Final Report

Port Facility Analysis for Kawaihae Harbor

October 15, 2006, Kiholo Bay Earthquake

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ASCE	American Society of Civil Engineers
DC	dynamic compaction
DOT	State of Hawaii Department of Transportation
DOT Harbors	State of Hawaii Department of Transportation, Harbors Division
DSM	deep soil mixing
EERI	Earthquake Engineering Research Institute
FE	finite element
FEMA	Federal Emergency Management Agency
ft/sec	feet per second
FRP	fiber-reinforced polymer
IBC	International Building Code
km	kilometer (km)
ksf	kip(s) per square foot
m	meter(s)
M	moment magnitude
makai	seaward
mauka	land side
MLLW	Mean Lower Low Water
MMI	Modified Mercalli Intensity
MOTEMS	Marine Oil Terminal Engineering and Maintenance Standards
NEES	Network for Earthquake Engineering Simulation
NEHRP	National Earthquake Hazards Reduction Program
PA	Public Assistance (Program)
PBD	Performance Based Design
PGA	peak horizontal ground acceleration
PML	Probable Maximum Loss
psf	pounds per square foot
SASW	Spectral Analysis of Surface Waves
Sea Engineering	Sea Engineering, Inc.
SPT	standard penetration test
TAC	Technical Assistance contractor
UBC	Uniform Building Code
UFC	United Facilities Criteria

USACE	United States Army Corps of Engineers
USCG	United States Coast Guard
USGS	United States Geological Survey
VR	vibro replacement
VS	shear-wave velocity

SECTION ONE INTRODUCTION AND PROJECT DESCRIPTION

On October 15, 2006, a moment magnitude (**M**) 6.7 earthquake struck the northwest coast of the island of Hawaii (County of Hawaii in the State of Hawaii). The earthquake, known as the Kiholo Bay earthquake, was centered 12.5 miles northeast of Kona Airport and occurred at a depth of 24 miles. The earthquake caused significant damage to port facilities at Kawaihae Harbor, which is located on the coast of the island of Hawaii about 20 miles from the earthquake's epicenter. Figure 1-1 provides a map of the area. A **M** 4.0 aftershock on October 17, 2006, did further damage to the port facilities. The damage from these events forced the temporary closure of the port. The port serves as the point of entry for most of the goods for the western side of the island of Hawaii and is critical to the continued well-being of island residents and businesses.

In response to the earthquake, the President of the United States declared the event to be a disaster, allowing the Federal government to provide assistance for recovery, including repairs to damage at the port. Federal response and recovery assistance was coordinated by the U.S. Department of Homeland Security's Federal Emergency Management Agency (FEMA). Among other activities, FEMA provided funding through the Public Assistance Program for repairs to damaged harbor facilities. Given the vulnerability of the port to damage from seismic events and the critical nature of the port to the economic viability of the island of Hawaii, FEMA conducted an evaluation of the damage to the port and an assessment of potential mitigation measures to yield recommendations that would reduce the risk of damage during future events. The conclusions of this case history study are relevant to the potential hazard risks at other Pacific island ports having similar local-shipping-based economies, construction resources, coral soils, and seismic risk. Further, given the potentially significant national economic impact that could result from earthquake damage to other larger U.S. port facilities on the Pacific Ocean, FEMA is documenting the lessons from this event to aid broader hazard mitigation efforts. This report, which FEMA has prepared under the Hazard Mitigation Technical Assistance Program, documents the results of the Kawaihae Harbor investigation.

1.1 BACKGROUND

Experience from around the world has shown that ports often suffer severe damage from earthquakes and that this damage can result in extensive interruptions to shipping operations and significant economic impacts, both locally and nationally. Port facilities are especially vulnerable to damage from earthquakes due to conditions that are common at many ports, including

- Soils close to the water tend to be alluvial in nature and of relatively poor quality.
- Fill soils are often present and also tend to be of poor quality.
- Soils with a high water content are often present; these soils can liquefy in severe earthquakes.
- The types of structures present at ports (such as piers or wharves) may be prone to damage during shaking.

Common forms of liquefaction-induced damage at ports have included the following:

- Deformation and failure of dikes

- Lateral, rotational, and vertical displacement and deformation of retaining structures and their backfill materials
- Buckling, yielding, and fracture of pile supports at piers and wharves (particularly batter piles)
- Failure and collapse of cranes due primarily to large relative movement of landward and seaward crane rails that are not structurally connected
- Extensive settlement and cracking of pavements in storage yards and along access roadways
- Damage to buried pipelines

The January 1995 Kobe (Hyogo-Ken Nanbu) earthquake in Japan presents a prominent example of the extensive damage that can occur to port facilities. This **M** 7.2 earthquake caused severe damage to the Port of Kobe, which is one of the largest ports in Japan. The three main port facilities consisted of perimeter quay walls filled with granular hydraulic fill on sea-bottom clay. The earthquake caused severe damage to cranes and other handling equipment as well as buildings, bridges, and other structures. Large sections of wharf and warehousing areas sank and were covered with water.

Recent earthquakes in the United States have not significantly affected port facilities, due primarily to the location of those events. Both the 1989 Loma Prieta earthquake in the San Francisco Bay Area and the 1994 Northridge earthquake in the Los Angeles area occurred a sufficient distance from nearby ports to spare those facilities from significant damage. However, even though the 1989 Loma Prieta earthquake was over 60 miles to the south, the Port of Oakland nonetheless suffered damage, due primarily to the liquefaction of hydraulic fill.

Many of our country's most important ports are located in areas of seismic risk. The Ports of Long Beach, Los Angeles, Oakland, and Seattle, which handle almost all of cargo shipments for the West Coast, are all vulnerable to earthquake damage. Also, several East Coast ports, including the ports of Boston, New York, and Charleston, are located in moderate or high seismic areas and are at even higher risk because most of them were not designed to account for earthquakes. The vulnerability of ports is particularly pronounced on Pacific islands, such as Hawaii, where ports are the lifelines for the populations; damage from an earthquake that closes an island port could eliminate the primary means for providing relief supplies and encouraging economic recovery.

In the past, FEMA has attempted to work with the port industry to develop guidance to address these hazards. In the update process that resulted in the 2003 edition of FEMA's *Recommended Provisions for New Buildings and Other Structures* (FEMA 450) (FEMA 2003), FEMA attempted to work with representatives of the port industry to include guidance on the seismic design of port facilities. However, the material that they provided did not pass the required consensus ballot, primarily because they had proposed using smaller earthquake design maps than those already referenced in the document. Additional initiatives undertaken with the port industry, including those of the American Lifelines Alliance and the American Society of Civil Engineers (ASCE), have yet to develop consensus on design issues.

The earthquake damage to Kawaihae Harbor has provided FEMA with the opportunity to document earthquake damage to a U.S. facility. Documenting the performance of various port

facilities and structures allows FEMA to develop recommendations that can be used by port facility owners in American territories in the Pacific, where ports have been constructed according to similar design criteria and are subject to similar risks, as well as by port facility owners in the rest of the country.

1.2 SCOPE AND PREPARATION OF THE REPORT

1.2.1 Kawaihae Harbor

This report focuses on damage observed at Kawaihae Harbor—specifically, Piers 1, 2A, and 2B and their cargo yards. The general location of the harbor is shown on Figure 1-1. Landside port facilities are shown on Figure 1-2. The harbor is operated by the State of Hawaii Department of Transportation Harbors Division (DOT or DOT Harbors).

FEMA has prepared this report to accomplish the following:

- Document both the earthquake damage to the Kawaihae Harbor facilities and the structures that performed satisfactorily.
- Prepare a report on the findings of the investigation, including recommendations on how the design of these facilities might be improved.
- Provide the report to port facility owners in other parts of the United States, particularly ports on Pacific islands.
- Provide the report to the ASCE Standards Development Committee on Seismic Design of Ports for use by this committee.

The report undertook the following actions:

- Obtained available structural and geological records for the wharves and cargo yard structures
- Evaluated ground motion based on available records
- Reviewed bathymetry and dredging history
- Described visual assessment of the structural damage to the cargo yard structures, the above-deck portions of the wharves, and the underwater portion of the mooring dolphins
- Reviewed the damage assessments performed by others, including FEMA's Public Assistance Program and other studies performed by the State of Hawaii
- Documented the emergency measures implemented at the port so that it could return to service as quickly as possible
- Evaluated the performance of the wharves, mooring dolphins, and cargo yard structures
- Prepared recommendations for seismic mitigation at the port
- Prepared recommendations that can be used by other port owners.

With the support of URS Corporation, FEMA initiated this investigation in November 2006, before the implementation of permanent repairs at the port. The field investigation was

completed in June 2006, when an underside inspection of the wharves was performed. DOT Harbors provided access to the port, plans for port facilities, and other documentation.

1.2.2 Seismic Mitigation Guidelines for Pacific Island Ports

This report also offers seismic mitigation guidelines for Pacific island ports. The seismic mitigation guidance for Pacific island ports includes the following:

- Summary of the typical design and construction features of Pacific island ports
- Recommendations for seismic mitigations for Pacific island port facilities
- Recommended prioritization of areas of need

Key issues include liquefaction of coral soils, unconventional seismic attenuation, and clearly graded degrees of structural damage correlated with age of design. The guidance draws on the example of how responders performed emergency measures to provide timely and effective restoration of services at Kawaihae Harbor after the Kiholo Bay earthquake. Concepts for recommended mitigations are aimed at utilizing limited locally available resources, identifying low-cost mitigations, and providing education on concepts of tolerable risk.

1.3 ORGANIZATION OF THIS REPORT

After this introduction, Section 2 describes the port facilities at Kawaihae Harbor. Section 3 discusses the geologic conditions in the vicinity of Kawaihae Harbor. Section 4 describes the Kiholo Bay earthquake and its effects on Kawaihae Harbor. Section 5 discusses the emergency response measures taken at Kawaihae Harbor after the earthquake. Section 6 details the observations of damage to the harbor facilities as a result of the earthquake. Section 7 is a seismic performance evaluation of the Kawaihae Harbor facilities. Section 8 takes the results of the seismic evaluation of Kawaihae Harbor and discusses how the findings are relevant to other Pacific island ports. Specifically, this section offers general seismic mitigation guidelines for Pacific island ports as well as other ports of similar construction and configuration.

Section 9 discusses the limitations of this analysis. Section 10 lists the documents and drawings consulted in preparing this report.

Appendix A shows photographs of the Kawaihae Harbor facilities and the earthquake damage. Appendix B contains a comparison of different codes and design procedures. Appendix C contains excerpts from geotechnical data, boring logs, and pile driving records. Appendix D provides seismicity data, ground motion evaluations, and a liquefaction primer and evaluation. Appendix E provides excerpts from FEMA 310 (Seismic Evaluation Handbook). Appendix F provides excerpts from seismic mitigation references.

Throughout this report, the reference drawings are identified in brackets [ ], as appropriate. The drawings consulted in the preparation of this report are listed in Section 11.

1.4 URS PROJECT TEAM'S SITE VISIT

During the week of December 4, 2006, the URS project team performed an inspection of the facilities at Kawaihae Harbor to assess the damage that had occurred as a result of the October 15, 2006, Kiholo Bay seismic event. After project orientation with FEMA in Honolulu on

December 4, the URS project team conducted an initial site visit with FEMA Public Assistance Program representatives on December 5. During the initial inspection, the URS project team met with harbor operations personnel and observed key facilities, including the bulkhead at Pier 1, the concrete wharves at Piers 2A and 2B, the two metal buildings on Pier 1 (the North and South Metal Buildings), and the one metal building (Transit Shed) on Pier 2A. A subsequent orientation and debriefing meeting was held with FEMA and DOT Harbors in Honolulu on December 6. After this meeting, reference drawings were obtained for many of the structures from the DOT Harbors archives.

On December 7, the URS project team made a follow-up site visit to Kawaihae Harbor. In the follow-up visit, several other site facilities were observed, including the fuel tanks, the concrete plant, the Brewer Building, the control building, light poles and canals, and the outer harbor and breakwater. On the final day of the investigation, the URS project team collected additional archived documents, visited Hilo Harbor to meet with operations personnel (to discuss similarities and differences between conditions at Hilo and Kawaihae), and visited the Hawaii Volcanoes Observatory for a project orientation with the United States Geological Survey (USGS) and an exchange of data with the USGS.

SECTION TWO KAWAIHAE HARBOR FACILITIES

This section describes the Kawaihae Harbor facilities based on the field observations and the archive drawings. The harbor facilities described in this section are shown on Figure 1-2.

2.1 STRUCTURAL: WHARVES AND MOORING DOLPHINS

2.1.1 Pier 1

Pier 1 is part of a dredging and site improvement project constructed in March 1954 [HC 916A]. The project, also known as the Barge Pier, consists of a 410-foot-long concrete sheet pile bulkhead. The bulkhead was constructed using 16-inch-thick by 24-inch-wide by 35-foot-long precast concrete sheet piles. The elevation of the top of the wall is at +8.00 feet mean lower low water (MLLW), and the dredge line in front of the wall is approximately at elevation -20.00 feet MLLW. The bulkhead is tied back to a continuous 1-foot-wide by 4-foot-deep concrete anchor with a tie rod anchor system consisting of four #8 reinforcing bars encased in an 8-inch by 8-inch concrete jacket spaced at 10 feet on center. The anchor block is approximately 40 feet behind the back of the bulkhead wall, and the top of the anchor is approximately 6 inches below the surface. The anchor system is attached to the bulkhead through a 2-foot, 4-inch-wide by 4-foot, 6-inch-deep reinforced concrete coping beam. A 6-inch-thick reinforced concrete slab-on-grade also ties the coping beam and the anchor block together. Figures 2-1, 2-2, and 2-3 show a typical section through Pier 1, a typical pile bulkhead wall, and a typical section of the bulkhead wall and anchor system at Pier 1, respectively.

2.1.2 Pier 2A

Pier 2A is a 38-foot-wide by 602-foot-long concrete wharf supported by vertical concrete piles; Pier 2A was constructed in January 1958 [HC 1031**].** The top of deck elevation is at +8.00 feet MLLW. The maximum dredge line is approximately at elevation -35.00 feet MLLW. The wharf deck is a 1-foot-thick reinforced concrete slab supported by 2-foot, 6-inch-wide by 3-foot-deep reinforced concrete pile cap beams at 14 feet on center. The concrete beams are supported by seven 18-inch square conventionally reinforced vertical concrete piles spaced at 5 feet, 9 inches on center. The rear cutoff wall is constructed using a 1-foot, 3-inch-thick by 8-foot deep reinforced concrete wall. The bottom of this cutoff wall is at elevation +0.00 feet MLLW. The remainder of the cutoff wall from elevation +0.00 feet is constructed using steel sheet piles. Figure 2-4 shows a typical section through Pier 2A.

2.1.3 Pier 2B

Pier 2B is an extension of Pier 2A and was constructed in May 1990 [HC 5219]. The pier is a concrete wharf that is approximately 44 feet, 9 inches wide by 600 feet long and is supported by both vertical and battered concrete piles. The top of deck elevation is +8.00 feet MLLW. The maximum dredge line is at approximately elevation -35.00 feet MLLW. The wharf deck is constructed using an 8-inch-thick topping slab over 10-inch prestressed concrete planks. The planks are supported by three longitudinal reinforced concrete pile cap beams: one along the front of the wharf, one at the middle of the wharf, and one along the back edge of the wharf. The

concrete pile cap beams are supported by vertical 20-inch octagonal precast, prestressed piles. The piles occur at 5-foot spacing along the front pile cap, 6-foot, 10-inch spacing along the middle pile cap, and 4-foot, 6-inch spacing along the back pile cap. The lateral loads are resisted by battered piles at 50-foot spacing in the longitudinal direction and battered piles at 4-foot, 6-inch spacing in the transverse direction. The piles are battered at an approximate slope of 2.5 vertical to 12 horizontal (2.5V:12H). A tie beam and anchor block (deadman) system is also provided at the back of the wharf. Figures 2-5 and 2-6 show a partial pier plan and a typical section through Pier 2B, respectively.

2.1.4 Mooring Dolphins

Three reinforced-concrete mooring dolphins are located on the west side of the harbor adjacent to the breakwater. They are used occasionally for operations associated with the United States Coast Guard (USCG) and Hawaii National Guard properties. The two larger dolphins are supported by concrete piles, and the small dolphin is supported on steel piles. The dolphins are believed to have been constructed during the 1980s. The dolphins were not in use during the earthquake. The location of the mooring dolphins is shown on Figure 1-2.

2.2 STRUCTURAL: WATERFRONT BUILDINGS

2.2.1 Pier 1: North Metal Building

The Pier 1 North Metal Building (also referred to as the North Transit Shed or N. Transit Shed) is a single-story, structural-steel building. The roof measures approximately 90 feet by 60 feet in plan, including a 15-foot-wide canopy that extends on two sides of the structure. The enclosed floor area is approximately 3,600 square feet.

Steel channel purlins supporting corrugated metal roofing span between steel wide-flange girders. The girders are supported by wide-flange columns at the exterior wall and are connected together at the ridge with a full-penetration weld. Horizontal steel-channel girts span between columns and support the exterior metal wall siding. The interior floor is a concrete slab-on-grade, as is the enclosed overhang area.

Lateral loads in the transverse direction are resisted by a combination of steel moment-resisting frames on the interior and tension-only braced frames on the end walls. In the longitudinal direction, tension-only braced frames are provided along each wall.

Figures 2-7 and 2-8 show, respectively, a floor plan of the North Metal Building and a section through the North Metal Building.

2.2.2 Pier 1: South Metal Building

The Pier 1 South Metal Building (also referred to as the South Transit Shed or S. Transit Shed) is a 5,000-square-foot single-story pre-engineered metal building constructed around 1968. The enclosed floor area measures approximately 50 feet by 100 feet in plan. The roof extends beyond the longitudinal exterior walls, providing an additional 3,500 square feet of covered area.

Metal purlins supporting sheet metal roofing span between tapered structural steel girders, which in turn are supported by tapered columns at the exterior wall. Horizontal metal wall girts with

sag rods span between columns and support the exterior metal wall siding. The foundation system consists of isolated concrete footings below each steel column, and the interior floor is a concrete slab-on-grade.

Lateral loads on the building are resisted by steel moment-resisting frames in the short direction and tension-only braced frames in the longitudinal direction. The configuration of the structural system, connection details, and material strengths, was taken from construction drawings dated January 1968 [HC 1331R].

Figures 2-9 and 2-10 show, respectively, a floor plan for the South Metal Building and a section through the South Metal Building.

2.2.3 Pier 2A: Transit Shed

The Pier 2A Transit Shed is a single-story, structural-steel building constructed around 1959. It is located immediately adjacent to Pier 2A. The roof measures 196 feet by 80 feet in plan, including a 20-foot-wide canopy that runs the length of the building. The enclosed floor area is approximately 12,000 square feet.

