTCP structures located within the mapped floodplain according to FEMA's Base Flood Elevations (BFEs) for the preferred design alternative and then developed a reliable means for mitigating for that loss of storage volume. Table 1 below summarizes the displaced volumes associated with each of the three significant components of the project located in the mapped FEMA flood zone.

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Table 1. Volume displaced by structures proposed under the preferred alternative for the HHCTCP station and elevated guideway corridor in Waipahu.

Structure	Total displaced volume (cubic feet)
25 guideway support columns located in mapped flood zone	1,385
North entrance building	249
South entrance building	1,650
Total displaced volume loss	**3,284**

Appendix A describes in detail the data and methods used to perform the volume displacement calculations. Table A-1 in Appendix A lists the individual footprint areas, water depths, displaced volumes, and blocked conveyance areas for each column and each proposed entrance building according to the most recent preferred design alternative (Alternative 3, NHC 2010). According to Table 1 in this section and Table A-1 in Appendix A, the total displaced volume from all of the 25 guideway support columns located within the mapped flood zone is 1,385 cubic feet. The displaced volumes for the north and south entrance buildings are 249 cubic feet and 1,650 cubic feet, respectively. The total combined volume displaced by all project features is 3,284 cubic feet. Therefore, it is necessary to replace at least 3,284 cubic feet of storage volume back to the floodplain in the vicinity of the project.

2.3. Volume Compensation

Several alternative means for mitigating for the loss of storage volume were considered and a simple and reliable method for mitigating for storage volume loss was selected. The recommended method requires only that a 700-foot long section of the center median along Farrington Highway be lowered between the curbs by about 6 inches during project construction. The volume provided by this very modest adjustment to this portion of the Farrington Highway median will more than compensate for the total combined volume displaced by all project features. Following is an explanation of how this was determined.

Section 2.2 discussed how volume losses were computed. Volume gains were also calculated using the same data and procedures that were used to compute the volume losses. NHC calculated the volume gain due to the removal of the existing building that resides at the proposed location of the south entrance building (see Figures 1 and 2 and Table 2 below). Removal of the north entrance building will not generate a volume gain because it is presently located outside of the mapped FEMA flood zone. The volume gained after removal of the existing building located south of Farrington Hwy is listed in the first row in Table 2. In addition to the volume gained from the removal of the existing building (820 cubic feet), NHC investigated other mitigation alternatives to compensate for the total displaced volume (3,284 cubic feet) listed in Table 1. The recommended method (concept) for mitigating for the total volume losses due to the project is to construct a very mild six inch deep swale within a 700-foot long segment of the center median of Farrington Highway between Waipahu Depot Street and Mokuola Street near the entrance buildings. A sketch showing the location of the modified swale is shown in Figure 3. The mild swale can be either grass lined as it is today, or surfaced with porous concrete or pavers placed on a gravel layer above the native soil to allow infiltration of

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rain and flood water that may enter the swale. The swale will have a trapezoidal section with only a 6-inch depth and 1:1 side slopes. It will extend for approximately 700 feet along a portion of the center median within the city block located in the immediate vicinity of the two station entrances (Figure 3). Appendix B includes the preliminary conceptual design details and the volume calculations for this preferred alternative. Details will be refined during final design. The estimated approximate volume gained with this alternative is 5,775 cubic feet which is listed in the second row in Table 2.

Figure 2. Vicinity map showing locations of two entrance buildings (blue), guideway support columns (blue), existing building to be removed (orange) and replaced by south entrance building, and center median along Farrington Highway (red).

Table 2. Volume gained by removal of existing building and installation of proposed mitigation alternative in center median along Farrington Highway.

Description	Volume gain (cubic feet)
Existing building (light orange outline in Figure 2)	820
Six inch depression along 700-linear feet of center median (red outlined mitigation areas shown in Figure 3)	5,775
Total volume gain	**6,595**

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Table 1 provides a listing of the total volume loss due to the project and Table 2 provides the total volume gains after the proposed mitigation alternative is installed. Comparison of the total losses and gains listed in Tables 1 and 2 shows that there will be a net gain in floodplain storage volume of approximately 3,311 cubic feet after the project is constructed and the modest 700 foot long mitigation swale is installed. Also, there are many more linear feet of center median available along Farrington Highway for this type of mitigation measure if additional mitigation is needed in the future. Therefore, the proposed mitigation alternative provides a conservative (more than required) volume to compensate for the volume displaced by all the project features at the Waipahu site. This conservative solution provides the PB design team with an opportunity to refine the preliminary mitigation design shown herein for the center median to a different size, shape or location(s) depending on what the RTD and DPP reviewers suggest. Design details for the center median will be prepared during the next design phase of the project.