Steel channel purlins supporting corrugated metal roofing span between steel wide-flange girders. The girders are supported by wide-flange columns at the exterior wall and are connected together at the ridge with a full-penetration weld. Horizontal steel-channel girts span between columns and support the exterior metal wall siding. Columns along the south wall are founded on the first bent of the abutting Pier 2A pile-supported concrete wharf; columns along the north wall are supported on precast concrete piles with concrete pile caps. The interior floor is a concrete slab-on-grade, as is the enclosed overhang area on the southeast corner of the building that houses the Young Brothers, Inc. (Young Brothers), shipping office.

Lateral loads in the transverse direction are resisted by a combination of steel moment-resisting frames on the interior and tension-only braced frames on the end walls. In the longitudinal direction, two bays of tension-only braced frames are provided on each wall. The configuration of the structural system, connection details, and material strengths, was taken from the construction drawings dated April 2, 1959 [HC 1031].

Figures 2-11 and 2-12 show, respectively, a plan of the Transit Shed and a section through the Transit Shed.

2.2.4 Brewer Building

The Brewer Building is a one-story-high bay concrete building that was constructed before 1970. The building is used for material storage. No construction documents were available to provide more detailed descriptions of the systems. Based on field observations, the concrete roof slab is supported by reinforced-concrete beams and columns. Lateral loads in each direction are resisted by concrete moment-resisting frames with concrete and masonry infill walls.

2.3 STRUCTURAL: OTHER SITE FACILITIES

Other site facilities observed and photographed during the URS project team site inspection include fuel tanks, a concrete plant, the harbor control building, light poles, a lined drainage canal, underground utilities, and pavements. Each of these facilities is briefly described below to

provide perspective on the extent of developments at Kawaihae Harbor. No drawings were available for any of these structures.

2.3.1 Fuel Tanks

Two fuel tank farms are located within Kawaihae Harbor: the Union Oil tank farm, which is located north of the port office and adjacent to the cement plant, and the Standard Oil tank farm, which is located immediately south of the main entrance. The tank farms are in unpaved areas along the mauka (landside) boundary of the harbor facilities; each has four tanks of varying sizes, and each appears to be supported by shallow ring-wall footings, as is typical for this region. Each tank farm appears to be in good condition. Service has continued uninterrupted since the earthquake.

2.3.2 Concrete Plant

The concrete plant includes a steel silo, mechanical equipment, and a two-story administration building. No construction documents were available to provide a more detailed description of these systems. Field observations indicated that the two-story administration building appears to be constructed of load-bearing masonry walls with a concrete floor and roof.

2.3.3 Control Building

The control building consists of a 4,000-square-foot, two-story, light-frame structure supported on shallow footings. The lower floor is constructed of masonry block, and the upper floor is constructed of a single-wall wood frame. The Kawaihae Harbor personnel occupying the building during the earthquake reported that the upper floor of the control building was subjected to significantly more shaking than the lower floor, as evidenced by the toppling of bookshelves and computers on the upper floor.

2.3.4 Light Poles

Several dozen light poles are located throughout the harbor yard. The light poles are constructed of tapered steel shafts bolted to 2-foot-diameter concrete base pedestals.

2.3.5 Drainage Canal

A grouted riprap and concrete-lined open channel canal forming a storm water collection system surrounds the perimeter of the main Kawaihae Harbor facilities. The canal discharges to the harbor between Pier 1 and Pier 2A. The canal was constructed in 1958 in association with the development of Pier 1. Various areas of ground and concrete lining patches were evident, with some localized areas of exposed rebar at the culvert walls where the canal passes beneath the roadway connecting Pier 1 to the main terminal yard (behind Pier 2) and at the canal outlet, where the canal connects to the pier bulkheads and riprap protection.

2.3.6 Underground Utilities

Underground utilities at Kawaihae Harbor are relatively sparse, and all are of small diameter. Utilities include electrical conduits, water lines, sewer lines, and a 6-inch cement supply line between Pier 1 and the concrete plant.

2.3.7 Pavements

The surface area of the terminal yard, which is the main area of interest for this case history, is composed of paved asphalt abutting flush with Piers 2A and 2B pile-supported reinforced-concrete decks and a concrete slab-on-grade within the anchored sheet pile bulkhead at Pier 1. Service roads are also paved asphalt. The interiors of the cargo buildings are also predominantly asphalt, with some local concrete service pads or offices with a slab-on-grade. The age of the pavements varies generally with the age of the piers. The oldest pavements are in the yard behind Pier 1, and the newest are in the yard behind Pier 2B, which was reconstructed in the early 1990s [HC 5313].

Pavements in the terminal yard generally consist of 4 to 6 inches of asphalt cement underlain by 6 to 12 inches of aggregate base course. Drawings of the appurtenant service roads are not available, though local practice is generally to use 2 inches of asphalt cement underlain by 6 inches of aggregate base course. Crushed basalt materials are typically used for the base course.

SECTION THREE GEOLOGIC CONDITIONS

This discussion of geotechnical conditions at Kawaihae Harbor covers local geology, surface conditions, bathymetry and dredging history, and subsurface conditions.

3.1 LOCAL GEOLOGY

The island of Hawaii consists of five young, historically active volcanic mountains that increase in age from the southeast to the northwest. Kawaihae Harbor is situated on a broad shelf reef formation at the coalescence of the northwest flank of Mauna Kea and the southwest flank of Kohala Mountain. This latter mountain, which is the oldest volcano on the island of Hawaii, shields the harbor from trade winds and causes a more arid climate and less erosion at Kawaihae Harbor than occurs on the north Kohala coast. Two geologic units flank the north and south of Makahuna Gulch and stream, which feed the natural estuary and associated reef immediately south of the area where Kawaihae Harbor was dredged and the reclaimed land was created. The Pololu volcanic series to the north of the gulch is the oldest of Kohala basalt flows; typically, these flows are 5 to 20 feet thick. The Hamakua volcanic series to the south of the gulch is the oldest of Mauna Kea basalt flows; these flows are similar to the Kohala flows, though slightly younger and typically capped by a thin layer (less than 7 feet) of Pahala ash, a yellowish to reddish brown vitric ash and pumice carried by the wind. All of the deposits are determined to be less than 500,000 years old (MacDonald et al. 1983). Outer seafloor conditions slope fairly uniformly to the west and southwest for a distance of over 20 miles before forming the top of the submarine Kiholo Ridge at the earthquake epicenter (Figure 3-1).

The stress history and evolution of coastal reef formations in Hawaii are closely associated with past interglacial periods, where the sea level rose above the present level and then regressed to well below the present level during the glacial stages. These cycles of advance and retreat of the sea have produced reef deposits and later coralline limestone at varying levels. Surface erosion producing alluvial outwash combined with marine and wave action to create sandy beaches from intermixed soil, coral, and microfossil fragments. Silty lagoons can form near the beaches when the sea level rises and falls during glacial and interglacial periods. During periods of low sea level, alluvial channels (valleys) and erosional surfaces can also develop and extend well below the current mean sea level.

The resulting local geology in the coastal area of Kawaihae consists of a predominant volcanic basalt island core with flanks of shallow beaches and a fringe coral reef having local erosional channels, estuaries, and occasional intermixed newer lava flows. Primary local geologic units at the site include fills, coralline deposits, and underlying volcanic basalts. The broad coral reef formation that dominates the subsurface engineering considerations at Kawaihae extends approximately 1 mile offshore at a natural surface elevation of less than 5 feet below sea level. Its thickness varies with the naturally sloping basalt island cores, ranging from 20 feet to over 100 feet deep at the outer breakwater. The natural reef structures include variations of cemented rock, gravels, sands, and silts, with intermixing alluvial deposits, precipitations, solution cavities, and degrees of weathering (Figure 3-2).

3.2 SURFACE CONDITIONS

Kawaihae Harbor was conceived for construction in the 1950s as a dredged basin within the natural broad tidal flat associated with coral reef formation that would provide natural protection from the open ocean and abundant supply of dredge spoils for reclamation. The elevation of the reclaimed lands is approximately 8 to 15 feet MLLW datum, and these lands extend approximately 500 feet mauka of the waterfront piers to create the terminal yards. The slope of Kohala Mountain flanks the eastern boundary of the reclaimed harbor terminal yard and Kawaihae Road. The slope rises at approximately 20 percent from horizontal.

The reclaimed reef area south of Kawaihae Harbor creates a broad peninsular flat that extends approximately 1,000 feet makai (seaward) of the piers across the natural reef, forms a beach along the south harbor boundary, and terminates at the outer breakwater and small craft harbor (Figure 1-2). The beach and adjacent reclaimed areas are unpaved; the United States Coast Guard and the Hawaii National Guard use these areas for training exercises. The reclaimed area south of the harbor terminates at the outer breakwater and small craft harbor. The United States Army Corps of Engineers (USACE) owns the breakwater. The breakwater forms the west perimeter of the harbor, extending to the exposed northwest entrance channel. After conducting a performance assessment in 2005, USACE is currently evaluating the breakwater separately and anticipates future modifications.

The reclaimed area south of the harbor also connects to a broader low-lying flat bounded by a previously existing causeway extending over a half mile south to the ancient Kawaihae Heiau (temple) site, which is situated atop a prominent natural terrace overlooking the harbor.

Surface elevations in the terminal yard gradually rise in the mauka direction from 8 feet MLLW elevation at the bulkheads to 10 to 15 feet MLLW for the majority of the terminal yard.

3.3 BATHYMETRY AND DREDGING HISTORY

The original reef flat (with elevations of less than 5 feet below MLLW) was dredged during construction in the 1950s to create an original harbor bathymetry of 35 feet below MLLW in the basin and 40 feet below MLLW in the 360-foot-wide entrance channel extending over half a mile to the northwest beyond the reef boundary.

Construction drawings from July 1955 at Pier 1 [HC 916A] indicate a dredge depth of 20 feet below MLLW that extends 40 feet makai of the bulkhead, where a vertical cut then transitions to the harbor basin depth of 35 feet below MLLW. The newer Piers 2A and 2B were designed with sloping dredge limits extending from the 35 feet below MLLW harbor basin depth up to near the water surface elevation adjacent to cutoff walls at the rear of the wharf structures.

Because of the natural variations of strength and cementation with Hawaiian reef formations, occasional local sloughing, or dislodging of sloping armor stone during storms may all have been sources of maintenance dredging. Harbor personnel indicated that sedimentation feed by Makahuna Gulch and natural currents have also prompted harbor dredging. Maintenance dredging was performed in 1960 at Pier 1 [HC 1155], and then again in 1990 in conjunction with the development of Pier 2B, including a new bridge over the drainage channel [HC 5218, HC 5248].

The bathymetry survey that USACE performed in 2003 indicated local harbor bottom depths adjacent to Pier 1 that ranged from 10 to 28 feet below MLLW. The findings suggest some localized areas of sedimentation or overdredging, though the transition to the general harbor basin depth of 35 feet MLLW remains within a discrete zone, suggesting that the vertical cut is generally intact. The recorded harbor channel elevations range from 38 to 40 feet below MLLW, with an average elevation of approximately 42 feet below MLLW. The harbor channel passes approximately 100 feet in front of Pier 1 and within 20 feet of the front of Piers 2A and 2B. The remainder of the harbor basin depths range from approximately 35 to 39 feet below MLLW, transitioning to shallow depths along the south end of the harbor at the beach and gulch inlet. In summary, the 2003 bathymetry conditions appear to be generally consistent with the original construction provisions and the maintenance dredging in 1990.

In September 2006, not long before the Kiholo Bay earthquake, Sea Engineering, Inc. (Sea Engineering), performed a bathymetry and dive survey for DOT Harbors (Appendix C6). This work was conducted in association with the harbor expansion for the inter-island ferry system that is currently being developed. The September 2006 bathymetry data are generally consistent with the 2003 data.

After the earthquake, FEMA performed a lidar survey of the bathymetry of the harbor and reef shoals (Figures 3-3 and 3-4). When the December 2006 data are superimposed over the 2003 survey data, the results show a reasonable correlation in the areas of interest at Piers 1, 2A, and 2B, though the vertical accuracy of the 2003 contours is lacking (Figure 3-5). Potential earthquake movement is apparent at the north end of Pier 1. (Potential earthquake movement at Pier 1 is discussed in more detail in Section 7.)

3.4 SUBSURFACE CONDITIONS

The URS project team reviewed boring logs from the construction drawings for Piers 1, 2A, 2B, and their container yards extending northeast to Kawaihae Road (Figure 1-2) [HC 916A, HC 5218, HC 5219, HC 5313]. The construction drawings from Pier 1 also indicate original dredge fill limits over the original ground in the area of the terminal yard. An older dike is located approximately 80 feet from the water; details include a 1H:1V slope, a crest elevation of 7 feet, a core of dredged coral, armor stone of 2 feet in diameter, and a downslope terrace with an approximate width of 5 feet that is located approximately 80 feet from the water. The dike extends to an area between Piers 1 and 2A, where the downslope terrace forms a ledge along the edge of the harbor. Elsewhere the dike is buried beneath dredged fills.

Pile-driving records were also examined as an indication of soil conditions and the depth to the basalt formation [HC 5218, HC 5219]. A summary of the test pile program at Pier 2B provides a reasonable representation of general pile-driving conditions at Kawaihae (Appendix C2). Table 3-1 shows the estimated general conditions for the soil strata, the Standard Penetration Test (SPT) *N* values, and descriptions.

Table 3-1. General Subsurface Conditions at Kawaihae

Strata	Depth (ft)	SPT N	Description
1	0–5	10–30	Compacted medium dense coral sand and gravel dredged FILL.
2	5–10	4–10	Loose coral sand, silty sand and gravel dredged FILL.
3	10–45	10–50	Uncemented or weakly cemented loose to dense natural coral SAND & GRAVEL.
4	45–70	>20	WEATHERED BASALT with interbedded cinders and gravels.
5	55–70	Refusal	Layered BASALT rock, competent fresh formations.

Some borings contain loose fill in Stratum 1 or dense fill in Stratum 2. In general, the lower 5 feet of Stratum 3 were dense. Both the weathered and the fresh basalt interfaces at Strata 4 and 5 are sloping and variable: as shallow as 40 feet in some areas and over 70 feet in others. The general slope of the underlying basalt bedrock is approximately 10 percent. Refusal of both SPT and pile driving occurred in the fresh basalt (Figure 3-6).

The estimated general strength and engineering properties of the soils at Kawaihae are correlated from the SPT *N* values or are based on descriptions of density and composition in older borings with no SPT testing (Bowles 1996) (Table 3-2). Appendices C1 and C5 provide more details about the estimated strength and engineering properties of the soil. Appendices D3 and D4 provide additional data on coral soil behavior and properties for the liquefaction evaluation.

Table 3-2. Estimated General Strength and Engineering Properties of the Soils at Kawaihae Harbor

Type	General Strata	USGS Classifications	Unit Weight (pcf)	Moisture Content (%)	Friction Angle (degrees)	Cohesion (psf)	Young's Modulus (ksf)
Dense coral Sands & gravels	1,2,3	GP, GM, SP, SM	125-135	10-20	33-38	0-50	250-1000
Loose coral Sands & gravels	1,2,3	GP, GM, SP, SM	95-105	15-35	28-32	0-50	100-250
Basalt (weathered)	4	GP, GM, rock	115-140	5-15	35-40	>1000	<50,000
Basalt (fresh)	5	rock	125-145	5-15	40	>30,000	>50,000

SECTION FOUR KIHOLO BAY EARTHQUAKE AND SEISMIC HAZARDS

In this section, the URS project team performed a seismic hazard evaluation of Kawaihae Harbor to analyze the ground shaking, coralline liquefaction, lateral spread, and other geologic hazards. The evaluation included a study of earthquake strong motion records from the Kiholo Bay earthquake to provide a basis for estimating the ground shaking at Kawaihae Harbor. Unfortunately, no strong motion recordings are available for the Kiholo Bay earthquake at the harbor.

4.1 GROUND MOTION EVALUATION

As part of this study, the URS project team estimated the ground motions in Kawaihae Harbor during the October 15, 2006, **M** 6.7 Kiholo Bay earthquake. To characterize the ground shaking in Kawaihae Harbor, it is essential to understand the strong motion records of the earthquake from the USGS Hawaiian Strong Motion Network. (In this section, the measurements are generally expressed in metric units, as the scale used in seismology uses metric units.)

4.1.1 Hawaii Seismicity

The island of Hawaii is one of the most seismically active regions in the United States (Figure 4-1), with seismicity and seismic hazard on par with coastal California, though Hawaii is far from a tectonic plate boundary. Most of this seismicity is at least indirectly related to volcanism, which explains why 43 **M** > 6 earthquakes have occurred on the volcanically active island of Hawaii since 1868, whereas only 8 such events have occurred in the rest of the Hawaiian island chain (Figure 4-1) (Klein et al. 2001).

Earthquakes in Hawaii fall into several categories. Most are concentrated under the active volcanoes of Mauna Loa, Kilauea, and Hualalai and are related to the migration and intrusion of magma under and into the volcanoes. The migration of magma often causes small earthquakes to occur in shallow swarms, especially preceding an eruption. Hundreds of such earthquakes may occur in the days leading up to an eruption, but they rarely cause significant damage. Also, many of Hawaii's largest earthquakes are related to magma injection along the major rift zones that flank the active volcanoes. Intrusions of magma into the rifts introduce compressive stresses that are stored in the adjacent rock and periodically released in large earthquakes. These earthquakes are triggered when the seaward block flanking the rift zone slips laterally on a subhorizontal décollement away from the rift as it makes way for intruding magma and relieves the stored stress (Klein et al. 2001; EERI 2006). These décollements are typically 3 to 4 kilometers (km) deep and lie at the boundary between old oceanic crust and the relatively newly emplaced volcanic edifice (EERI 2006). Two of Hawaii's largest historical earthquakes, the 1975 **M** 7.2 Kalapana earthquake and the 1868 **M** 7.9 Kau district earthquake, both occurred as a result of such décollement slip (Figure 4-1) (Wyss 1988; Stover and Coffman 1993).

Another category of earthquake in Hawaii is indirectly related to volcanism; this category includes earthquakes that occur primarily in the upper mantle (i.e., 21 to 61 km deep). These earthquakes tend to occur in a ring surrounding the island and are probably caused by fracturing in response to lithospheric flexure under the weight of the overlying volcanic edifice (EERI 2006; Kirby and Klein 2006). Earthquakes of this type include the 1938 **M** 7 Maui, the 1973 **M** 6.2 Honomu, and the 2006 **M** 6.7 Kiholo Bay earthquakes (Figure 4-1).

The largest historical Hawaiian earthquake was the April 2, 1868, earthquake, which occurred in the Kau district, along Mauna Loa's southeastern flank (Figure 4-1); this earthquake had an estimated magnitude of **M** 7.9 (Stover and Coffman 1993). Wyss (1988) argued that rupture of a décollement under the south flanks of both Kilauea and Mauna Loa caused the earthquake. The earthquake caused extensive damage throughout the island of Hawaii, tearing houses off their foundations, knocking down masonry buildings, and triggering ground fissures, coastal subsidence of up to 2.4 meters (m), landslides, and a tsunami. The landslides killed 31 people, and the tsunami, which struck the Kau-Puna coast and rose as high as 15 m, killed 46 people. This event was preceded by an **M** 7 foreshock five days earlier that caused damage at Kahuku, Kona, and Waiohinu, and was followed by a large aftershock on April 4 (Stover and Coffman 1993).

Other large historical earthquakes in Hawaii include an **M** 6.8 earthquake in 1908 off the southeast coast of the island of Hawaii, the August 21 **M** 6.9 Kealakekua earthquake in 1951, and the November 29 **M** 7.2 Kalapana earthquake in 1975 (Figure 4-1). The Kealakekua earthquake was probably caused by rupture of the Kealakekua fault, with an epicenter just west of Kealakekua Bay on the Kona coast. It caused extensive damage on the Kona coast, including the collapse of stone walls, houses, and water tanks; the earthquake triggered landslides and disabled almost all seismograph stations, but caused no deaths (Stover and Coffman 1993). The 1975 Kalapana earthquake occurred along the southeast coast of Hawaii; this earthquake was accompanied by the rupture of the Hilina fault system, individual faults of which experienced surface displacements of over a 1 m. Deformation during the earthquake included horizontal displacement of the southern flank of Kilauea by as much as 8 m and coastal subsidence of up to 3.5 m (Cannon et al. 2001; Stover and Coffman 1993). The earthquake caused $4.1 million worth of damage and was accompanied by a tsunami that was locally as high as 14 m and killed two people. Both the Kealakekua and the Kalapana earthquakes were probably flank earthquakes, caused by slip on a décollement as the western flank of Mauna Loa and southern flank of Kilauea, respectively, slid seaward. Concurrent rupture and aftershocks occurred on steeply dipping normal faults, such as the Kealakekua and Hilina fault systems, at the head of the flank blocks. The rupture and aftershocks were reflected in the surface displacements observed on these structures (Cannon et al. 2001).

Although most large earthquakes in the state of Hawaii have occurred on the island of Hawaii, **M** 7 earthquakes occurred on Lanai on February 20, 1871, and north of Maui on January 23, 1938. The former earthquake was felt throughout the island chain, caused landslides and ground fissures, and damaged stone and adobe buildings in Lanai, Molokai, and Maui. The latter earthquake caused moderate damage (about $150,000) to roads, walls, tanks, and pipelines and triggered landslides throughout the island of Maui.

The October 15, 2006, **M** 6.7 Kiholo Bay earthquake was situated toward the western end of the Hualalai rift zone, about 16 km south-southwest of Kawaihae Harbor (Figure 4-1). It occurred on a north 14° west-striking fault dipping 76° down to the east at a depth of 39 km (Johnston et al. 2007).

4.1.2 Strong Ground Motion Recordings of the Kiholo Bay Earthquake

The 2006 Kiholo Bay earthquake provided the largest suite of strong motion records ever produced for an earthquake in the state of Hawaii and the best opportunity to understand the

processes of strong ground shaking on the island of Hawaii. The earthquake was recorded on 19 of the 22 current free-field strong motion instruments that the USGS operates on the island of Hawaii (Figure 4-2 and Table 4-1). Unfortunately, no strong motion stations exist at or in the vicinity of Kawaihae Harbor (Figure 4-2), so no local records were recovered from the October 15 earthquake sequence.

Table 4-1. List of USGS Hawaii Strong Motion Sites

Station No.	Island; City, Institution Location	Latitude (degree)	Longitude (degree)
2810	Hawai'i; Kailua-Kona, Fire Station	19.6477	-155.9923
2812	Hawai'i; Pahala, Ka'u Hospital	19.1999	-155.4723
2816	Hawai'i; Pahoa, Fire Station	19.4934	-154.9466
2817	Hawai'i; Hilo, University of Hawai'i	19.7034	-155.0805
2818	Hawai'i; Hilo, USDA Lab	19.7277	-155.0974
2822	Hawai'i; Waiohinu; Ka'u Baseyard	19.07	-155.615
2824	Hawai'i; Mauna Loa, Weather Observatory	19.5363	-155.577
2825	Hawai'i; Waimea, Fire Station	20.0230	-155.6614
2826	Hawai'i; Kapaau, North Kohala Police Station	20.300	-155.799
2829	Hawai'i; Mauna Kea State Park	19.752	-155.530
2830	Hawai'i; Mauna Kea Summit	19.826	-155.473
2832	Hawai'i; Honokaa, Police Station	20.0775	-155.4625
2833	Hawai'i; Laupahoehoe, Post Office	19.9835	-155.2326
2834	Hawai'i; Honomalino, Mac Farms	19.169	-155.868
2836	Hawai'i; Volcanic Nat'l Park, HVO Service	19.420	-155.288
2839	Hawai'i; Hilo, Old Hospital	19.722	-155.115
2845	Hawai'i; Honaunau, Post Office	19.4174	-155.8805
2846	Hawai'i; Mountain View, Post Office	19.5504	-155.1083
2847	Hawai'i; Anaehoomalu, Waikoloa Beach Hotel	19.919	-155.887
2849	Hawai'i; Kea Lakekua, Kona Hospital	19.5215	-155.9181
2852	Hawai'i; Kamuela, South Kohala Fire Station	19.9464	-155.8343
2853	NWS Data Regional Center	19.7174	-155.0494

Peak horizontal ground accelerations (PGAs) ranged up to 1.05 *g* as recorded at the Waimea station (Figures 4-2 and 4-3). Such a high PGA is surprising given the depth of the earthquake (39 km). The PGA at the North Kohala Police Station may also have exceeded 1 *g*. In contrast, the nearest strong motion site, at Waikoloa Beach Hotel, recorded only 0.19 *g* (Figure 4-2). In addition to the mainshock, a triggered event of **M** 6.0 on the same day (located at a shallower depth of 19 km) near Mahukona and an **M** 5.0 aftershock were recorded by the USGS Hawaiian

strong motion network (Figure 4-3). Eighteen records resulted from the **M** 6.0 event on the island of Hawaii, and 13 records resulted on the island from the **M** 5.0 aftershock.

To assess the level and nature of ground shaking at Kawaihae Harbor, an empirical ground motion predictive model is required. The models that the USGS used in the state hazard maps include Munson and Thurber (1997), Boore et al. (1997), Sadigh et al. (1997), and Campbell (1997) for the shallow earthquakes and Youngs et al. (1997) for the deep events (>20 km) (Klein et al. 2001). The USGS used the Youngs et al. (1997) relationship by default, given that no other model is available for the deep Hawaiian events. The relationship by Munson and Thurber (1997) is the only model that has been developed specifically from strong motion data of Hawaiian earthquakes (22 events, **M** 4.0 to 7.2, 51 PGA values), but the model is for shallow crustal events (< 20 km). The other models are appropriate for crustal earthquakes in tectonically active regions such as California.

4.1.3 SASW Surveys

To be able to utilize the strong motion data recorded by the USGS Hawaiian strong motion network, knowledge of the subsurface site conditions beneath the USGS stations was required. The subsurface geology and, more important, the shear-wave velocity (V_S) structure beneath the USGS stations has been unknown to date. The information is invaluable to verify the appropriateness of the empirical ground motion attenuation models being used in the state hazard maps produced by USGS and in site-specific hazard analyses for engineering design.

To obtain V_S information at Kawaihae Harbor and beneath the USGS strong motion sites, Spectral Analysis of Surface Waves (SASW) surveys were performed by the University of Texas, Austin, and URS Corporation in January 2008 (Wong *et al*. 2008) (Figures 4-4 and 4-5). The SASW technique has been used to obtain V_S profiles at other USGS strong motion sites (e.g., Seattle, the Imperial Valley, and Los Angeles), and this technique has been well validated against other approaches, such as down-hole surveys (e.g., Wong and Silva 2006). The technique has been particularly useful in volcanic regimes where interbedded volcanic sequences can result in low-velocity zones (e.g., Yucca Mountain and Los Alamos).

The SASW methodology is a non-destructive and non-intrusive seismic method. It utilizes the dispersive nature of Rayleigh-type surface waves propagating through a layered material to estimate the shear-wave velocity profile of the material (Stokoe et al. 1994; Joh 1996). In this context, dispersion arises when surface-wave velocity varies with wavelength or frequency. Dispersion in surface-wave velocity arises from the changing stiffness properties of the soil and rock layers with depth. Spectral analysis is used to separate the waves by frequency and wavelength to determine the experimental ("field") dispersion curve for the site. An analytical procedure is then used to theoretically match the field dispersion curve with a one-dimensional layered system of varying layer stiffnesses and thicknesses. The one-dimensional V_S profile that generates a dispersion curve that matches the field dispersion curve is presented as the profile at the site.

SASW measurements involve generating surface waves at one point on the ground surface and recording them as they pass by two or more locations. All measurement points are arranged along a single radial path from the source (Figure 4-5). Successively longer spacings between the receivers and between the source and first receiver are typically used to measure progressively longer wavelengths. The distance between the source and first receiver (d) is kept equal to the

distance between receivers. Measurements are performed with several (typically seven or more) sets of source-receiver spacings. Phase plots from surface wave propagation between the receivers are recorded for each receiver spacing. From each phase plot, the phase velocity of the surface wave can be calculated at each frequency from:

$$V_R = f \cdot \frac{360}{\phi} \cdot d$$

where V_R is the phase velocity in feet per second (ft/sec) or meters per second (m/s), f is the frequency in Hertz (cycles per sec), ϕ is the phase angle in degrees (at frequency f), and d is the distance between the receivers in the same length units as used to represent V_R. From this calculation, a plot of phase velocity versus frequency, called an individual dispersion curve, is generated.

This procedure is repeated for all source-receiver spacings used at the site and typically involves significant overlapping in the dispersion data between adjacent receiver sets. The individual dispersion curves from all receiver spacings are combined into a single composite dispersion curve called the experimental or field dispersion curve. Once the composite dispersion curve is generated for the site, an iterative forward modeling procedure is used to create a theoretical dispersion curve to match this experimental curve. The stiffness profile that provides the best match to the experimental dispersion curve is presented as the shear-wave velocity profile at the site.

An active seismic source is required for the SASW surveys. In these surveys, one of the NSF's Network for Earthquake Engineering Simulation (NEES) mobile vibrators, known as "Thumper," was used (Figure 4-4). Thumper has been designed to be a moderate- to high-frequency vibrator for use in seismic reflection and surface wave projects. Thumper is housed on a small vehicle, which aids in its transportation to and from sites and also allows it to be used in urban environments such as for the sites in Hilo. Some important characteristics of Thumper are: it is mounted on Ford F650 truck; the total weight is about 9,900 kilograms (kg); and three vibration orientations (field transformable in a few hours): vertical, transverse, or longitudinal. These characteristics make Thumper an excellent vibrational source for shallow (depths less than 100 m) surface wave testing. The maximum force output is about 27 kilonewtons (kN) over the frequency range of 17 to 225 Hertz. The force output decreases outside of this frequency band. The relatively low-force output (27 kN) made Thumper an excellent vibrator for testing in urban environments, where disturbance or possibly damage to existing aboveground and belowground facilities could occur.

The surveys took place from January 7 to 17, 2008. The 22 USGS strong motion sites surveyed are listed on Table 4-2 and are shown on Figure 4-2. Several surveys were also performed at Kawaihae Harbor (Figure 4-6). The results of the SASW surveys are shown in Appendix D2. A sample V_S profile for the North Kohala Police Station is shown on Figure 4-7. The high PGAs recorded at the Waimea Station and the North Kohala Police Station are probably due to thin soil site amplification where a strong velocity contrast exists between the soil and underlying basalt (Figure 4-7). Based on the survey results, all of the 22 USGS strong motion sites are "soil" sites (Table 4-2) with V_S30 values ranging from 442 ft/sec at the USDA Laboratory in Hilo (National Earthquake Hazards Reduction Program [NEHRP] site class E) to 1,812 ft/sec at the South Kohala Fire Station (NEHRP C). Surprisingly, none of the strong motion sites had rock-like

V_S30 values, even sites where basalt outcropped at the surface, such as at the University of Hawaii at Hilo.

A total of nine SASW lines were surveyed at Kawaihae Harbor (Figure 4-8). The results of the SASW surveys indicate fill and soil thickness of 40 to 90 ft over basalt (Figure 4-8). The basalt V_S appears to be about 3,000 ft/sec. This jump in V_S probably results in amplification of short-period ground motions. The V_S30 average for the harbor is 1,345 ft/sec and thus the generalized NEHRP site class is C. However, the low blow count SPT data in the coralline soils and observed liquefaction confirm that the upper 30 to 50 ft are building code site class F.

Table 4-2. Site Characteristics of Island of Hawaii Strong Motion Stations

Station No.	Location	V_S30 (ft/sec)	NEHRP Site Class	Surficial Geology
2810	Kailua-Kona Fire Station	1609	C	Fill/Soil
2812	Ka'u Hospital, Pahala	1304	C	Soil
2816	Fire Station, Pahoa	1580	C	50 ft Soil/Rock
2817	University of Hawai`i, Hilo	1595	C	70 ft/Soil/Rock
2818	USDA Lab, Hilo	442	E	Soil
2822	Ka'u Baseyard, Waiohinu	1325	C	Soil
2824	Mauna Loa Weather Observatory	1068	D	Soil
2825	Fire Station, Waimea	1465?	C?	40 ft Soil/Rock
2826	Kapaau Police Station, Kohala	947	D	120 ft Soil/Rock
2829	Mauna Kea State Park	1133	C/D	140 ft Soil/Rock
2830	Mauna Kea Summit	1092	D	80 ft Soil/Basalt
2832	Honokaa Police Station	1214	C/D	90 ft Soil/Rock
2833	Laupahoehoe, Post Office	999	D	Soil
2834	Mac Farms, Honomalino	1007	D	120 ft Soil/Rock
2836	HVO Volcanic Nat'l Park	844	D	Soil
2839	Old Hospital, Hilo	1462	C	100 ft Soil/Rock
2845	Honaunau Post Office	1506	C	Fill/Soil
2846	Mountain View Post Office	1197	C/D	Fill/Soil
2847	Waikoloa Marriott Hotel, Anaehoomalu	1594	C	Soil
2849	Kona Hospital, Kea Lakekua	1431	C	90 ft Soil/Rock
2852	South Kohala Fire Station Kamuela	1812	C	100 ft Fill and Soil/Rock
2853	NWS Data Regional Center, Hilo	1135	C/D	Soil

4.1.4 Evaluation of Strong Motion Data

Figure 4-9 shows the recorded PGA values as a function of NEHRP site categories for the mainshock and the attenuation relationship of Youngs et al. (1997). (Appendix D1 provides other comparisons between the recorded PGA values for the mainshock and the Mahukona event and the attenuation relationships of Youngs et al. [1997], Munson and Thurber [1997], Abrahamson and Silva [1997; 2007], and Boore and Atkinson [2006] [an update of Boore et al. 1997].) We have plotted the relationship for soil. The Youngs et al. (1997) relationship for intraslab events appears to significantly underestimate the PGAs for the mainshock (Figure 4-9).

Based on the pattern of mainshock ground motions (Figure 4-2), the V_S profile at Kawaihae Harbor, and the PGA values versus the Youngs et al. (1997) relationship, the URS project team estimates that the ground shaking at Kawaihae Harbor during the 2006 Kiholo Bay earthquake was characterized by a PGA of about 0.6 ± 0.3 g. This estimate represents a relatively broad range that reflects a large uncertainty. The estimate still represents a significant level of ground shaking (Figure 4-2) and is certainly capable of generating the observed liquefaction at the harbor. The hypocentral distance to the harbor is about 45 km. The uncertainty in the PGA estimate is large because what is needed is a site-specific ground motion attenuation relationship for the event, which is beyond the scope of this study.

The current building code for the island of Hawaii is still the 1991 Uniform Building Code (UBC) with Seismic Zone 4 assigned to the island, which has an effective PGA of 0.4 *g*. For a 2,475-year return period, the USGS 2002 National Hazard Maps indicate firm rock PGAs ranging from 0.5 *g* to about 1.0 *g* for the northwestern portion of the island of Hawaii (Klein et al. 2001). This range is similar to the range of PGA values observed in the mainshock (Figure 4-2). The maps are for NEHRP site class B/C (firm rock) and a V_S30 of 760 m/sec (2,500 ft/sec). The observed PGAs in the 2006 earthquake, when adjusted for site response effects, are consistent with the USGS National Hazard map for the island of Hawaii. It would seem prudent that the use of the 1991 UBC seismic provisions be revisited.

4.2 OTHER SEISMIC AND GEOLOGIC HAZARDS

4.2.1 Coralline Liquefaction and Lateral Spread

Liquefaction is a soil behavior phenomenon where shear strength loss occurs due to the rapid build-up of excess pore-water pressure, which reduces effective stresses in the soil to zero. Liquefaction is most commonly generated by strong earthquake ground shaking but can also occur as a result of cyclic loading conditions generated by ocean waves or vibratory construction equipment. In general, soils most susceptible to liquefaction are loose, saturated, uniformly graded sands containing few or no fines, such as dredged fills used to construct reclaimed landside areas of the harbor. The simplified method of evaluating liquefaction potential required by current building codes and design standards has evolved since the time of the construction of Kawaihae Harbor (Youd et al. 2001; Seed et al. 2003; Idriss and Boulanger 2004). Also, coralline soils exhibit unique liquefaction behaviors, and these behaviors require judgment when applying a method that was developed mainly for quartz sands (Mejia and Yeung 1995; Morioka and Nicholson 2000; Nicholson 2006).

Other hazards associated with liquefaction failure are flow failure and cyclic mobility, both of which manifest as lateral spreads. Flow failure occurs when the shear stresses required to maintain the static equilibrium of a slope or bank are greater than the shear strength of the soil in its liquefied state. Cyclic mobility occurs on gently sloping or relatively flat ground and differs from flow failure in that the static shear stresses are less than the shear strength of the soil in its liquefied state. The current simplified methods of estimating lateral spread have also evolved since the construction of Kawaihae Harbor (Youd et al. 2002).