Figure 3. Volume displacement mitigation areas (white lines) located in center median along Farrington Highway between Waipahu Depot Street and Mokuola Street in vicinity of project station.

2.4. Conveyance Area Computations

Conveyance Loss Created by Guideway Support Columns. Analyses were conducted to assess possible changes in flood conveyance area due to the construction of project features. There are 25 guideway support columns located within FEMA's mapped AE zone (see Figure 1). The additional support columns located outside of the mapped floodplain area will not affect flood conveyance and were not considered. NHC calculated the total conveyance area blocked by the submerged portions of the 25 guideway support columns. Appendix A summarizes the

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data and methods used to compute the submerged conveyance areas for each of the 25 columns. These values vary across the floodplain depending on the ground elevation and 100-year flood water levels at each column location. NHC used FEMA's BFEs and GIS mapping techniques to create a 100-year water surface over the affected floodplain along the corridor alignment. Water depths at the center of each column were computed by comparing the 100-year water level with the ground elevations at the center of each column. The conveyance area affected by the presence of each column was determined by multiplying the water depth at each column by the diameter of the column. The total conveyance area blocked by all of the 25 support columns is approximately 267 square feet. Construction of the proposed volume mitigation method described in Section 2.3 (Volume Compensation) and in Appendix B will provide an additional 350 square feet of conveyance area "gain" in the vicinity of the project. Therefore, by installing the 6-inch lowered center median along Farrington Highway, it will more than mitigate for lost floodplain storage created by all project features, and it will also provide sufficient gain in conveyance area to mitigate for the 267 square feet of conveyance lost due to the guideway support columns. Design details for the center median will be prepared during the next design phase of the project.

Conveyance Loss Created by Entrance Buildings. Analyses were also conducted to assess possible changes in flood conveyance area due to construction of the two entrance buildings. Figure 1 shows the Waipahu project corridor alignment, locations of the guideway support columns, and location of the two entrance buildings (red). Also shown in Figure 1 is the extent of the FEMA AE flood zone (shaded area) and the locations of the four transects used to determine if the locations and configurations of the entrance buildings (as proposed by the new preferred entrance building alternative) will cause any loss in flood conveyance or rise in 100-year water levels.

Figure 4 (To simplify the text Figures 4 – 11 are located at the end of this section) shows Transect A which is a south viewing elevation transect that shows the existing ground profile and blockages created by existing buildings located along the transect shown in Figure 1. The proposed north entrance building, outlined by the red dashed line, will replace an existing building in the same location as viewed from Hikimoe Street (Figure 5a) and from Farrington Highway (Figure 5b). Figure 1 clearly shows that the northern one third of the proposed north building is located outside of the mapped FEMA AE zone. Also, Figures 4, 5a and 5b show that under existing conditions today there is no opportunity for flow during the 100-year flood to move (be conveyed) through the site located between the adjacent existing buildings. PB's preferred alternative will remove the existing building shown in Figures 5a and 5b and will replace it with a new north entrance building and narrow plaza on that site. The elevation of the first floor in the north entrance building as well as the narrow plaza west of the entrance building will be built on fill so all finished floor and plaza elevations are above FEMA's BFE throughout that site. Therefore, no flow can move through the site today and the preferred alternative will not change that condition, so there is no effect on 100-year flood conveyance caused by the proposed north building. Sections 2.2 and 2.3 discuss the volume that will be displaced by the north entrance building and how that displaced volume will be replaced through mitigation measures.

Figure 6 shows Transect C, which is a south viewing elevation transect that shows the existing

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ground profile and blockages created by existing buildings located along the transect shown in Figure 1. The proposed south entrance building, outlined by the red dashed line, will replace an existing building in the same approximate location as viewed from Farrington Highway (see Figure 7). Figure 1 shows that this site is located in a mapped FEMA AE zone. PB's preferred alternative will remove the existing building shown in Figure 6 and in the photo shown in Figure 7, and replace it with a new south entrance building that will be constructed on the same approximate location and will fit into the rectangular area (approximately 20 ft by 40 ft) outlining the maximum external dimensions of the existing building (see Figure 2). Therefore, the new south entrance building will not change the flow conveyance for flows moving in the southerly direction. Also, shown in Figure 7 is the large apartment building immediately behind (to the south of) the existing building. The large apartment building presents an even larger blockage to southerly flow than does the small existing structure due to its close proximity and much larger footprint.

Figure 8 shows Transect D which is an east viewing elevation transect that shows the existing ground profile and blockages created by existing buildings located along the transect shown in Figure 1. The proposed south entrance building, outlined by the red dashed line, will replace the existing building in the same location (see Figure 9). The new south entrance building will be constructed at the same approximate location and will fit into the rectangular area (approximately 20 ft by 40 ft) outlining the maximum external dimensions of the existing building (see Figure 2). Therefore, the new south entrance building will not change flow conveyance for flows moving in a southerly or easterly direction.

Figure 10 shows Transect B which is a south viewing elevation transect that shows the existing ground profile and blockages created by the proposed guideway support columns located along the transect. Figure 11 shows a photo of the center median in the vicinity of the proposed transit station. Sections 2.2 and 2.3 discussed how the volume loss created by the presence of the 25 support columns will be mitigated in the center median along Farrington Highway. Similar calculations were conducted to determine the amount of flow conveyance area lost due to the presence of the 25 columns in the 100-year floodplain. The 25 support columns are inundated to depths that vary from 0.26 to 3.33 feet deep. Knowing the depth at the center of each column and the diameter of each column allows us to compute the conveyance area below the 100-year water level (according to FEMA's BFEs) for each column. The sum of the conveyance areas for all of the columns along the guideway that are in the AE zone is 267 square feet. The proposed volume mitigation measure described in Section 2.3 (creating a slightly depressed center median along a 700-foot portion of Farrington Highway) will provide an additional 350 square feet of conveyance along Transect B. This is shown in Figure 10 between stations 1600 and 2400. The modified center median mitigation measure will more than compensate for the small conveyance loss created by the support columns.

AR00041607

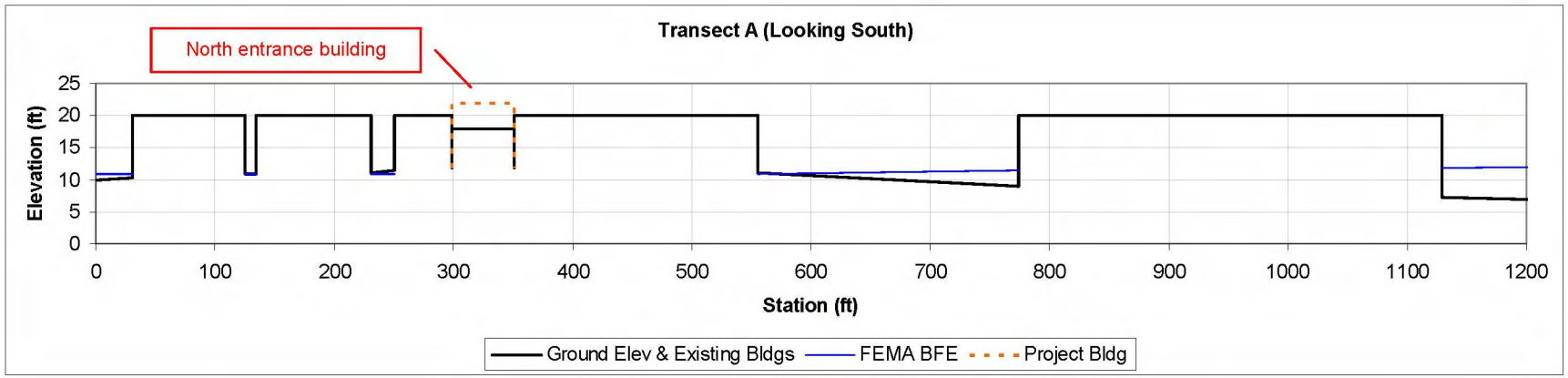

Figure 4. Floodplain Transect A (see Figure 1) showing how the north entrance building will replace the existing building in the same location.

AR00041608

Figure 5a. Photo of existing building (looking south from Hikimoe Street) that will be removed and replaced by north entrance building. Image from www.maps.google.com.

Figure 5b. Photo of existing building (looking north from Farrington Highway) that will be removed and replaced by north entrance building. Image from www.maps.google.com.