An evaluation of liquefaction potential and lateral spread using current methods was performed with existing boring logs for comparison with observed performance. The results of both evaluations are discussed in Section 6. More rigorous analysis of the dynamic stability of the wharf foundations for incorporating the time history loads and nonlinear soil-structure interaction behaviors of port structures and buildings for liquefied conditions is beyond the scope of this study, but is required by current codes and standards (i.e., United Facilities Criteria [UFC] 4-152-01; Marine Oil Terminal Engineering and Maintenance Standards [MOTEMS] 2003; and IBC 2006). An example of a more rigorous pseudo-static analysis of non-linear soil-structure interaction and stress distributions is provided in Section 7 for Pier 1, which suffered the most damage during the Kiholo Bay earthquake.

4.2.2 Other Geologic Hazards

Kawaihae Harbor is subject to varying degrees of risk from other geologic hazards associated with the environment of the island of Hawaii. The proximity of the site, which is adjacent to Kohala Mountain, the most dormant of the island's volcanoes, contrasts with Mauna Kea to the east and Hualalai to the south, where eruptions in 1859 and 1801, respectively, produced lava flows that damaged coastal villages 10 to 15 miles south of Kawaihae. The existing lava flows that constitute the local bedrock geology are also prone in general to the potential subsidence of shallow lava tubes, submarine fractures, or landslides, though none are known to exist at Kawaihae. Also, no evidence of faulting exists at Kawaihae, though local hidden features associated with the Kohala rift zone (10 miles north) may exist. Most of the identified submarine faults on the east coast of the island of Hawaii have been mapped south of Kohala Bay and the location of the event.

In general the alluvial fan and estuarine soils and corals at Kawaihae (Figure 3-2) tend to obscure such hazards, if present immediately at the site. The coral reef also poses potential geologic hazards, including subsidence from solution cavities or tidal wash migration of soil particles and slope fractures, though the formation appears largely intact since original harbor construction in the 1950s.

Tsunami hazards are also significant at Kawaihae, with two measurable events occurring since the October 2006 earthquake, and more than 20 damaging historical events affecting the island of Hawaii in the past century. These events have produced run-up elevations of up to 12 feet at Kawaihae (MacDonald et al. 1983). The location of the harbor within the relief of Ulopu Point to the north, Keahole Point to the south, and within the shadow of Molokai, Maui, and Lanai may deflect energy from tsunamis originating to the north or northwest. Thus, location may account for the significantly higher run-up heights experienced on the Kohala coast and Hilo. However, the northwest exposure of the harbor entrance remains prone to strong winter waves, a potential direct attack from tsunami sources directly west or southwest of the harbor, or local landslides

such as the Alika slide south of Kiholo Bay, which is believed to be responsible for the tsunami deposits found on Lanai (Dudley and Lee 1998). Therefore, in general the harbor is considered to have the potential for large tsunami hazards.

SECTION FIVE EMERGENCY RESPONSE MEASURES FOR KAWAIHAE HARBOR FACILITIES

After the October 15, 2006, seismic event, DOT Harbors took the following emergency measures:

- Immediate closure of operations
- Reconnaissance and inspections
- Temporary mooring anchors at Pier 1
- Fill and pavement placement in areas of subsidence
- Reopening of operations
- Continuing interim restrictions
- Coordination with FEMA Public Assistance (PA) program

These measures are described in more detail below. The discussion is preceded by a detailed list of the sequence of events.

5.1 IMMEDIATE CLOSURE AND SEQUENCE OF EVENTS

October 15, 2006 (Sunday)

- A magnitude 6.7 earthquake struck the northwest coast of the island of Hawaii at approximately 7:07 A.M.
- Immediately after the earthquake, Harbor personnel contacted the State DOT Harbors Division via the 24-hour manned Harbor Traffic Control office, which initiated division-wide and statewide government notifications within minutes.
- The Marine Cargo Specialist at Kawaihae for the DOT Harbors Division, Hawaii District, performed the first inspection of Kawaihae Harbor at 8 A.M.
- The Terminal Supervisor of the Petroleum Facilities inspected the Tank Farm.
- Harbor personnel inspected utilities. During the inspection, two fire line leaks, three domestic line leaks, and one damaged light fixture were noted. Leaking air relief valves were also noted under Pier 2.
- The Captain of the Port closed Piers 1, 2A, and 2B in accordance with Title 33 Code of Federal Regulations Part 6.14. All the work described here was done on an emergency basis to bring Kawaihae Harbor back into service. It was determined that severe economic conditions would result if the port were not put back into service as soon as possible. Ships were temporarily rerouted to Hilo Harbor.
- The District Manager met with the Coast Guard and a Marine Safety Officer Representative on-site at Kawaihae Harbor within hours of the event while the State Civil Defense coordinated with the Governor's office and FEMA. Kawaihae Harbor was closed by the Coast Guard based on an initial inspection during this on-site joint meeting.

Ships were rerouted to Hilo Harbor while a more detailed inspection commenced, engaging local consultants for structural geotechnical, and dive inspections.

- DOT Harbors engaged the following consultants to assist with the emergency investigation:
  - Structural engineers: Miyasato, Kuniyoshi Engineers, LLC (Miyasato-Kuniyoshi Engineers)
  - Certified dive team: Sea Engineering, Inc.
  - Geotechnical engineers: Geolabs, Inc.

October 16, 2006 (Monday)

- A state and FEMA support team with the engineering inspectors arrived via a Coast Guard C-130 out of Barbers Point Naval Air Station, Oahu.
- Team members met with representatives of harbor users Matson and Young Brothers.
- The team members started surveying earthquake damage to the harbor and conducted dive investigations of Pier 2 into the night in advance of pending forecasted inclement weather.

October 17, 2006 (Tuesday)

- DOT Harbors, FEMA, and the Coast Guard each conducted an inspection. The team completed its inspection of Pier 2. Dive team members also conducted underwater investigations of Pier 1. The engineers determined that Piers 2A and 2B could be used for operations, with some limitations on the use of Pier 2A.
- Yamada Construction was engaged to fill cracks in the pavement at Piers 1, 2A, and 2B to stabilize voids and mitigate tripping hazards.
- Yamada Construction also performed pavement repairs at Piers 2A and 2B to enable the heavy equipment traversing the piers to make a smooth transition from the pier to the asphalt pavement. Yamada Construction worked through the night and into the morning hours of October 18, 2006.

October 18, 2006 (Wednesday)

- Pier 2B was reopened by the State and the Coast Guard, and the port users convened with government officials to discuss interim operations restrictions. The first research inspections were made by the Harbor's insurance company, the USACE, and the University of Hawaii Earthquake Engineering Research Institute (University of Hawaii 2006; EERI 2007). Repairs to pavements and utilities continued for a week, including installation of a new temporary mooring anchor at Pier 1.

October 19–25, 2006

- Initial emergency repair work was undertaken on pavements, utilities, and an emergency temporary mooring anchor at Pier 1 to prepare the terminal yard for limited use. Structural and underwater inspections continued. Initial repairs were substantially complete by October 25, 2006, when Pier 1 was able to offload cement with special mooring and continuous tugboat guidance.

5.2 RECONNAISSANCE AND INSPECTIONS

The State of Hawaii Governor's office issued an immediate request for a federal disaster declaration on the day of the event, which was granted by the President. The disaster declaration allowed FEMA to begin mobilizing equipment and personnel the same day. The first team arrived within 24 hours of the event and began to establish a field office in nearby Waimea for island-wide support. Also, an academic research team from the University of Hawaii that was sponsored by ASCE and EERI was organized. This research team also arrived on the island of Hawaii the day after the event to assist with damage assessments throughout the island, including Kawaihae Harbor buildings piers and utilities. DOT Harbors granted the research team access to Kawaihae Harbor while it was developing temporary damage repair provisions. The academic research team posted its observations online within one week of the event. These observations generally agree with the findings provided herein (University of Hawaii 2006).

The URS project team visited the site December 5, 2006, to perform the damage observations described in Section 7. A debriefing meeting with DOT Harbors and FEMA personnel was held December 6, 2006, in Honolulu at the DOT Harbors complex, to report on observations and the planned study. Additional records were obtained from DOT for the study.

Miyasato-Kuniyoshi Engineers provided DOT Harbors with an emergency structural assessment report (Miyasato 2006). The report provided immediate repair recommendations and conclusions for various pier structures. The immediate repairs included filling in voids in the asphalt pavement behind the piers and installing a temporary mooring device at Pier 1. After the immediate repairs were completed, Miyasato-Kuniyoshi Engineers provided the following recommendations regarding the use of the various pier structures until permanent repairs could be made:

- Pier 1: Permitted temporary use of Pier 1 for cement offloading only
- Pier 1 North Building: Restricted to temporary access only
- Pier 1 South Building: Continued use of building was permitted
- Pier 2A: Continued use of the pier was permitted, except vehicular traffic was not permitted between bents 13 and 14
- Pier 2A Transit Shed: Continued use of building was permitted
- Pier 2B: Continued use of the pier was permitted
- Brewer Building: Restricted to temporary access only

The recommendation for permanent repairs of various pier structures noted in the Miyasato-Kuniyoshi Engineers emergency structural assessment report (Miyasato 2006) had not been repaired or were not visible during the December 5, 2006, site visit by the URS project team.

The observations that the URS project team made during their site visits are discussed in this section and are limited to visible distress that were, in the opinion of the team, caused by the October 15, 2006, Kiholo Bay earthquake. Typical concrete distress such as cracks, spalls, and corroded reinforcing bars are not discussed in this report and are covered under the Public Assistance project worksheet reports.

5.3 TEMPORARY MOORING ANCHORS AT PIER 1

After the immediate evaluation of Pier 1 on October 15, it was determined that the use of the pier to moor ships was not recommended, and the pier was closed indefinitely. However, temporary mooring cables were installed at Pier 1 on October 20, 2006, to facilitate the offloading of cement products until the pier could be further evaluated. The temporary mooring cables were anchored to a large beam, placed in an excavation and backfilled with several feet of cover. This limited use, which does not involve the wharf structure, will likely continue until alternative cement material piping can be routed to a new off-loading area at one of the other piers.

5.4 FILL AND PAVEMENT PLACEMENT IN AREAS OF SUBSIDENCE

The voids behind the piers due to the soil loss caused by liquefaction were filled by Yamada Construction with low-strength concrete and repaved. New asphalt ramps were installed at Piers 1, 2A, and 2B where soil had settled behind the piers. Appendix A1 provides photos that show the filling of the voids and the asphalt repairs.

5.5 REOPENING OF OPERATIONS

The general harbor facilities were back in operation to approximately 50 percent capacity (Pier 2A) within two days of the seismic event. However, Pier 1 was closed indefinitely, except to cement offloading that does apply load to the wharf structure. DOT Harbors worked with the Coast Guard to install temporary buried mooring anchors behind Pier 1 on October 25, 2006, to allow cement to be offloaded in the harbor. By that date, the cement silos were empty and the West Hawaii construction industry would have been severely impacted.

Operations and storage areas were reconfigured around the operable Piers 2A and 2B with modified shipping to Hilo Harbor to accommodate the reduced wharf area. The economic impacts to Kawaihae Harbor were minimal because the harbor was operated at reduced capacity for a short period.

5.6 CONTINUING INTERIM RESTRICTIONS

The only continuing interim restrictions are at Pier 1, where the pier remains closed, except to cement offloading.

5.7 COORDINATION WITH FEMA PUBLIC ASSISTANCE PROGRAM

The FEMA Public Assistance (PA) Program for island-wide support was established at nearby Waimea, and became fully staffed within a week. PA program emergency response officials and professional engineers serving as technical assistance contractors (TACs) worked with DOT personnel to assess the scope of earthquake damage to Kawaihae Harbor and develop repair construction costs. The damage and costs are documented in the Project Worksheets for Kawaihae Harbor and several hundred other sites associated with this disaster. The PA program TAC responsible for preparing the Kawaihae Project Worksheets escorted our field team on the site visits, which occurred several weeks after the event. The PA program provided access to areas of damage and shared the Project Worksheet data available at the time, including photographs taken immediately after the event, preliminary assessments by an insurer's engineer, and details regarding the scope of damage identified at that time.

The Project Worksheets for Kawaihae were substantially completed in February 2007, at which time the full PA program demobilized, leaving a coordinator on-island to follow up on minor issues. Relevant technical information from the Project Worksheets for Kawaihae includes a dive survey and structural assessment of the wharf structures that were performed within a week of the event, a geotechnical investigation with post-event borings, and a comparative liquefaction assessment performed in January 2007. This information from the PA program is provided as Appendices C3, C4, and C5 in Appendix C (Geotechnical Data). This information also informs our discussion of observed damage (Section 6). The individual findings in the Project Worksheets for Kawaihae are discussed in Section 8.

SECTION SIX OBSERVED DAMAGE TO KAWAIHAE HARBOR FACILITIES

6.1 WHARVES AND MOORING DOLPHINS

6.1.1 Pier 1

Above Deck

- **Observations by Miyasato-Kuniyoshi Engineers:** A separation had opened in the ground along the joint between the concrete pavement and the asphalt concrete pavement (for the entire length of the pier), directly over the deadman wall location. The width of the separation varied from 2 inches at the south end to over 12 inches in the riprap at the north end, but some of the wider separations probably resulted when the asphalt concrete pavement and soil crumbled into the void. The maximum separation width within the bulkhead structure was approximately 5 inches, and the depth of the void varied from 3 feet to 4 feet, though this depth was obscured somewhat by the excavation and immediate pavement repairs behind the bulkhead slab. At the bulkhead, a separation gap had occurred between the concrete pavement and the concrete sheet pile cap. The gap was approximately ½ inch wide at the north section of the pier and 1 inch wide at the center section (where the pavement had not been repaired). No gap was present at the south section, where the concrete pavement had recently been replaced. Some repairs had been made to the tiebacks at the south section when the concrete pavement in this area was replaced. No continuous parallel surface rupture, buckling, or significant crack along the concrete pavement outboard of the deadman was noticed that would suggest severe passive soil failure fronting the deadman.

  At the southern end of the pier, beyond the concrete pavement, the bulkhead consists of cantilevered steel sheet piles with a concrete pile cap. At this section of the pier (from the south end of the concrete pavement to where the bulkhead intersects the adjacent drainage canal), an approximately 1-inch separation gap was visible between the bulkhead pile cap and the adjacent pavement.

- **Observations by URS Project Team:** The only visible distress to the pier structure that the URS project team observed as a result of the seismic event was visible lateral (seaward) movement of the rear anchor block. The amount of lateral movement appeared to be between 4 and 6 inches. A 1-inch separation was observed between the slab and the front concrete beam. The front concrete beam had numerous cracks and spalls along its entire length. The cracks and spalls appeared to be a result of large mooring and berthing forces on the wall. The URS project team was unable to determine if any of these cracks were new as a result of the rotation of the concrete sheet piles during the seismic event. The amount of movement or the amount of rotation of the bulkhead wall could not be determined.

Tieback Anchors and Sheet Piles

- **Observations by Miyasato-Kuniyoshi Engineers:** An underwater investigation of the sheet piles at the bulkhead was performed on October 17, but the heavy rains of the preceding night caused poor visibility and the condition of the bulkhead structure could not be confirmed. During a subsequent underwater investigation of Pier 1 on October 19, a crack of approximately ¾ inch in width was noted in one concrete sheet pile for the full width of the pile at mid-height. Two spalls, one at the ¾-inch crack and one near the mud line of another sheet pile, were also observed. At the center of the pier, a "large talus cone of gravel" was observed at the base of the sheet piles. Some soil materials had seeped through gaps in the sheet pile bulkhead. Measurements of the vertical face of the pile cap at the bulkhead indicated a slope of 1 percent to 2.5 percent from vertical leaning toward the ocean.

  Based on these observations, Miyasato-Kuniyoshi Engineers came to the conclusion that the entire pier structural system, including the bulkhead wall, pavement, and deadman, displaced outboard at least a few inches at the surface, as evidenced by the observed separation/void along the entire deadman wall. Also, the higher lateral pressures associated with the earthquake loading could have caused the horizontal crack and spalls in the concrete sheet pile noted in the underwater investigation and may have resulted in yielding/fracture of the tiebacks.

- **Observations by URS Project Team:** The URS project team did not perform an underwater survey of the sheet piles and will rely on the observations and assessments that Miyasato-Kuniyoshi Engineers and Sea Engineering made during their site visits.

During mid-December 2006, DOT Harbors personnel exposed the anchor blocks and tie rods behind the bulkhead at Pier 1. DOT and FEMA PA personnel reported to the URS project team that the connection to the sheet pile bulkhead yielded, and the concrete encasement around the anchor system appears to have helped reduce corrosion and protected the tie rods, reducing deformations. The tie rods also yielded but did not break, which is consistent with the site conditions that the URS project team observed.

Appendix A2 shows photos of the damage that the URS project team observed to Pier 1 bulkhead.

6.1.2 Pier 2A

Above Deck

- **Observations by Miyasato-Kuniyoshi Engineers:** Significant ground settlement occurred in the asphalt concrete pavement behind the wharf bulkhead. The settlement appeared to be fairly uniform along the entire length of Pier 2A; the asphalt concrete pavement was approximately 3 to 4 inches lower than the pile-supported concrete wharf deck, thus creating a step at that point. A minor gap of up to 1 inch occurred at this step at the portion of Pier 2A that is south of the existing adjacent slab.

- **Observations by URS Project Team:** The URS project team did not observe any visible distress or visible structural damage to the concrete wharf deck as a result of the seismic

event. The wharf structure appeared to have moved slightly (approximately 1 inch) seaward.

Below Deck

- **Observations by Miyasato-Kuniyoshi Engineers:** On the underside of the concrete wharf deck, extensive spalling to the slab soffit, concrete beams and struts, and pile caps was noted throughout the pier. These conditions appeared to be the result of the long-term corrosion of the embedded reinforcing steel rather than earthquake damage, but the earthquake likely shook off pre-existing delaminated concrete cover to expose additional reinforcing steel, thus creating more open spalls. The divers noticed some newer spalled concrete sections at the bottom of the harbor, below the pier. These newly created open spalls expose the concrete wharf reinforcing and could cause accelerated corrosion.

  Some horizontal cracks were noted at the outboard side of the tops of some of the piles. The cracks were less than ⅛ inch wide and located at random piles, and are likely to be flexural cracks that were created by the lateral displacement of the wharf superstructure during the earthquake. These cracks did not appear to be an immediate structural concern, but are a long-term durability concern.

- **Observations by URS Project Team:** The URS project team did not perform an underwater survey of the pier and will relying on the observations and assessments that Miyasato-Kuniyoshi Engineers and Sea Engineering made during their site visits.

Appendix A3 shows photos of the damage that the URS project team observed on the above deck portion of Pier 2A and photos that FEMA staff took of the underside of the deck.

6.1.3 Pier 2B

Above Deck

- **Observations by Miyasato-Kuniyoshi Engineers:** Minor ground settlement occurred in the asphalt concrete pavement behind the wharf bulkhead. The settlement appears to be essentially uniform and is indicated by minor cracks in the pavement along most of the length of the pier and water ponding in this area. No signs of structural distress of the concrete wharf structure from earthquake loading were noticed.