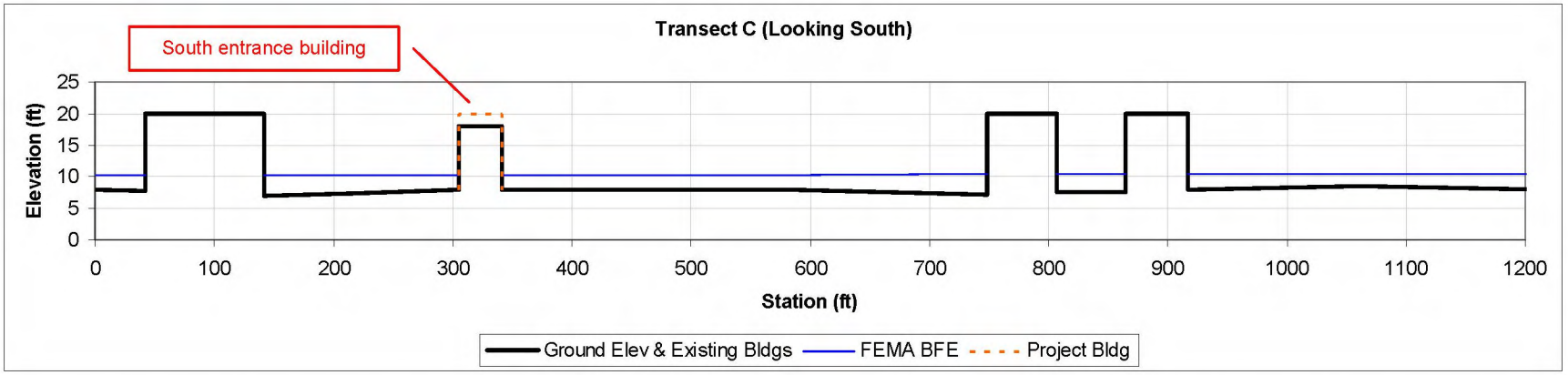

Figure 6. Floodplain Transect C (see Figure 1) showing how the south entrance building will replace existing building with the same exterior dimensions in the same approximate location.

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Figure 7. Photo of existing building (looking south from Farrington Highway) that will be removed and replaced by the south entrance building that will have the same exterior dimensions. Image from www.maps.google.com.

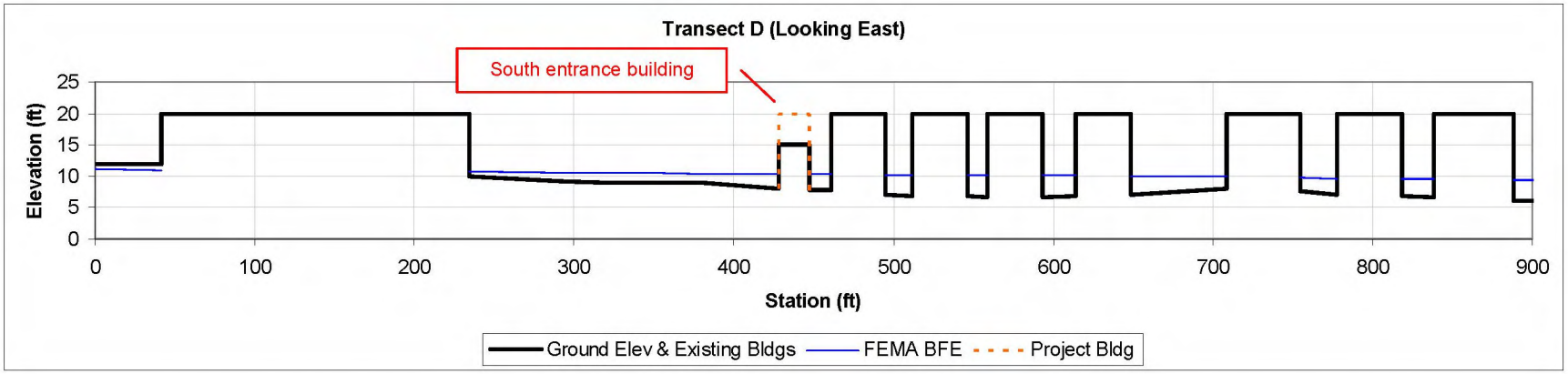

Figure 8. Floodplain Transect D (see Figure 1) showing how the south entrance building will replace the existing building with the same exterior dimensions in the same approximate location.

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Figure 9. Photo of existing building (looking east along Farrington Highway) that will be removed and replaced by the south entrance building that will have the same exterior dimensions. Image from www.maps.google.com

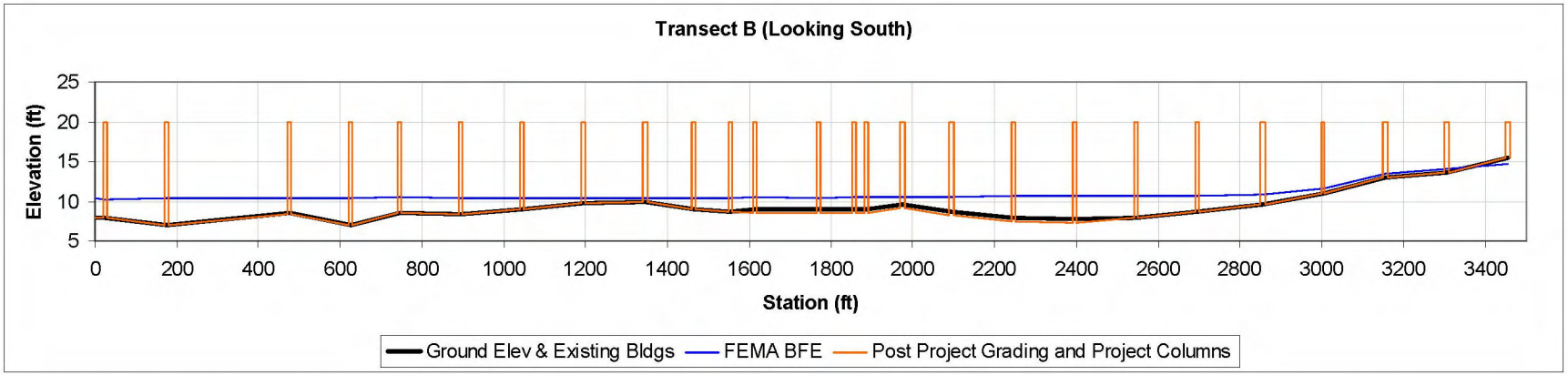

Figure 10. Floodplain Transect B (see Figure 1) showing locations of guideway support columns and proposed mitigation grading in a 700-foot long portion of center median along Farrington Highway.

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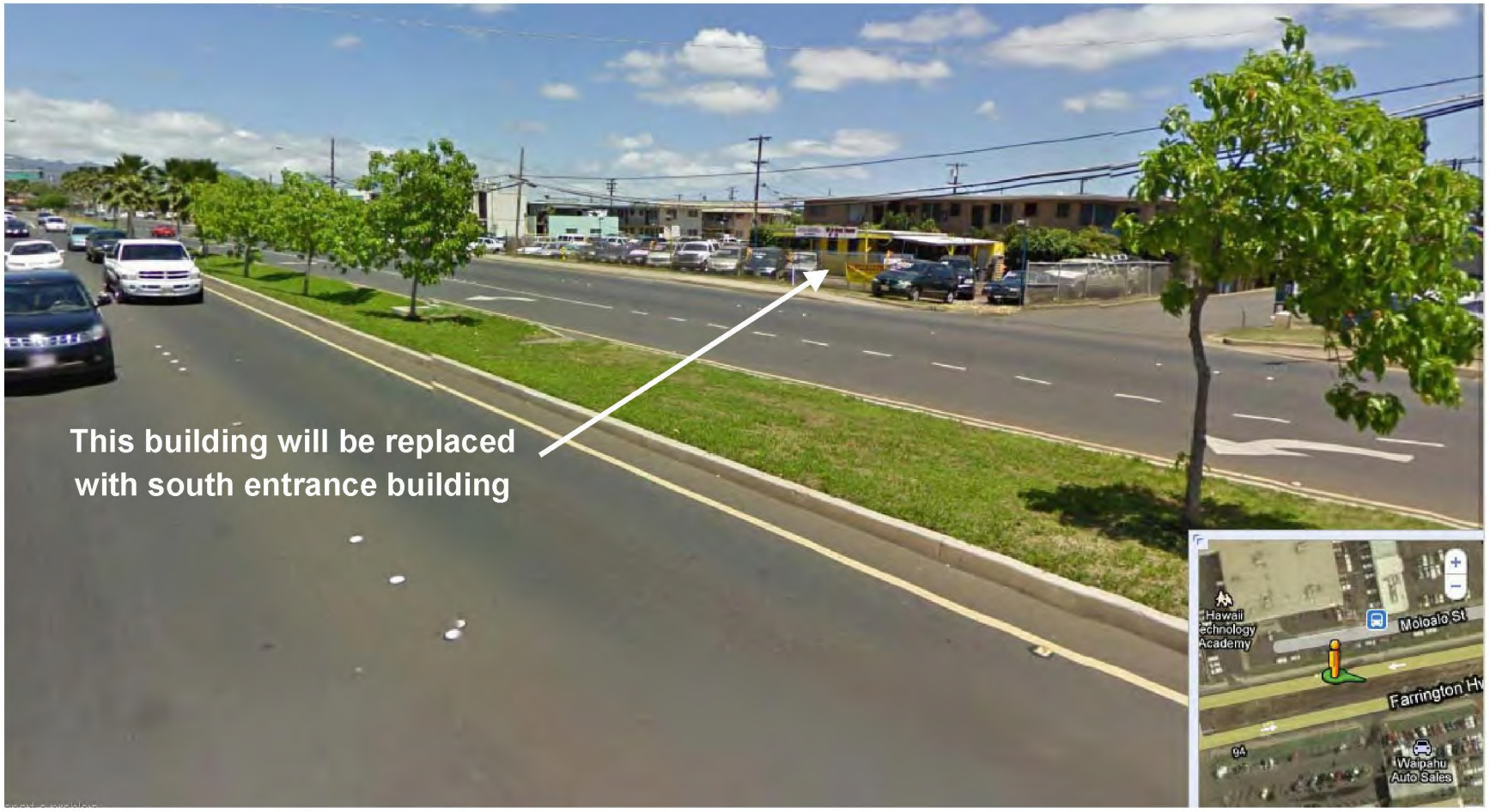

Figure 11. Photo of center median along Farrington Highway (looking south-east). Guideway support columns will be constructed along this median. Image from www.maps.google.com.