- **Observations by URS Project Team:** The URS project team did not observe any visible evidence of lateral movement of the wharf structure or any distress or visible structural damage to the concrete wharf deck as a result of the seismic event.

Below Deck

- **Observations by Miyasato-Kuniyoshi Engineers:** No visible defects in the piles or bulkhead were observed below the water. Minor spalling was observed above the water on the lower outboard face of the longitudinal beam in the D-pile row (center row). At each of these locations, corroded reinforcing steel was exposed within the spalled area, indicating that the damage was a result of corrosion of the reinforcing steel and not a

result of the earthquake. No visible defects were observed in the piles or bulkhead below the water.

- **Observations by URS Project Team:** The URS project team did not perform an underwater survey and is relying on the observations and assessments that Miyasato-Kuniyoshi Engineers and Sea Engineering made during their site visits.

Appendix A4 shows photos of the damage that the URS project team observed to the above deck portion of Pier 2B.

6.1.4 Mooring Dolphins

- **Observations by Miyasato-Kuniyoshi Engineers**: A condition assessment of the mooring dolphins was not included in the Miyasato-Kuniyoshi Engineers report.
- **Observations by URS Project Team:** The URS project team performed an underwater inspection of the dolphins on July 17, 2007. The concrete on the smaller dolphin, which is supported by steel piles, showed some deterioration. Signs of rust were present on the concrete. The rust appears to be from the steel bollard mounted on top of the dolphin. The two larger dolphins appears to be newer and in good condition. The URS project team did not observe any visible signs of concrete spalls or corroded reinforcing on the two larger dolphins. The concrete and steel piles both above and below the water appeared to be in good condition, except damage to one of the concrete piles in the larger dolphin. This is reasonable given the dolphins were reported to be unmoored during the earthquake.

Appendix A5 shows photos of the above-water and below-water damage that the URS project team observed at the mooring dolphins.

6.2 WATERFRONT BUILDINGS

6.2.1 Pier 1: North and South Metal Buildings

The North and South Metal Buildings at Pier 1 sustained only limited structural damage due to the earthquake. Differential slab settlement from soil liquefaction was observed in both buildings. The URS project team also observed some racking of exterior doors in both buildings. Although the damage from these issues was minimal, it did limit access to and from the buildings.

The most significant structural damage occurred to a steel cross-brace tie rod that sustained pull-out failure at a turn-buckle connection. It appeared that this damage resulted from an insufficient amount of thread engagement. The North Metal Building has “racked” to some degree due to this failure.

Appendix A6 shows photos of the damage that the URS project team observed at the North Metal Building on Pier 1.

6.2.2 Pier 2A: Transit Shed

The transit shed on Pier 2A sustained only limited structural damage due to the earthquake. The URS project team observed that the asphalt pavement settled from 4 to 6 inches throughout the

building due to soil liquefaction. Evidence of a 1-inch seaward movement was observed around the columns at the north end of the building. Damage to the siding was also observed at the intersection of the office building and the main building.

Appendix A7 shows photos of the damage that the URS project team observed at the Transit Shed on Pier 2A.

6.2.3 Brewer Building

The east side of the Brewer Building is constructed with large openings between concrete columns. The openings between the columns were filled in with masonry and louvers. The only earthquake-related damage that the URS project team observed was severe cracking at the top of the concrete columns that surround the openings and the dislodging and falling out of a portion of the masonry in-fill wall at the northeast corner of the building.

Appendix A8 shows photos of the damage that the URS project team observed at the Brewer Building.

6.3 OTHER SITE FACILITIES

Except for pavement performance issues, no structural damage or distress was observed at any of the ancillary facilities or in their foundations after the earthquake, despite reports of strong shaking and violent oscillations of light poles that resulted in occasional minor cracking. Pavement performance correlated closely with the ground conditions discussed in Section 3.

Appendix A9 shows photos of URS project team observations at other site facilities.

6.4 GEOTECHNICAL HAZARDS

6.4.1 Coralline Liquefaction

Clear evidence of liquefaction was observed in the coralline fills within the Kawaihae Harbor terminal yard. This type of liquefaction is a relatively rare occurrence; the only other known case of similar coralline liquefaction occurred in Guam during the 1993 **M**.8.3 earthquake (EERI 1995; Mejia and Yeung 1995). The adoption of liquefaction evaluation requirements since UBC 1997 has prompted several researchers to investigate the comparative behavior of coralline soils in the Pacific and quartz sands primarily on the U.S. mainland. These investigations, which have occurred during the past decade, are complicated by widely varying engineering properties in corals, such as cementation, within a short distance (Nicholson 2006; Mejia and Yeung 1995). Examples of the unique liquefaction behavior of coral soils are provided in Appendix D3.

The observed liquefaction at Kawaihae Harbor provides a basis for comparing coralline liquefaction behavior with quartz sand liquefaction behavior. This analysis may improve reliability in predicting liquefaction risk at many coralline sites throughout the Pacific. Observations of liquefaction behavior at Kawaihae included identifying the locations of occurrence, magnitude of settlement, magnitude of lateral spread, and influence of liquefaction on structural performance, all of which are typical aspects of concern.

Observations by the University of Hawaii and EERI: These initial island wide reconnaissance reports documented significant evidence of liquefaction at Kawaihae Harbor terminal in the vicinities of Piers 1 and 2 and related pavements. They reported no other evidence of liquefaction at other sites investigated during reconnaissance (EERI 2006; University of Hawaii 2006). This was confirmed by conversation with the lead geotechnical investigator, acknowledging the recon efforts were oriented at areas of reported infrastructure and building damage, rather than for fundamental liquefaction research purposes, which could have prompted closer examination of saturated natural beach sand deposits along the west coast of Hawaii, for example (Nicholson 2007).

Observations by URS Project Team: The URS project team observed liquefaction evidence reported by the reconnaissance report largely intact, including settlement and cracking of asphalt pavements, separations at foundations and pier deck transitions to asphalt pavements, and sand boil residue expunged from earthquake-induced asphalt cracks. The settlements occurred in areas of dredged fill within the paved areas of the main terminal yard, generally in the vicinity of the wharves and waterfront structures, including pavements within the structures. The observed settlements ranged up to 7 inches, with an average settlement in area of visible distress on the order of 2 to 3 inches. Sand ejecta primarily consisted of fine sands, with some intermixed small fractions of base course. Many areas have swale-like localized depressions that display variability in crack patterns, which is indicative of variable subsurface conditions.

Cracking patterns in the pavements extended broadly over about half of the terminal yard area, with the most significant settlements occurring around the buildings at Pier 1 and Pier 2A, the yard located between them, and the pavement transitions to concrete decks at Piers 2A and 2B. The longer, more predominant cracks, some of which were over 100 feet in length, were observed in parallel to the wharf and bulkhead structures. These cracks were typical of movements in the offshore direction of the sloping harbor bottom and thicker fill depths. Crack widths ranged from about 1/16 of an inch to 2 inches, and averaged about ½ inch. (Appendix A).

Some of the observed liquefaction settlements may involve other factors, such as dynamic settlement of loose fills above the water table, lateral deformation of pier bulkhead walls, and/or migration of fines through tidal wash within the bulkheads (discussed below). Pavement joints adjacent to the wharves had already been patched at the time of our observations, and a land-based topographic survey of the terminal yard has not been performed since the earthquake. Therefore, an assessment of the distribution of liquefaction through the subsurface profile involves considerable uncertainty.

The URS project team performed a traditional liquefaction evaluation using the current simplified methods that building codes require (Idriss and Boulanger 2004; Youd et al. 2001). The analysis used selected borings to correlate density and blow-count depth distributions with observed settlements (Appendices D3 and D4). Observed vertical settlements from liquefaction settlements (<7 inches) appear to be of the same order of magnitude with the predicted settlements (<9 inches) using simplified liquefaction procedures with an assumed PGA of 0.4 *g*. This correlation suggests ground motions toward the mean to lower bound of those estimated from the evaluation of strong motion data and shear-wave velocity data in Section 4. The observed settlements are also consistent with the lack of significant change in topography or bathymetry revealed by post-event lidar survey and the lack of significant damage to Pier 2A and 2B piles observed in dive surveys.

However, our SPT-based findings suggest that the liquefaction settlements could have occurred in either saturated fills combined with dynamic settlements above the water table or saturated fills with natural corals predominating within the upper 20 feet, but not likely in both together. The former is more probable than the latter because of the likely loose condition and lesser cementation in dredged fills and the relative proximity of the observed settlements to the piers and sloping harbor surface.

In general, the observed settlement at Kawaihae Harbor with likely PGA values of over 0.3 *g* suggests that coral fill soils at Kawaihae may have liquefaction behavior that is similar to U.S. mainland quartz soils of similar density. However, using the available data, we are unable to confirm the depth at which the liquefaction occurred and whether the natural corals below the fill liquefied, as the correlation of SPT values to density of very loose weakly cemented corals and to seismic settlement of liquefied corals is not well developed. Very weakly cemented natural coral formations have shown discrepancies from quartz sands in both shear-wave velocity and cyclic behaviors (Nicholson 2006). This discrepancy remains evident from the Vs data in the coral soils at Kawaihae (presented in Section 4), which were generally greater than 1,000 ft/sec and would be found to not liquefy according to Vs methods of liquefaction potential assessment. Because the soils are known to have liquefied, the soil structure may have degraded, breaking weak cementation bonds during shaking. The resulting building code site classification for Kawaihae is thus F (liquefiable soils, SPT based), but the NEHRP site classification remains C, based on Vs measurements.

Unfortunately, the URS project team was not able to further investigate potential liquefaction at other saturated coral fill or natural (beach) sites along the west coast of Hawaii, though during our inspection of the beaches at the southern harbor shoreline and the outer portions of the harbor, we did not see evidence of liquefaction.

6.4.2 Lateral Spread

No observations of lateral spread were reported in the site reconnaissance reports, nor was the URS project team able to distinguish lateral spread from wharf structural displacements during the site visit. The amount of bulkhead movement appears to have been less than 4 inches at Piers 2A and 2B and less than 18 inches at Pier 1.

Differential lateral movements were observed along Pier 2A in the vicinity of the historical dike. Also, the area of the dike between Pier 1A and Pier 2A with a down-slope terrace ledge along the waterline would more closely represent typical conditions used to develop the lateral spread equations, though this area too appears to have remained largely intact. A portion of the original sheet pile cutoff within the dike at Pier 2A was constructed before the pier was observed to have separated, though barnacles within the area of separation suggest that the initial movements occurred before the earthquake. Observed nearshore bathymetry conditions after the earthquake included occasional sloughing and dislodged riprap or armor stone. Sea Engineering corroborated these conditions in its post-event dive survey (Appendix C3), as did the comparison of the pre-event bathymetry with the post-event lidar survey (Figures 3-3 to 3-5).

The most pronounced lateral displacements occurred north of Pier 1, where the shoreline consists of an ungrouted large riprap dike. Separations of armor stone greater than 1 foot were evident, as was cracking along the connection to Pier 1. Some tidal wash was evident between the stone and pier displacements. This evidence raises the question of the potential for loss of backfill and

increasing instability before the earthquake. The December 2006 bathymetry data indicated that a noticeable new slump feature has developed since 2003. This feature is located directly offshore of the riprap dike (Figure 3-4) and may be evidence of a lateral spread flow failure. Other areas of 2006 bathymetry along Piers 1, 2A, and 2B appear relatively unchanged since 2003, suggesting that the submarine lateral spread displacements were less than the degree of accuracy in the bathymetry maps (estimated at 5 feet).

These observed movements appear to be reasonably consistent with the predicted liquefaction-induced lateral spread, which is predicted to be approximately 1 foot using the standard simplified method with 0.4 *g* PGA (Youd, Hansen, and Bartlett 2002). A PGA higher than the 0.4 *g* assumed for the analysis would suggest non-classical lateral spread behavior, which is possible. The analysis for Kawaihae assumes a 1H:1V slope along the harbor (Appendices D3 and D4). The amount of spread tended to be less sensitive to potential PGA variations than to vertical settlements, given the relatively remote distance to the epicenter (20 miles). Because of proximity to the wharf structures, lateral spread magnitudes are strongly affected by the displacement resistance of the structures. One would expect the embedded structure to provide restraint to the fill soils, producing observed lateral spread magnitudes that are less than the calculated spread prediction, assuming a free field natural slope or cut. However, potential foundation instability and the resulting bulkhead slope instability may dominate the future localized deformations in the immediate vicinity of the wharf structures more than lateral spreading.

A discussion of the potential combined effects of liquefaction, lateral spread, and wharf structure instability at Pier 1 is provided with a simplified soil-structure interaction analysis in Section 7.

SECTION SEVEN FACILITY SEISMIC PERFORMANCE EVALUATION

This section summarizes the structural performance of the facilities at Kawaihae Harbor and the resulting damage. The performance evaluation is presented in terms of observed damage for pier structures designed to DOT standards and the Probable Maximum Loss (PML) criteria for waterfront structures designed to UBC criteria.

7.1 PERFORMANCE EVALUATION FOR WHARVES AND DOLPHINS

7.1.1 Pier 1

The visual damage that was observed to the bulkhead wall and surrounding structures indicated that the area around Pier 1 experienced significant movement. The bulkhead system, which includes the concrete bulkhead wall, tie rods, and anchor block, appears to have translated seaward approximately 4 to 6 inches. The north end of the pier appears to have displaced approximately 12 to 15 inches, as evidenced by the large crack in the rock revetment at the north end of the pier. Significant settlement was also observed behind the anchor block that supports the bulkhead. This settlement is probably due to the lateral movement of the pier and the liquefaction of the soils behind the anchor block. The only visible damage to the pier structure that the URS project team observed was the following:

- Significant settlement behind the anchor block
- Yielding of the tie rods, but no breakage (confirmed in post-event excavation)
- Large crack in the rock revetment along the north end of the pier

The URS project team concludes that the existing bulkhead system underwent significant movement. This movement resulted in the yielding of the existing tie rod system, and this result has reduced the ability of the system to resist additional soil pressure from another seismic event. For a system constructed in 1954, the system performed well enough to prevent a catastrophic failure of the wall and subsequent failure of the structures behind the wall. The measured displacement and the yielded but unbroken tie rods provided a basis on which to calibrate the earthquake loads induced to the structure. The simplified soil-structure interaction analysis is described below.

To evaluate the performance of Pier 1, the URS project team developed a two dimensional, non-linear soil-structure interaction model of the Pier 1 profile using Plaxis version 8.2 finite element (FE) software (Brinkgreve et al. 2006). The model geometry was developed using construction drawings, the harbor dredging plan, and recent bathymetric survey data. An interpreted subsurface profile was modeled from the soil layers and strength parameters presented in Tables 3-1 and 3-2, which are based on the available boring data in the area for hydraulic fill, natural corals, and sloping basalt bedrock. Mohr coulomb soil properties were used to model ductile yielding in both soil and structural elements. Liquefied soil conditions were modeled as selected layers with reduced shear strength behaving as undrained cohesion of approximately 200 to 600 pounds per square foot (psf) and a Young's modulus of approximately 50 kips per square foot (ksf). The structural elements modeled included the reinforced-concrete sheet piles, the deadman systems, and steel tie rod, using the elastic properties of each. The resulting model section with soil and structural elements is shown in Figure 7-1.

This conceptual model is not intended to provide a definitive analysis of the structural behavior but to illustrate the stress distributions within the system and the order of magnitude earthquake forces feasible to produce the observed displacements. No modeling of the liquefaction triggering mechanism, including a pore pressure generation scheme and cyclic strength degradation, was included, as this modeling would be beyond both the scope of the available subsurface data and the intended level of effort for this investigation.

The approach to the analysis involved using staged construction that was similar to the historical soil stresses and load paths. Model staging starts with the initial site conditions before the harbor was developed, then the hydraulic fill was placed, then the sheet pile system was inserted, and finally the harbor was dredged.

A stability analysis of the bulkhead was performed using the strength reduction module in Plaxis version 8.2, which is analogous to common limit equilibrium methods for deriving factors of safety (Brinkgreve et al, 2006). Three stability conditions were analyzed: initial static (pre-earthquake) stability, pseudo-static (earthquake) stability, and liquefied (post-shaking) conditions.

These modeled conditions were calibrated to compare with the observed maximum horizontal deformations of approximately up to 18 inches in the riprap and 5 inches in the bulkhead tie-back structure, and vertical deformations of up to 7 inches, without breakage of the tie rod. The resulting calculated factor of safety for static conditions was 1.4.

At an applied force of approximately 0.3*g*, the pseudo-static factor of safety fell below unity and produced large, unstable deformations. A stable condition was obtained with a force of 0.25*g*. This finding is reasonable in that it follows the common practice of assuming a pseudo-static acceleration coefficient of approximately 50 to 65 percent of the recorded peak ground acceleration, representing a more sustained average loading. Assuming this relationship holds true for actual attenuation in coralline soils with underlying basalts at Kawaihae (which are not well defined), a general estimate would be that PGA values were closer to 0.45*g* at Pier 1, falling between the 0.3*g* PGA minimum and the 0.6*g* PGA mean estimated by the seismicity study.

After removal of the pseudo-static load, post-liquefied soil strengths were applied in the loose soil deposits to simulate the final seaward displacement of the pier. The result was approximately 12 to 15 inches of horizontal deformation, which is more consistent with the observed movements in the riprap. The FE model mesh was tested for reliability by varying the soil properties and earthquake loads within the data range to verify reasonable model performance. The observed displacements of the bulkhead structure under stable conditions varied from 1.5 feet for a pseudo-static force of 0.25*g*, 0.5 feet at 0.2*g*, and 0.2 feet at 0.15*g*, suggesting the pseudo-static force producing the observed 5-inch bulkhead deformation was approximately 0.2*g*. This result also illustrates the relatively sensitive relationship between pseudo-static acceleration, soil strength, and deformation, and the possible near-collapse condition of the bulkhead. The deformed mesh under the initial 0.3*g* pseudo-static load is shown in Figure 7-2 to illustrate the simulated deformation during application of the seismic load. The relative stresses modeled during the post-liquefied condition that illustrate potential zones of liquefaction are given in Figure 7-3.

In summary, the URS project team estimates that the stability of Pier 1 after the earthquake was marginal and potentially near incipient total collapse, with incipient failure and 5 inches of deformation at the wharf under pseudo-static acceleration values near 0.2*g*, allowing for likely

occurrence of transient spikes at higher values. The observed deformations reasonably agree with model results using traditional residual post-liquefied soil strengths for assumed liquefaction limited to loose layers. It is likely that localized conditions of both loose liquefied zones and weakly cemented unliquefied zones exist in the highly variable coral deposits, though the model represents a possible condition using generalized soil stratigraphy.