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3. No Rise Analysis

Section 2 discusses the analyses and results from the volume displacement and conveyance calculations that were performed for this project. Results presented in Section 2 show that the preferred project alternative for the two entrance buildings will not obstruct flow or change present day conveyance conditions at either entrance building site. Results also show that all of the storage volume losses (3,284 cubic feet) and the very small amount of conveyance loss (267 square feet) caused by the 25 support columns located in the floodway are easily replaced with simple modifications to a portion of the center median along Farrington Highway. Therefore, because the project will not change or obstruct any conveyance due to the new entrance buildings and because all other volume and conveyances losses are small and will be completely mitigated within the median along Farrington Highway, there will be no effects from the project on FEMA's 100-year water levels.

A more detailed hydraulic evaluation of floodplain flooding conditions throughout the Waipahu study area was initially completed by NHC in November 2009 and reported in April 2010. Results from those advanced modeling studies identified important flow characteristics near the project and led to the preparation of the proposed preferred project alternative that is described as Alternative 3 in Section 2, above. Construction of the preferred project Alternative 3 along with the proposed volume and conveyance mitigation measures reported herein, will not create adverse changes in 100-year flood water levels, available floodplain storage volume, or conveyance.

Reference

NHC, (2010), "Advanced Floodplain Evaluations for Waipahu Transit Center Station - Waikele, Kapakahi and Wailani Streams." Draft report prepared for Parsons Brinckerhoff Inc. by Northwest Hydraulic Consultants Inc., April 30, 2010.

Appendix A: Data and Methodology Used for Computing Project Displaced Volume and Conveyance Changes

- **Original Data Sets Used:**

  Ground Elevations
  Dataset Name - new_lidar_2
  Spatial Reference: NAD_1983_StatePlane_Hawaii_3_FIPS_5103_Feet
  Linear Unit: Foot US
  Vertical Unit: feet
  Cell Size: 5x5 ft
  Vertical Datum: NGVD 29
  Horizontal Datum: NAD 83
  Note: This dataset contains ground elevations from the "Project Corridor Contours", FEMA LiDAR, NGA LiDAR, and has been further processed. However, the building and column areas in this analysis fell within the "Project Corridor Contour" region of the ground elevation raster dataset. For more details about this dataset, please read the following document: FEMA LiDAR Checking Memo.doc

  Water Surface Elevations
  Dataset Name - S_BFE.shp
  Spatial Reference: NAD_1983_StatePlane_Hawaii_3_FIPS_5103_Feet
  Linear Unit: Foot_US
  Vertical Unit: feet
  Vertical Datum: NGVD 29
  Horizontal Datum: NAD 83
  Note: Water surface elevations were calculated by linearly interpolating between the S_BFE base flood elevation lines, provided by the FEMA DFIRM database. For more information regarding the FEMA DFIRM database, please read the following document: 15003C_20050602_metadata.txt

  Flood Zone Areas
  Dataset Name - S_Fld_Haz_Ar.shp
  Spatial Reference: NAD_1983_StatePlane_Hawaii_3_FIPS_5103_Feet
  Linear Unit: Foot_US
  Vertical Unit: feet
  Vertical Datum: NGVD 29
  Horizontal Datum: NAD 83
  Note: Flood Zones, specifically the floodway & AE Zones, were used to eliminate building & column areas from the displacement calculation analysis. The flood zones were provided by the FEMA DFIRM database. For more information regarding the FEMA DFIRM database, please read the following document: 15003C_20050602_metadata.txt

Buildings & Column Areas
Dataset Name - Buildings_Columns.shp
Spatial Reference: NAD_1983_StatePlane_Hawaii_3_FIPS_5103_Feet
Linear Unit: Foot_US
Horizontal Datum: NAD 83
Note: Building & Column touch-down locations & geometry for the Waipahu Transit Center Station was provided by Parsons Brinkerhoff, 10/2009.

- **Methodology Used to Compute Project Displaced Volume and Conveyance Changes**

1. The project buildings and columns shapefile (Buildings_Columns.shp) was clipped with the DFIRM database flood zones (S_Fld_Haz_Ar.shp), specifically the floodway & AE flood zone areas. Building and column areas outside these zones were not considered during the displacement calculation analysis.
2. The building and column areas inside the Floodway and AE zones were subdivided into 1x1 ft grid polygons. (Note: the grid polygons along the outer perimeter of buildings and columns are less than a square foot.)
3. A water surface elevation surface was generated by linearly interpolating between the DFIRM database base flood elevation lines (S_BFE.shp).
4. Each 1x1ft grid building/column polygon was assigned with an interpolated water surface elevation from the surface generated in Step 3.
5. Each 1x1ft grid building/column polygon was assigned with a ground elevation from the new_lidar_2 dataset.
6. A water depth calculation was generated where the ground elevations were subtracted from the water surface elevations for each 1x1ft grid polygon. If a negative value was rendered, then that 1x1ft grid polygon was classified as "not flooded", and was then disregarded for the remainder of the analysis.
7. For the remaining "flooded" 1x1ft grid polygons, an area attribute was calculated. (Note: the grid polygons along the outer perimeter of buildings and columns are less than a square foot.)
8. Displacement volumes were calculated by multiplying area and depth.
9. A new building/column shapefile was generated, called "Building_Displacement_Analysis.shp", which summarizes the results from the 1x1ft grid polygon shapefile. Attributes of this shapefile are listed below:

    - TYPE – type of structure, building or column
    - AVG_BFE – mean of all 1x1ft grid polygons for 'flooded' FEMA BFEs - in NGVD29 feet
    - AVG_LIDAR – mean of all 1x1ft grid polygons for 'flooded' LiDAR ground elevations - in NGVD29 feet
    - AVG_DEPTH – mean of all 1x1ft grid polygons for 'flooded' depths (ground elevations subtracted from BFEs) – in feet
    - VOL – Total volume displaced tallied from the 1x1ft grid polygons shapefile – in cubic feet

Table A-1. Summary of displaced volumes and conveyance changes associated with proposed HHCTCP structures in Waipahu.

Structures	Displaced volume			Blocked conveyance area	
	Footprint area (ft^2)	Average water depth (ft)	Volume displaced by feature (ft^3)	Water depth at center of project structure (ft)	Conveyance area blocked by feature (ft^2)
North Entrance	1309.43	0.19	249.0	NA*	No Change*
South Entrance	708.00	2.33	1650.0	NA**	No Change **
Column 1	BFE is lower than the ground elevation, so no flooding at this column				
Column 2	BFE is lower than the ground elevation, so no flooding at this column				
Column 3	13.62***	1.64	23.5	0.00	0.0
Column 4	23.86***	0.60	14.4	0.57	3.4
Column 5	36.00	0.35	10.9	0.40	2.4
Column 6	36.00	0.26	9.4	0.61	3.7
Column 7	36.00	0.94	32.8	1.11	6.7
Column 8	36.00	1.84	66.0	1.98	11.9
Column 9	36.00	2.65	95.5	2.66	15.9
Column 10	36.00	2.65	95.2	3.34	20.0
Column 11	36.00	2.63	94.7	3.13	18.8
Column 12	36.00	1.78	64.0	2.24	13.4
Column 13	36.00	0.93	33.6	1.42	8.5
Column 14	25.00	1.48	37.0	2.01	12.1
Column 15	25.00	1.48	37.0	1.99	12.0
Column 16	36.00	1.47	52.9	1.96	11.8
Column 17	36.00	1.46	52.4	1.96	11.8
Column 18	36.00	1.49	53.6	1.53	9.2
Column 19	36.00	0.46	16.4	0.49	2.9
Column 20	36.00	0.54	19.3	0.62	3.7
Column 21	36.00	1.44	51.8	1.47	8.8
Column 22	36.00	2.07	74.4	2.12	12.7
Column 23	36.00	1.69	60.7	1.96	11.8
Column 24	36.00	3.33	119.9	3.46	20.8
Column 25	36.00	1.93	69.5	1.96	11.8
Column 26	36.00	3.30	118.6	3.34	20.0
Column 27	36.00	2.26	81.3	2.24	13.4

* Presently there is no flow (only shallow stagnant ponding) during a 100-year event at location where the north entrance building will be constructed so there is no change in conveyance (see Section 2.2.4).

** The south entrance building will replace an existing building with the same maximum exterior dimensions so there is no change in conveyance (see Section 2.2.4).