7.1.2 Pier 2A

Based on the results of the URS project teams' inspection of the top side of the wharf and the results of the under-deck survey by Sea Engineering, Pier 2A wharf structure appears to have had good overall seismic performance. There was evidence of significant vertical movement of the ground behind the wharf, but little lateral movement of the wharf was observed by the URS project team. Relatively minor cracks and spalls in the top of the wharf structure were observed, but in the opinion of the URS project team these cracks and spalls are a result of long-term corrosion of the reinforcing steel, not a result of the seismic event.

The cracks in the piles that were noted in the under-deck survey that Sea Engineering performed appear to be flexural cracks. In the opinion of the URS project team, these cracks are a result of three things: (1) lateral displacements of the superstructure during the seismic event, (2) poor detailing, and (3) lack of ductility in the connections of the piles to the wharf deck.

Pier 2A was built in 1958; it was constructed using 18-inch-square conventionally reinforced vertical concrete piles. Constructing a wharf using all vertical piles was not the common method of wharf construction at that time. The vast majority of wharf structures built during that period were constructed using both vertical and battered piles. Currently, the use of battered piles in wharf structures is discouraged because of their poor performance in past seismic events. The fact that the wharf structure has only vertical piles is probably why the wharf performed as well as it did.

The performance of the lateral resisting system of a wharf depends on the magnitude of the event, the direction of the shock wave relative to the lateral resisting system, the configuration and quantity of the lateral resisting system, and the soil profile and properties. To determine the actual lateral capacity of the system would require an in-depth analysis of the wharf and soil, which was not within the scope of this report.

7.1.3 Pier 2B

Based on the results of the URS project team's inspection of the top side of the wharf and the results of the under-deck survey by Sea Engineering, Pier 2B wharf structure appears to have had good overall seismic performance. Little lateral movement of the wharf was observed. The URS project team observed relatively minor cracks and spalls in the top deck of the wharf structure, but concludes that these cracks and spalls are a result of the long-term corrosion of the reinforcing steel, not a result of the seismic event.

The under-deck survey that Sea Engineering performed revealed no signs of structural distress in any of the lateral or vertical resisting systems.

Pier 2B was built in 1990; it was constructed using vertical and battered 20-inch octagonal precast, prestressed piles. Although battered piles are currently discouraged because of their poor performance in past seismic events, this wharf appears to have performed well. A possible reason

for the wharf's performance is that the wharf was built using relatively current design procedures and detailing practices.

7.1.4 Mooring Dolphins

Based on the results of the URS project team's inspection of above water and underwater portions of the mooring dolphins, the mooring dolphins had good overall seismic performance. The URS project team observed relatively minor cracks and spalls in the dolphin structure, and damage to one of the concrete piles, but conclude that the cracks and spalls and the damaged pile do not appear to have been a result of the seismic event.

7.1.5 Conclusions

After reviewing the available data, it is the opinion of the URS project team that both of the pile-supported structures (Piers 2A and 2B) had good overall seismic performance, with very little damage to the lateral and vertical support systems. The majority of the damage observed on the wharves was as a result of the long-term corrosion of the reinforcing steel, not a result of the seismic event.

However, Pier 1 performed poorly. The seismic event caused significant lateral movements of the riprap wall system, cracks in the wall panels, and cracks in the pile cap. In the opinion of the URS project team, this damage was due to the fact that the bulkhead system was not designed to resist the higher lateral pressures associated with earthquake loading. Even with the increased lateral pressure, the wall system, though damaged to the point that it could no longer be used, did not fail completely.

7.2 PERFORMANCE EVALUATION FOR WATERFRONT BUILDINGS

The URS project team performed a PML analysis on the waterfront buildings at Kawaihae Harbor. PML is an evaluation tool used to estimate the expected damage/loss to a building for a given earthquake.

The PML analysis uses FEMA 310 *(Handbook for the Seismic Evaluation of Buildings)* (FEMA 1998). FEMA 310 identifies "weak links" in a structure by using a checklist of compliant and non-compliant statements regarding building features and conditions. The building characteristics based on the completed FEMA 310 checklists are quantified according to their potential to cause loss to the building. Each characteristic is assigned a modifier range reflecting how great an effect that characteristic would have on loss. These quantification algorithms are based on knowledge developed from previous earthquakes and data from thousands of building evaluations. Additional information is provided in Appendix E.

The probabilistic Modified Mercalli Intensity (MMI) estimate at Kawaihae Harbor for a risk level consistent with a 10 percent chance of exceedance in 50 years is IX-X. This value assumes stiff soil/bedrock soil conditions and does not include the effects of soil damping or amplification. Table 7-1 is provided as an aid to interpreting the level of building damage represented by the PML analysis:

The amount of earthquake-related damage to an individual building depends on a number of variables, including the intensity and duration of ground shaking, building configuration,

structural systems, the materials of construction, structural details, nonstructural components, and quality of construction. The PML methodology is a guideline for estimating seismic damage; however, the performance of any specific structure will generally deviate substantially from the average values given by these empirical loss functions.

Table 7-1. PML Analysis Damage Summary

PML Value	Damage State	Description of Expected Damage
0 to 1%	Slight	Limited localized minor damage not requiring repair.
1 to 10%	Light	Significant localized damage of some components generally not requiring repair.
10 to 30%	Moderate	Significant localized damage of many components warranting repair.
30 to 60%	Heavy	Extensive damage requiring major repairs.
60 to 100%	Major	Major widespread damage that may result in the facility being condemned, demolished, or repaired.

7.2.1 Pier 1: North Metal Building

The North Metal Building has seismic-related building characteristics similar to typical pre-engineered metal buildings constructed before 1970. Non-compliant FEMA 310 checklist items are primarily related to the lateral-force-resisting system (Table 7-2).

Table 7-2. North Metal Building: Non-Compliant FEMA 310 Checklist Items

Checklist Section	Deficiency
Lateral-Force-Resisting System Diagonals	The axial stress in the tension-only steel rod braces greatly exceeds the recommended maximum stress.
Lateral-Force-Resisting System Compact Members	The beam and column flanges of the moment frames are noncompact elements.
Connections Load Path at Pile Caps	The pile caps do not have top reinforcement to transfer uplift forces, if any, to the piles (i.e., not pile supported).

Table 7-3 provides a summary of the PML analysis results based on the completed FEMA 310 checklist:

Table 7-3. North Metal Building: Summary of Probable Maximum Loss Analysis Results Based on FEMA 310 Checklist

Probability of Exceedance in any 50-Year Period	Damage/Loss Estimates (As a Percentage of Replacement Cost)		
	Expected Loss	Probable Maximum Loss	Probable Loss
10%	11%	19%	15%

At this level of ground-shaking intensity, an expected, or mean, monetary loss of 11 percent is predicted. PML is estimated to be 19 percent, and probable loss is estimated to be 15 percent. These values correspond to a moderate level of expected earthquake-related damage and losses. Assuming a 40 percent minimum loss at abandonment and 3-month duration to reoccupy the facility after abandonment, no business interruption is expected for the 10 percent/50-year scenario.

7.2.2 Pier 1: South Metal Building

The South Metal Building has seismic-related building characteristics consistent with typical pre-engineered metal buildings constructed around 1970. Non-compliant FEMA 310 checklist items are primarily related to the lateral-force-resisting system (Table 7-4).

Table 7-4. South Metal Building: Non-Compliant FEMA 310 Checklist Items

Checklist Section	Deficiency
Lateral-Force-Resisting System Diagonals	The axial stress in the tension-only steel rod braces greatly exceeds the recommended maximum stress.
Lateral-Force-Resisting System Compact Members	The beam and column flanges of the moment frames are noncompact elements, which may experience premature local buckling and lead to poor inelastic behavior and ductility.
Lateral-Force-Resisting System Bottom Beam Flange Bracing	The bottom flanges of the moment frame beams are not braced out-of-plane to prevent lateral torsional buckling.

Table 7-5 provides a summary of the PML analysis results based on the completed FEMA 310 checklist:

Table 7-5. South Metal Building: Summary of Probable Maximum Loss Analysis Results Based on FEMA 310 Checklist

Probability of Exceedance in any 50-Year Period	Damage/Loss Estimates (As a Percentage of Replacement Cost)		
	Expected Loss	Probable Maximum Loss	Probable Loss
10%	12%	21%	16%

At this level of ground-shaking intensity, an expected, or mean, monetary loss (damage/replacement cost x 100) of 12 percent is predicted. PML (i.e., loss that has a 10 percent chance of being exceeded for a 475-year ground motion) is estimated to be 21 percent, and probable loss (i.e., the amount that a property is expected to meet or exceed on an average basis) is estimated to be 16 percent. These values correspond to a moderate level of expected earthquake-related damages and losses.

Assuming a 40 percent minimum loss at abandonment and 3-month duration to reoccupy the facility after abandonment, business interruption for the 10 percent/50-year scenario is estimated to average one month.

7.2.3 Pier 2A: Transit Shed

The Transit Shed has seismic-related building characteristics similar to typical pre-engineered metal buildings constructed before 1970. Non-compliant FEMA 310 checklist items are primarily related to the lateral-force-resisting system (Table 7-6).

Table 7-6. Transit Shed: Non-Compliant FEMA 310 Checklist Items

Checklist Section	Deficiency
Lateral-Force-Resisting System Diagonals	The axial stress in the tension-only steel rod braces greatly exceeds the recommended maximum stress.
Lateral-Force-Resisting System Compact Members	The beam and column flanges of the moment frames are noncompact elements.
Connections Load Path at Pile Caps	The pile caps do not have top reinforcement to transfer uplift forces, if any, to the piles (i.e., not pile supported).

Table 7-7 provides a summary of the PML analysis results based on the completed FEMA 310 checklist:

Table 7-7. Transit Shed: Summary of Probable Maximum Loss Analysis Results Based on FEMA 310 Checklist

Probability of Exceedance in any 50-Year Period	Damage/Loss Estimates (As a Percentage of Replacement Cost)		
	Expected Loss	Probable Maximum Loss	Probable Loss
10%	11%	19%	15%

At this level of ground-shaking intensity, an expected, or mean, monetary loss of 11 percent is predicted. PML is estimated to be 19 percent, and probable loss is estimated to be 15 percent. These values correspond to a moderate level of expected earthquake-related damage and losses. Assuming a 40 percent minimum loss at abandonment and 3-month duration to reoccupy the facility after abandonment, no business interruption is expected for the 10 percent/50-year scenario.

7.2.4 Brewer Building

A PML analysis was not performed on the Brewer Building since no construction drawings were available. Based on the construction type, relative age, and building configuration, the PML for the Brewer Building is expected to be between 20 and 40 percent for a 10 percent/50-year seismic event.

7.3 CONCLUSIONS

With the currently available information, the URS project team's observations of the seismic performance of Kawaihae Harbor support the following conclusions:

- **Site Response.** From the damage we observed, we believe that the bedrock peak ground acceleration at Kawaihae Harbor was likely on the order of 0.3*g* to 0.6*g*. The attenuation relationship for this event is not well defined by traditional correlations. We believe that the dredge coral fills and natural coral sands will slightly amplify surface ground motions.
- **Liquefaction.** The observed liquefaction settlements, which ranged up to 7 inches, are reasonably consistent with predictions using simplified methods for the estimated PGA range. This finding suggests similar triggering thresholds. However, given the known significant differences between coralline soils in Hawaii and quartz soils, which were used to develop the simplified methods, it is likely that the majority of liquefaction occurred in uncemented dredged fills or only very loose pockets within the natural corals.
- **Structure Movements.** From the observed cracking at building foundations, riprap jetties, utility connections, and pavements, we estimate that Pier 1 moved offshore by approximately 1 to 5 inches, from the south to the north end, respectively, with offshore movement of up to 15 inches in adjacent riprap; Pier 2A moved onshore by about 1 inch; and Pier 2B appeared to have no appreciable lateral movement. Based on industry experience, most of the damage to wharves with a battered pile lateral resisting system

occurs at the battered pile and the connection to the deck. The amount of damage the battered piles will exhibit depends on the size and direction of the seismic event.

- **Effect of Age of Structures.** In general, the newer structures performed better than the older structures at Kawaihae Harbor, as determined by deformation and levels of damage. We believe that this result reflects the combined effect of more modern code provisions and less wear. For a discussion of historical and current building codes for wharves in Hawaii, see Appendix B.
- **Emergency Response Operations.** State of Hawaii personnel and coordinating federal officials responded rapidly to the Kiholo Bay earthquake. This rapid response greatly reduced the impacts to port operations after the event and serves as a model for other similar facilities. Also, the Harbor Master and the Coast Guard issued timely joint directions that resulted in efficient reorganization of operations to accommodate partial usage of Pier 1.
- **Context of Regional Seismicity.** The October 15, 2006, Kiholo Bay earthquake represents a seismic event that is within the range of events accommodated by current building codes, and with the coral soil conditions is perhaps typical of the level of seismic risk at many other similar ports in the U.S. territory Pacific islands, including other Hawaiian ports, American Samoa, the Marshall Islands, Guam, and the Northern Marianas Islands. We strongly encourage careful consideration of these recommendations for hazard mitigations and the preparation of emergency response operations by all affected ports. Performance observations are also useful for larger U.S. mainland ports having similar or greater seismic risk.

SECTION EIGHT SEISMIC MITIGATION GUIDELINES FOR PACIFIC ISLAND PORTS

8.1 OVERVIEW

The URS project team's observations of the seismic performance of the Kawaihae Harbor facilities provide a relevant point of reference for seismic hazard mitigation at other Pacific island ports. The lessons learned can serve as both a warning and an organizational planning tool. The purpose of providing seismic hazard mitigation guidelines with this case history is to combine a vivid local example with simplified applications to help educate local users from the primary body of literature used in the seismic mitigation industry. In this section, we draw heavily on three reference documents, which we strongly recommend for design and planning work:

- American Society of Civil Engineers Technical Council on Lifeline Earthquake Engineering, *Seismic Guidelines for Ports,* edited by S.D. Werner (Reston, Va.: ASCE Press, Monograph No. 12, 1998) (ASCE-TCLEE 1998)
- International Navigation Association, *Seismic Design Guidelines for Port Structures: Working Group No. 34 of the Maritime Navigation Commission* (Wiltshire, U.K., 2001) (INA 2001)
- American Society of Civil Engineers Structural Engineering Institute, *Seismic Rehabilitation of Existing Buildings* (Reston, Va.: ASCE Press, Standard 41-06, 2006) (ASCE-SEI 2006) or
- Federal Emergency Management Agency, *Prestandard and Commentary for the Seismic Rehabilitation of Buildings* (FEMA 356, November 2000) (FEMA 2000)

Application of these guidelines to Pacific island ports poses unique limitations for seismic mitigation owing to:

- Remote proximity
- Relatively small size
- Relatively small tax base
- Critical role as essential lifeline to island populations (little or no redundancy)

Hawaii serves as a model for other Pacific island ports because the statewide governance allows for the sharing of the resources of Honolulu with lesser populated outer islands, yet the outer island ports still operate under many of the same physical and operational conditions as other locations in the Pacific. It is hoped that the common heritage and association of these disparate geographic locales that are linked by the sea will aid in collaboration to achieve more common measures of seismic hazard mitigation and loss prevention, as intended by the codes.

Although the ever-present logistical challenges that remote locations face can cause often-complex code details to seem irrelevant, the intent of code provisions is to provide regularly updated and improved safety and reliability based on science and a growing body of experience. Even the smallest and most remote ports can adopt many of the seismic risk mitigation measures. Specifically, this report seeks to do the following:

- Advocate as the highest planning priority the adoption of more current standards or codes.
- Educate local port communities about Performance Based Design (PBD), which provides flexibility for local compliance with codes.
- Identify suitable risk-based practices to ensure that precious island resources are used wisely to achieve the best possible investment, reduce loss of life, and minimize economic disruptions. Prominent risk issues include liquefaction, structural floor systems, and lateral displacement performance.

The report concludes with key mitigation recommendations for Kawaihae Harbor and Pacific ports in general.

8.2 COMPARISON OF SEISMIC DESIGN STANDARDS FOR WHARVES

The basic intent of the seismic provisions in the Uniform Building Code (UBC) and the current International Building Code (IBC) is to protect life and safety and to prevent the collapse of buildings (IBC 2006). These same provisions have been used for waterfront structures as well. Generally, engineers have designed wharves using the requirements for Nonbuilding Structures, as described in the UBC. But these provisions do not address the specific issues that relate to waterfront structures. Waterfront structures do not have the same type of connections as buildings. Buildings are designed to have weak beams and strong columns, whereas waterfront structures are typically designed based as a weak column (pile) strong beam (deck) frame concept. The current building code (2006 IBC) and the associated standard (ASCE 07) have provided a section under Seismic Design Requirements for Nonbuilding Structures Similar to Buildings, which includes a section on piers and wharves. But these provisions still do not adequately address the specific requirements related to waterfront structures. Therefore, many port authorities, such as the Port of Los Angeles and the Port of Long Beach, have started to develop their own design standards for use with the UBC. The USACE and the Department of Defense have also developed a design standard for piers and wharves within the United Facilities Criteria (UFC) (DOD 2005).

These new design standards are based on Displacement-Based Design and Limit-States Design and on the concept that damage to a structure is strain-related and the displacement can be directly related to the strain. These standards set limits on the drift, strain, and component capacities. The basic design criterion of these standards is that the displacement capacity must be greater than the displacement demand.

These standards recommend that wharves be laterally supported only by vertical piles; battered piles are not recommended for lateral support because of their poor performance in past seismic events. Their poor performance is mainly due to poor detailing of battered pile connections. The connections were not designed or detailed to have adequate ductility or strength when subjected to earthquake loads and deformation demands, resulting in brittle failure and damage. Battered piles are typically designed by assuming relatively little moment fixity at their connections to the other elements of the structure. As a result of this design approach, these connections have only limited ductility, which limits the piles' energy-absorbing capabilities and ability to resist seismic excitations without damage, particularly where there is soil movement.

8.3 INTRODUCTION TO PERFORMANCE-BASED DESIGN

Current practice for the design of wharf structures is to use the response spectrum method of dynamic analysis, as presented in the UBC and the IBC. Although not widely used at this time, Performance-Based Design (PBD) is gaining popularity as an alternative to the building code method of design. Currently, both the Port of Los Angeles and the Port of Long Beach have developed seismic codes and design standards that require the use of PBD, in conjunction with the building codes, to design of any new container wharf. Therefore, this report discusses PBD as an alternate or supplementary method to the requirements of the Building Code for analyzing new and existing waterfront structures.

The Applied Technology Council, under the sponsorship of FEMA, is currently engaged in a project (FEMA 445, Next Generation Performance-Based Seismic Design Guidelines) to advance the state of practice for Performance Based Design. FEMA 445 provides performance-based seismic design procedures and guidelines for structural and non-structural components in new and existing buildings as well as new and existing waterfront structures, such as piers and wharves.