*** Columns 3 and 4 are only partially flooded.

Appendix B: Volume Gain Calculations Needed for Volume Loss Mitigation

NHC calculated the volume that can be gained by constructing a shallow swale along a portion of the median dividing Farrington Highway in the immediate vicinity of the two entrance buildings (see Figure B-2). The median section between Mokuola Street and Waipahu Depot Street is 1,130 ft long and with an average width of 17 ft. Approximately 700 ft of the median is available for creating a volume and conveyance mitigation swale in this location. A sketch illustrating the plan and profile view of the conceptualized mitigation swale is provided in Figure B-1. Based on the dimensions given in Figure B-1, the added volume created by the 6-inch deep mitigation swale is 5,775 cubic feet. See below:

Added volume = 700 feet x (17 feet + 16 feet) / 2 x 0.5 feet = **5,775 cubic feet**.

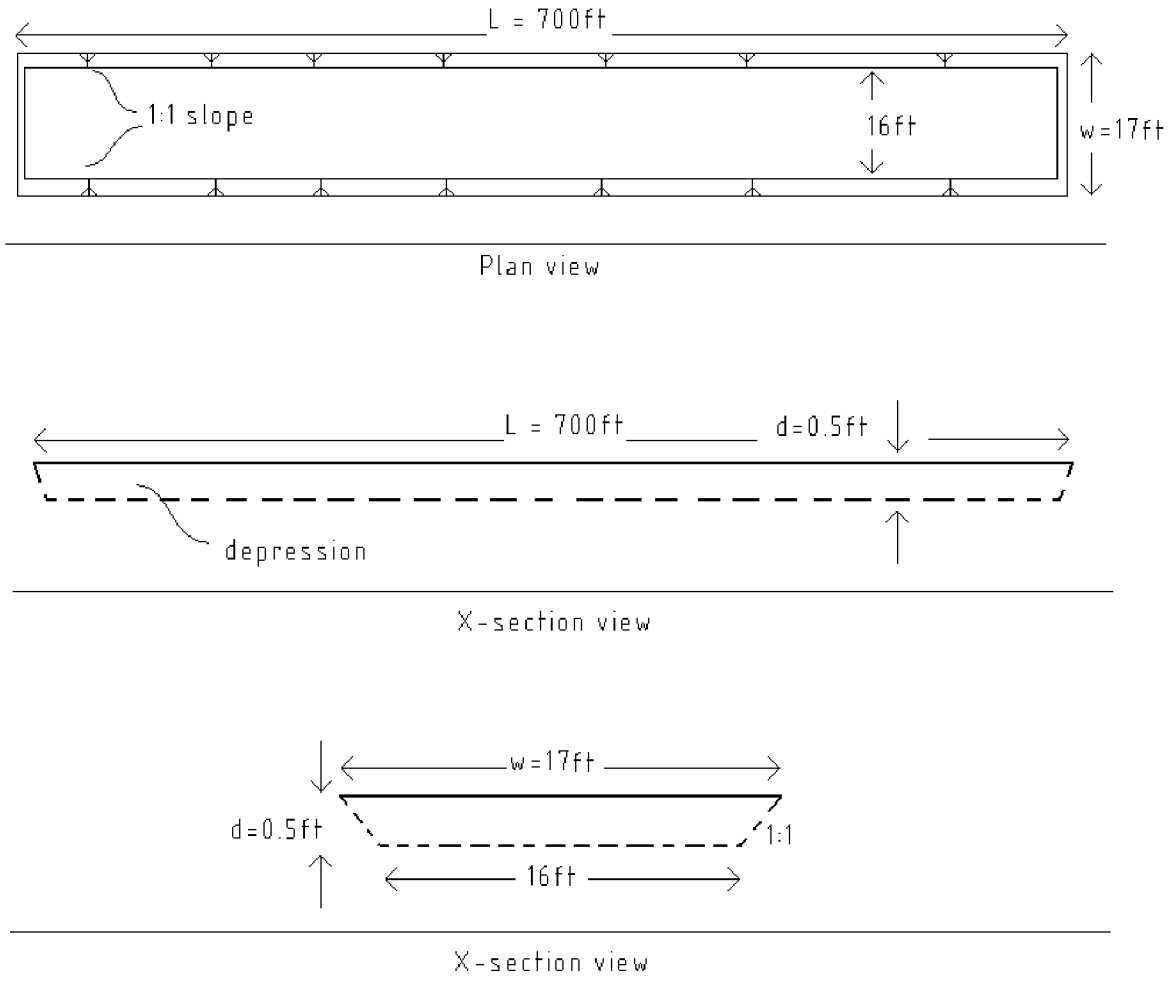

Figure B-1. Mitigation swale along a portion of the center median, Farrington Highway.

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Figure B-2. Recommended locations for modification of Farrington Highway median between Waipahu Depot Street and Mokuola Street in vicinity of project station (white lines).