In FEMA 445, performance-based design is described as follows:

> Performance-based seismic design evaluates how a building is likely to perform in an earthquake, given the potential hazard it is likely to experience, considering uncertainties inherent in quantifying potential hazards and uncertainties in assessing actual building response. It permits design of new buildings or upgrade of existing buildings with a realistic understanding of the risk of casualties, occupancy interruption, and economic loss that may occur as a result of future earthquakes. It also establishes a common vocabulary between stakeholders and design professionals on development and selection of design options. It provides a framework for deciding the level of safety and level of property protection, at what cost, are acceptable to building owners, tenants, lenders, insurers, regulators and other decision makers for specific project needs.
>
> In contrast to prescriptive design approaches, performance-based design is a systematic methodology for assessing performance capability of a building, system, or component. It can be used to verify the equivalent performance of alternatives, deliver standard performance at a reduced cost, or confirm higher performance needed for critical facilities. (FEMA 2006)

In the 1990s, the first generation PBD procedures introduced the concept of performance in terms of five defined performance levels named to connote the expected level of damage for evaluating existing structures: Collapse, Collapse Prevention, Life Safety, Immediate Occupancy, and Operational Performance. These procedures also introduced the concept of performance related to damage of both structural and nonstructural components. Performance objectives were developed by linking one of these performance levels to a specific level of earthquake hazard. Although intended for existing buildings, these procedures are being extrapolated for new structures.

To illustrate application of PBD to Pacific island ports, the anticipated event (basic considerations of seismic risk exposure) and the anticipated response (port facility structures or components and performance) are discussed in Section 8.4. Recommendations are discussed in Section 8.5.

8.4 RISK MITIGATION PRACTICES

To apply PBD for seismic hazard mitigation to an entire port operation requires understanding event risk (site seismicity), the anticipated performance (both components and system), and the consequences of failure for each component of the port system (INA 2001; GAO 2007). A risk profile can then be developed to compare vulnerabilities and prioritize repairs. Each of these risk mitigation steps is briefly described below as it relates to the generalized conditions of Pacific ports.

8.4.1 Regional Seismic Hazard

The **M** 6.7 Kiholo Bay earthquake and the resulting ground motion experienced at Kawaihae Harbor are likely typical of the potentially severe seismic events that could occur at other U.S. territory Pacific island ports, such as other Hawaiian ports, American Samoa, Marshall Islands, Guam, the Northern Mariana Islands, Micronesia. All of these locations within Oceania have histories of volcanism (albeit older than Hawaii) and in many cases have nearby faulting or subduction zones. This seismicity is loosely associated with the greater Pacific ring of fire, which has historically caused more damage in Japan and the west coasts of North and South America. Although Oceania subduction zones, such as the Marianas Trench, pose the potential for significantly high earthquake magnitudes (such as the August 8, 1993, **M** 8.3 Guam earthquake), the resulting attenuation and ground motions are less well defined and differ considerably from their continental counterparts. For example, all known recorded ground motions at ports within Pacific island subduction zones thus far are within the ranges observed in the Kiholo Bay event. In general, all of the identified Pacific ports should be preparing on a basic level to deal with damaging design-event PGA values of at least 0.3*g*, and in some locations PGA values in excess of 1.0*g* and related potential near-source and far-source tsunami risk.

8.4.2 Anticipated Performance

This section discusses typical port infrastructure features and their anticipated performance for selected U.S. territory Pacific island ports, with an emphasis on ports in Hawaii and Guam, which have the majority of the available information. Data were obtained from the team's project files or other available sources. Typical features include wharf structures (piers, bulkheads, and dolphins), waterfront buildings, fuel tanks, utilities, and site development facilities (reclaimed land, breakwaters, and jetties). The discussion covers typical construction methods, typical standards, general geologic and geotechnical conditions, known seismic remediations (if any), and a generalized judgment of anticipated performance.

Wharf Structures (Piers, Bulkheads, and Dolphins)

The most critical structures for port operations are the wharf facilities used for mooring and the daily offloading of goods. Where smaller ports have only one or two proper wharf structures, dolphins are commonly used for both mooring and staging offloading via ramps to a nearby pier or shore. Construction practices predominantly use reinforced-concrete pile foundations, frames, and decks; locally available aggregate is used where possible. Historically, local coral aggregates have been used, though more recent facilities generally require higher-quality aggregates with less variable sulfate and chloride behavior, such as local basalt (in the case of Hawaii) or other

higher quality imported aggregates from America or granites from Asia. Construction equipment of sufficient size and variety is usually available, though it tends to be old, with added improvisations from scarcity of repair parts. Piles are predominantly driven with open-ended diesel hammers and staging with single-boom crawler cranes.

Ground improvements for seismic remediation of wharf structures are known at this time to have been applied only recently (within the past decade) in the Pacific islands (at the Port of Guam, in a very limited portion of Pearl Harbor, and at Kings Wharf, Fiji [an Asian Development Bank project]). Current wharf structure designs for the large ports in Hawaii (Honolulu and Pearl Harbor) and Guam (Port of Guam and Apra Harbor) are starting to include heavier seismic-resistant components, but this practice has not yet been extended to smaller or remote facilities. Design standards for wharf structures vary significantly by locale; mostly, these standards are some derivative of allowable stress design.

Most of the structures in service are remnants or expansions of World War II–era facilities, though many are older, such as Pier 1 at Hilo Harbor, which dates to the 1920s, when the use of reinforced concrete was in its infancy. The warm humid environment and high saline environment results in steel corrosion problems, which cause cracking and spalling that require localized repairs. These repairs are the predominant maintenance and earthquake damage expenses. It can be difficult to distinguish the pre-existing hidden corrosion damage from damage caused by an earthquake. Sheet piles include both concrete and driven steel, with newer steel sheets usually having cathodic protection and older steel sheets and their tieback in advanced stages of corrosion, disrepair, or failure. A common mode of failure is advance corrosion of tieback rods. Where instability prevents mooring, local drop anchors are used and offloading operations keep to the rear of wharfs as much as possible.

In summary, because of the age of wharf structures and corrosion effects, seismic performance of all but the best-maintained or newer piers under a severe design event is anticipated to be poor. From this standpoint, performance at Kawaihae Harbor is believed to be typical, with well-maintained (Pier 2A) or newer (Pier 2B) wharves performing well and older wharves (Pier 1) performing poorly. If a port has predominantly older structures, then the port should be considered vulnerable to poor performance.

Waterfront Buildings

Consisting of mostly transit sheds and operations buildings, waterfront structures at Pacific island ports are a mixture of reinforced-concrete structures with masonry infill walls and tin roofs or lightweight steel buildings of an age that is similar to that of their associated wharves. The corrosion at waterfront structures tends to be less severe than that of the wharves, which are in contact with tidal cycles, though the eaves of waterfront structures can attract corrosion from ocean mist. Waterfront structures are often placed so that they bear directly on the adjacent wharf. The result is potentially differential movements, such as those that occurred at Kawaihae Harbor. This tendency is the case regardless of whether the remainder of the building is supported on shallow footings or a pile system, as the pile designs for buildings are usually more flexible than the pile designs for wharf structures.

Waterfront structures at Pacific island ports are generally designed using some version of the Uniform Building Code, and thus are anticipated to perform reasonably well (i.e., few collapses), though not without extensive damage such as what was observed at Kawaihae Harbor. We are

aware of few seismic retrofits or ground improvements at waterfront structures, so would expect the foundation movements of the buildings to be significant and the primary cause of damage, such as what occurred at Apra Harbor in Guam in 1993, where some buildings moved significantly (i.e., up to 20 feet) but remained intact.

Fuel Tanks

Perhaps second in importance only to wharf structures, fuel tanks pose a critical lifeline for energy supply to communities supported by Pacific island ports. Tanks are generally constructed of steel and are designed according to American Petroleum Institute provisions, but they are also generally old, often with kinks or irregular structural deformations from aging or use. Where regular maintenance and painting is provided, tanks older than 50 years often perform well with regard to leakage. However, condensation in the tanks tends to cause internal corrosion and pinholes throughout the roofs, so it is not uncommon to find rainwater on the surface of fuel oils stored in the tanks.

We are not aware of the catastrophic failure of a large fuel tank in recent history at a Pacific island port, though spill prevention measures generally have room for improvement. Given the ductility of tank structures, we would expect fuel tanks to perform well enough to avoid collapse, though foundation deformations could be significant, as with buildings, since we are not aware of ground improvements at fuel tank facilities at Pacific island ports, except for three tanks within the Sand Island fuel tank farm in Honolulu. Tanks with pile foundations may demonstrate added seismic resistance, though many piles failed from liquefaction and lateral spreading during the 1994 Northridge and 1995 Kobe earthquakes, and we are not aware of pile designs incorporating more current liquefaction-resistant design procedures.

Utilities

Electrical, water, sewer, and fuel lines at Pacific island ports tend to be varied in age and construction quality. Trench backfill is generally granular material; whether this material is well compacted or not, it is subject to seismic movements of the surrounding materials, which tend to be loose dredged fills. Concrete jacketed conduits are uncommon. Utility design generally reflects local practice, with little attention to structural or geotechnical issues. Repairs and expansions are often performed by maintenance personnel with only limited design work undertaken either before or after construction. Thus, the general performance of utilities at Pacific island ports during earthquakes is anticipated to be poor, with severed connections at the wharf structures and off-site hookups. The absence of utility damage at Kawaihae Harbor as a result of the Kiholo Bay earthquake is quite remarkable.

Site Development Facilities (Reclaimed Land, Breakwaters, and Jetties)

Reclaimed land, breakwaters, and jetties at Pacific island ports tend to be constructed of locally available materials, with the possible exception of riprap armor stone, which tends to be imported, except in the state of Hawaii, where local basalt sources are available. Dredging practices tend to be variable, with an apparent scarcity of dredging equipment in the region. Dredged fills are usually hydraulically placed and uncompacted, except in the upper 5 feet in association with pavement construction. Design and construction practices are largely local, as

no common standard exists for port site development, excepting jetties and breakwaters, which often use military construction standards and design guidance from USACE.

We are unaware of ground improvements at Pacific island ports other than the few cases of wharf and fuel tank improvements cited earlier. The predominant subsurface materials at Pacific island ports are natural coral formations, often to depths of over 100 feet, with widely varying degrees of compressibility, strength, and cementation. Thus, port performance is anticipated to be generally poor, with pervasive dynamic settlements, foundation instability, liquefaction, and lateral spreading, even where site works benefited from good design and construction practices.

8.4.3 Consequences of Failure

The consequences of the seismic failure of a port component or system will vary largely depending on the logistical environment and the availability of local resources for emergency response. This availability decreases with distance from Oahu and Guam. Pacific island ports generally function as the sole lifeline to their supporting communities, which can include smaller outer islands hundreds of miles away. Travel and shipping logistics to remote locations in the Pacific is generally very time consuming. In many cases of disaster relief, supplies are still air-dropped due to the large distances between relatively small populations. Typical lead times for shipping supplies would be several weeks, though special deliveries can be accommodated in several days if a private barge is secured. The airfreight delivery of materials is usually not feasible for general construction, excepting specific machined or fabricated, relatively small components. Therefore, we judge the consequences of failure to be generally high for Pacific island ports. Kawaihae Harbor was unique, even among Hawaiian outer islands, for its ability to coordinate with Hilo Harbor and re-route supply lines.

8.4.4 Risk Profile

By summarizing exposure, anticipated system performance, and the consequences of failure, a risk profile can provide a useful overview for prioritizing seismic hazard mitigations. Risk profiles can be expanded to include multiple hazards and detailed operational, economic, and environmental impacts (GAO 2007). Appendix F provides an example of a conceptual risk profile for selected U.S. ports. The profile includes multiple hazards (GAO 2007). Table 8-1 shows the conceptual risk profile that we have prepared for Pacific island ports. It is largely based on judgments from our experience and is therefore not definitive. The purpose of this analysis is to aid individual port owners in developing more detailed risk profiles for their own specific assets and to provide a frame of reference for regional seismic risk exposure.

8.5 RECOMMENDED SEISMIC MITIGATIONS FOR PACIFIC ISLAND PORTS

Recommendations for seismic mitigations of Pacific island ports can be developed from a basic understanding of current design standards, principles of performance-based design, and seismic risk profiles. The recommendations discussed include specific actions for Kawaihae Harbor based on the performance evaluation and the specific performance goals that DOT Harbors and the current scope of the PA program provided for Kawaihae Harbor. These recommendations also serve as an example for other ports. The recommendations encompass wharf structures,

buildings, geotechnical foundation improvements, site and utility improvements, and monitoring provisions. Additional seismic mitigation methods are listed in Appendix F.

Table 8-1. Conceptual Risk Profile for Selected U.S. Pacific Island Ports

Location[4]	Seismic Risk[1]	Anticipated Performance[2]					Consequence of Seismic Failure[3]
		Piers	Buildings	Fuel Tanks	Utilities	Site/ Jetties	
Kawaihae, Hawaii	3	1	3	2	2	2	1 – support from Hilo
Hilo, Hawaii	3	1	2	pending	2	1	1- support from Kawaihae
Kahului, Hawaii	3	2	pending	3	pending	2	2 – minor support from Lahaina
Lanai, Hawaii	3	2	pending	pending	pending	3	2 – support from Maui
Molokai, Hawaii	2	2	pending	pending	pending	2	2 – support from Oahu
Lihue, Hawaii	1	1	2	pending	2	1	1 –support from Pt Allen & Oahu
Pago Pago, Samoa	3	3	pending	pending	pending	pending	3 – lifeline to remote islands
Majuro, Marshall Islands	2	3	pending	pending	pending	pending	3 – lifeline to remote islands
Port of Guam	3	1	2	2	2	3	3 – critical strategic hub
Port of Saipan	3	2	pending	pending	pending	pending	3 – lifeline to remote islands
Koror, Palau	3	3	pending	pending	pending		3 – lifeline to remote islands

Notes:
1. Seismic risk values are estimated as 1 (low) to 3 (severe).
2. Estimated performance levels range from 1 (best) to 3 (worst).
3. Estimated consequences range from 1 (least impact) to 3 (most impact).
4. Honolulu Harbor, Pearl Harbor, and Apra Harbor were excluded, because they are either large or Department of Defense ports.

8.5.1 Public Assistance Program Repairs

The Federal Emergency Management Agency created the PA Program to provide assistance to states, local governments, and certain private non-profit organizations. Through this program, the federal government provides economic support for the repair, replacement, or restoration of facilities damaged during federally declared disasters (less applicable insurance claim payments).

After the Kiholo Bay earthquake, the DOT filed for funding assistance for many of the damaged facilities at Kawaihae Harbor. The eligible geotechnical repair, replacement, or restoration measures are as follows:

FEMA Project Worksheet No.	Description
70	Temporary mooring installation at Pier 1 to facilitate the unloading of cement and other construction materials. Installation of two deadman systems on concrete piles.
76	Repair of a 6-inch-diameter ductile iron fire water line at two locations in the container yard.
87	Pavement repair throughout the harbor, including 700 linear feet of cracking. Pavement repairs for heaving and settlement, including 780 square feet at Pier 1, 8,750 square feet at Pier 2A, and 6,900 square feet at Pier 2B.
279	Repair/replacement of subfloor/floor of Pier 2A Overseas Terminal Shed (12,474 square feet). Repair of small area of pavement outside the shed near the sliding door rail on the north side. Repair of 60-square-foot area of pavement adjacent to the southeast corner of the office area beneath the shed overhang. Additional funds for a geotechnical investigation to determine the extent of damage to the subfloor/floor of the shed and office area.
281	Funding to conduct a safety, structural, and liquefaction assessment of the harbor.
334	Repair of 2-inch water line near shed at Pier 1.
339	Repair of revetment east of Pier 1, including retrieval, placement, and grouting of 23 cubic yards of displaced rocks. Repairs to revetment east of Pier 2A, including cleaning and grouting of 9 cubic feet of cracks.
641	Modification of pier deadman system/column foundation for Pier 1 – Shed 1 with replacement option if required. Minor pavement crack repair.
663	Sheet pile repair, as noted in the underwater dive report. Infill of separation gap between the concrete bulkhead and pavement near the southern end of Pier 1 (adjacent to the drainage canal) using sand-cement grout.

DOT submitted other project worksheets in reference to Kawaihae Harbor; however, FEMA either did not approve the appropriation of funds for the project worksheets or the repairs were structural in nature and are not documented here.

8.5.2 Recommended Wharf and Pier Mitigations

Older wharf and pier structures have typically been constructed using non-prestressed vertical and battered piles. These piles usually do not have the ductility reinforcing and detailing normally required to resist seismic loads.

At least four options are available to improve the lateral resisting system of existing wharf structures. The method used will depend on the construction of the wharf, the amount of anticipated force that needs to be resisted, the geotechnical conditions, and a detailed analysis of the wharf and lateral resisting force system. The actual improvement could be one of the following options or a combination of these options.

- **Option 1:** Install a new independent lateral resisting system. This option would require the addition of new steel or prestressed concrete vertical piles capable of resisting the entire seismic load. The existing vertical piles would be used to support the vertical loads only, and the new piles would be designed to resist the entire lateral load. This option requires tying the new and old systems together and is typically installed under the existing deck. Additional pile caps or beams may be required to develop the forces into the deck. If the existing structure has battered piles, the battered piles would be removed by cutting off the pile between the bottom of the deck or pile cap and the mud line.
- **Option 2:** Add additional vertical piles. This option would require the addition of new steel or prestressed concrete vertical piles capable of resisting a portion of the seismic load. The existing vertical piles would be used to support both the vertical loads and a portion of the seismic loads. The loads would be distributed based on the stiffness of the new and existing piles and the capacity of the connection to the deck. This option requires tying the new and old systems together and is typically installed under the existing deck. Additional pile caps or beams may be required to develop the forces into the deck. If the existing structure has battered piles, the battered piles would be removed by cutting off the pile between the bottom of the deck or pile cap and the mud line. The difference between this option and option 1 is that in option 1 the entire lateral load is resisted by new vertical piles, whereas in option 2 the lateral load is resisted by both the new piles and the vertical existing piles.
- **Option 3:** Increase the vertical and lateral capacity of existing vertical piles by encapsulating the pile in a fiber-reinforced polymer (FRP) jacket. This jacket would be placed in such a manner as to provide a space between the pile and the jacket for the addition of high-strength grout and reinforcing. This grout and reinforcing would increase the vertical and lateral capacity of the pile. To increase the moment capacity at the joint, reinforcing dowels would be embedded into the existing deck and lapped with the new reinforcing steel around the pile. The bottom of the jack would need to extend a distance on the order of five to ten pile diameters from the pile cap or 5 feet below the mud line, whichever is greater.
- **Option 4:** Add tie rods and anchor blocks. This option would require the installation of a new continuous anchor block behind the wall and tie rods. The tie rods would be located perpendicular to the length of the wharf and at opposing 45 degree angles (one in each direction). The combination of these tie rods would resist both transverse and longitudinal seismic loads. The location of the anchor block is critical and should be placed as far back as practical to be outside the failure plane of the soil and to avoid possible failure of the anchor block. This option would require significant structural and geotechnical analysis to be sure that the anchor blocks would provide adequate lateral support for the wharf.

8.5.3 Recommended Harbor Building Mitigations

Most Pacific islands have limited harbor resources; therefore, it is often critical that harbors remain functioning after a natural disaster to be available for both normal service and potential emergency response. The seismic mitigation of harbor buildings should follow a two-tiered

approach based on the use of the buildings. An evaluation should be made to determine which buildings are essential to the continued operation of a given harbor and which are non-essential.

In addition to the evolution of building codes, the evaluation of seismic risk in the Hawaiian Islands has gone through substantial change in recent years. Before 1990, construction in the County of Hawaii (i.e., on the island of Hawaii) was based on Zone 3 requirements. However, more recent codes have recognized that the risk is greater, and the code requirements have subsequently been upgraded to Zone 4. Many older buildings may not meet the current code requirements for seismic resistance.

Harbor buildings that are deemed essential should undergo a thorough evaluation of existing conditions to determine their potential seismic design and/or construction deficiencies. The reference publication *Seismic Rehabilitation of Existing Buildings* (ASCE-SEI 2006) discusses the latest generation of performance-based seismic rehabilitation methodology and is a valuable tool to use in determining potential seismic and construction deficiencies and improving building performance in future earthquakes.

From our visual observation of the damage sustained at Kawaihae Harbor, we recommend that the following general mitigation measures be considered:

- Although the harbor buildings performed well during the Kiholo Bay earthquake, areas of structural weakness may exist in their systems that were not revealed by this particular seismic event. The existing construction of these buildings should be evaluated to determine the demand/capacity ratios under current seismic code requirements. This information can be used to determine which elements of the structures require upgrade.
- A cost-effective mitigation program could include an update of a building's lateral bracing systems. Both the strength and the stiffness of an existing building can be enhanced by replacing the existing tie rod bracing with larger members.
- Some damage appeared to occur to the concrete piers at the attachment of some of the column bases. These attachments and bases should be repaired utilizing epoxy injection.
- Harbor buildings that remain standing but have limited access due to liquefaction or racking will be of little use in post-earthquake operations.
- New buildings in areas susceptible to liquefaction should be designed with structural floor systems rather then slab-on-grade, which can settle and limit access. Entry areas to a facility should be designed with an entrance slab system that can bridge and rotate between areas that have settled due to liquefaction and the building.
- The industry standards for lateral deflection criteria for metal building warehouses result in structures that are inherently flexible. These buildings should either be designed to a more stringent requirement or the door systems should be detailed to accommodate the maximum anticipated building movements.

8.5.4 Recommended Geotechnical and Foundation Mitigations

The recommended geotechnical measures are closely related to structural mitigation concepts and must be considered in light of the desired performance-based design of nearby structures. Neither UFC-152-02, MOTEMS, nor IBC provide specific design recommendations for the

remediation of port structures against seismic or liquefaction hazards, but all require that remediation be provided. Both *Seismic Design Guidelines for Ports* (ASCE-TCLEE 1998) and *Seismic Design Guidelines for Port Structures* (INA 2001) include recommendations for damage criteria, methods of analysis, and methods of remediation, and these recommendations often include detailed descriptions.

Seismic hazards in soils, usually liquefaction or related seismic deformations, can be mitigated using one or a combination of the following soil treatment or ground improvement methods: densification, improvement of drainage characteristics, cementing of the soils, or use of structural elements to resist seismic loads. The most common methods to mitigate liquefaction hazard near or beneath foundations include the installation of vibro-stone columns, gravel drains, compaction grouting, and piles or micropiles. Other methods of seismic foundation mitigations are tabulated in Appendix F.

Vibro-stone columns use a combination of densification and drainage improvements. They are installed using a large vibratory compaction probe, which discharges sand or gravel into an annular column usually 2 to 3 feet in diameter (see Figure F-4 in Appendix F).

Gravel drains increase the drainage characteristics of the soil and are implemented by augering a hole to the desired depth and size and filling the hole with gravel (see Figure F-4 in Appendix F).

Compaction grouting increases the density of the surrounding soil by injecting thick mortar-type grout into the subsurface, which displaces soil and creates a strong grout column.

Structural solutions to mitigate liquefaction can also be used with or in lieu of ground improvements. Piles and micropiles have proven effective against liquefaction hazards by transferring the building loads into non-liquefiable soils, so long as the large horizontal forces can be adequately carried by the piles. Piles and micropiles are not effective when they do not extend sufficiently into non-liquefiable soils or where lateral spread is possible and may cause excessively large bending moments in the piles.

Other recommended geotechnical mitigations include the following:

- The stability of the slope can be improved using methods of ground improvement (see ASCE-TCLEE 1998, Chapters 4 and 6).
- The liquefaction hazard for all waterfront structures and buildings can be reduced using methods of ground improvement. Piles or micropiles may also be used where lateral spreading is not possible; lateral spreading near pile foundations may cause severe displacement failures and bending moments near the interface of liquefiable and non-liquefiable soil layers (see Figure F-6 in Appendix F) (INA 2001, Chapter 2; ASCE-TCLEE 1998, Chapter 8).
- Thickened-edge slabs can provide improved performance by resisting the differential settlements of the soil beneath the slab.

8.5.5 Site and Utility Restorations

Liquefaction hazard can be mitigated using methods of ground improvement to reduce the probability of liquefaction. Methods of ground improvement to mitigate liquefaction hazard for larger regions of reclaimed port sites, jetties, or near utilities include the installation of vibro-replacement stone columns, gravel drains, deep dynamic compaction (DC), deep soil mixing,

excavation removal and replacement with compacted soils, and compaction grouting. Other methods of ground improvement are tabulated in Appendix F.

Stone columns, gravel drains, and compaction grouting are described in the previous section. DC, deep soil mixing, and removal and replacement are best used on large site areas before construction. DC is densification of soil by dropping a weight of 10 to 30 tons from a height of 80 to 130 feet (25 to 40 meters) above the ground. Due to the vibrations generated, DC is not recommended near existing structures or utilities. Soil mixing improves the soil by injecting and mixing cement using mechanical means (paddles) or hydraulic means (jet nozzles).

Site restoration recommendations include the following:

- Remediation near critical utilities and lifelines should be a priority to ensure the landside operation of the port and its equipment after a seismic event. Lifelines include power, water, wastewater, communications, natural gas, and liquid fuel. Guidelines for utility and lifeline improvements can be found in ASCE-TCLEE 1998, Chapter 8.
- Pavements should be reconstructed or milled and overlaid to restore grades from liquefaction subsidence. Ground improvement may need to be performed in areas of heavy loads or more severe liquefaction potential (see ASCE-TCLEE 1998, Chapter 8.4).

8.5.6 Monitoring Recommendations

The monitoring of port facilities can be performed to aid maintenance assessments. Often, monitoring can identify early warning signs of vulnerabilities to a seismic event. Monitoring recommendations for seismic mitigation include the following:

- Wharves should be monitored regularly for potential tidal wash within cofferdams, bulkheads, and riprap to avoid erosion that creates unstable conditions that are more susceptible to failure during an earthquake.
- Bulkheads should be monitored regularly, both the top and the toe of walls and slopes, to provide early warning of potential instability in areas of ongoing movements.
- Topographic surveys of port facilities should be kept current. A topographic monitoring program of selected points at key structures can be performed annually for a minimal cost.
- Remote data such as aerial photography, satellite, and lidar imagery can be used to monitor significant ground displacements and general site conditions.
- Geophysical methods can be used to monitor structure movements; methods include three-dimensional laser scanning, ground-based lidar, and side-scan sonar.
- Instrumentation can be installed within critical structures or at critical site locations to monitor movements. Common examples of instrumentation are inclinometers, tilt meters, vibration monitors, and, more recently, low-cost alternative displacement measurement techniques, such as those using fiber optic cables.

8.6 CONCLUSIONS

After reviewing the characteristics of Pacific island ports, applicable codes, applicable standards, seismic mitigation guidelines, and risk profiles, we offer prioritized recommendations for Kawaihae Harbor. These recommendations serve as a specific example within these general guidelines for other Pacific island ports. The URS project team strongly encourages careful consideration of these recommendations for hazard mitigations and the preparation of emergency response operations for affected ports. Our recommendations are as follows:

- As the highest planning priority, adopt more current standards or codes.
- Educate local port communities about Performance Based Design, which may in the future provide additional flexibility for local compliance with codes.
- Identify suitable risk-based practices to ensure that precious island resources are used wisely to achieve the best possible investment, reduce loss of life, and minimize economic disruptions. For each port, which often acts as a sole lifeline to its supporting communities, we recommend the following:
  - Determine site seismicity and surface ground motions for each critical structure.
  - Develop a seismic risk profile for the port infrastructure system inventory.
  - Allocate hazard mitigation program funding in a way that complies with the eligibility requirements for FEMA assistance.
- Perform the following mitigations to critical structures using current methods:
  - Use high-strength piling and anchorage retrofits.
  - Apply ground improvements.
  - Monitor performance of the structures.
- Update the seismic response provisions in the general port emergency operations plan, including:
  - Use the lessons learned from the Kawaihae Harbor response while according with current Department of Homeland Security port provisions.
  - Spell out details of on-the-ground emergency command control between the harbor master and the Coast Guard or Department of Defense commander responsible for the harbor waters, assigning a single harbor representative with decision-making authority.
  - Maintain mooring capacity reserves by keeping mooring dolphins without moored loads and flexible piping to restore damaged fuel tanks.

SECTION NINE LIMITATIONS

The data, interpretations, and assessments contained in this report are based on the project concepts discussed herein, the available information about the project, the available subsurface exploration and geologic data, and our professional judgment and experience.

This report presents our opinion of the structural and subsurface conditions at Kawaihae Harbor and the properties of the materials at the time of the earthquake. To arrive at our opinion, it was necessary to interpolate or extrapolate from limited data (e.g., between soil borings), to estimate the conditions. Although the properties of the materials encountered in the field are expected to be within the ranges discussed, the actual distribution of materials encountered will likely vary from those discussed in this report. Should conditions be identified that differ from those reported herein, we should be contacted immediately so that we can review and modify our recommendations accordingly.

We have prepared this report for the use of FEMA, the State of Hawaii Department of Transportation Harbors Division, and other Pacific ports that are or may become eligible for FEMA assistance. The report is intended for educational and planning purposes in general accordance with the scope of services outlined in the URS contract agreement with FEMA. Detailed design of port structures or seismic retrofits should be provided by a qualified engineer in accordance with current local codes and practices.

The conclusions and recommendations of this report were developed with the standard of care commonly used as state of the practice in the profession. No other warranty, expressed or implied, is made for the professional advice included in this report. This report may not contain sufficient information for the purposes of other parties or for uses other than those described herein. URS does not assume responsibility for the accuracy of the project information provided by others.

SECTION TEN REFERENCES

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10.2 DRAWINGS

Drawing No.	Project Number	Location	Description
1	HC 916A	Pier 1	Dredging and other improvements, Kawaihae Harbor, Hawaii (includes boring logs and Pier 1 construction drawings) (March 1954).
2			Dredging and other improvements, Kawaihae Harbor, Hawaii (July 1955).
3	HC 1031 C	Pier 2A	Overseas Terminal at Kawaihae Harbor (January 1958).
4	HC 1031 D	Pier 2A	Metal transit shed building on pier (April 1959).
5	HC 1331R	Pier 1	New Freight Shed (South Metal Building). Kawaihae Harbor, South Bldg., Hawaii (January 1968).
6	HC 1155	Pier 1	Dredging at Kawaihae Barge Landing (July 1960).
7	HC 1226	Pier 1 and revetment	Repair and improvements of riprap wall and drainage ditch (December 1964, including soundings in front of revetment and Pier 1 and borings).
8	HC 5108	Pier 1 (North and South Metal Buildings)	Repainting steel members at freight sheds, Kawaihae Harbor, Hawaii (June 1977). (Plans show sections and floor plans for each building.)
9	HC 5141	Pier 1	Bulkhead repair at barge pier, Kawaihae Harbor, Hawaii (March 1980). (Repair at north end of pier.)
10	HC 5218	Pier 2A/2B	Drawings for landside improvements at Kawaihae Harbor, Hawaii (1990). (Includes drawings for heavy load bridge over drainage channel.)
11	HC 5219	Pier 2B	Drawings for dredging and overseas pier extension at Kawaihae Harbor by Nishimura, Katayama, Oki & Santo, Inc., Okahara & Associates, Inc. (May 1990).
12	HC 5248	Pier 2A	Maintenance dredge at Overseas Terminal (February 1992; soundings from February 1987 survey).
13			U.S. Army Corps of Engineers, harbor soundings (March 1994).
14	HC 5313	Behind Pier 2A	Pave additional barge terminal area, Kawaihae Harbor, Hawaii (October 1999).
15	HC 5335	Drainage ditch	Repair GRP drainage ditch (March 2001).

Drawing No.	Project Number	Location	Description
16	HC 5218	Pier 2A/2B	Pile driving logs; landside improvements at Kawaihae Harbor (June 1993). (8½- x 11-inch drawings)
17	HC 5219	Pier 2B	Ernest K. Hirata & Associates, Inc., Test Pile Driving Report, Extended Overseas Pier, Kawaihae Harbor Improvements. Summary of Test Pile Logs, Pile Driving Logs (May 1991). (8½- x 11-inch drawings)
18	HC 5219	Pier 2B	Dredging and Overseas Pier Extension at Kawaihae Harbor, Hawaii, Pile Driving Logs, Volumes 1, 2, and 3 (1991) (8½- x 11-inch drawings)
19	HC 5219	Pier 2B	Nishimura, Katayama, Oki & Santo, Inc., and Okahara & Associates, Inc., Structural Calculations for Dredging and Overseas Pier Extension at Kawaihae Harbor (May 1990). (8½- x 11-inch drawings)
20	HC 404	Hilo Harbor	Pier 2, Kuhio Bay, Hilo, Hawaii (September 1921).
21	HC 452	Hilo Harbor	Reconstruction of Pier 1, Kuhio Bay, Hilo, Hawaii (December 1923).
22	HC 486	Hilo Harbor	Pier 3, Kuhio Bay, Hilo, Hawaii (March 1926).

Figures

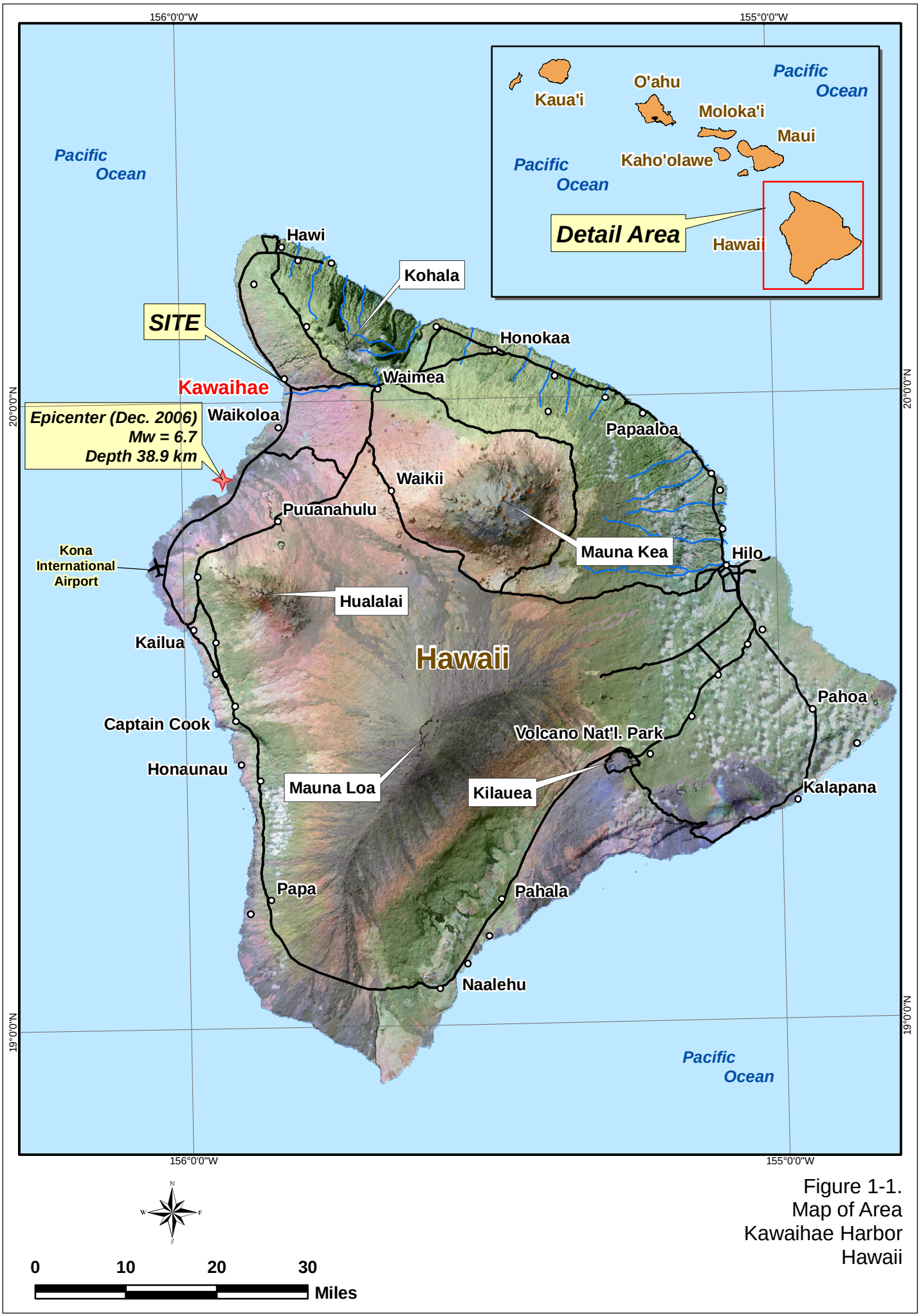


Figure 1-1.
Map of Area
Kawaihae Harbor
Hawaii

- Boring Location (1954)
- Boring Location (1964)
- Boring Location (1988)
- Boring Location (1989)
- Boring Location (1990)
- Boring Location (1999)

0 150 300 450 Feet

Figure 1-2.
Kawaihae Harbor Terminal Plan
Kawaihae Harbor
Hawaii

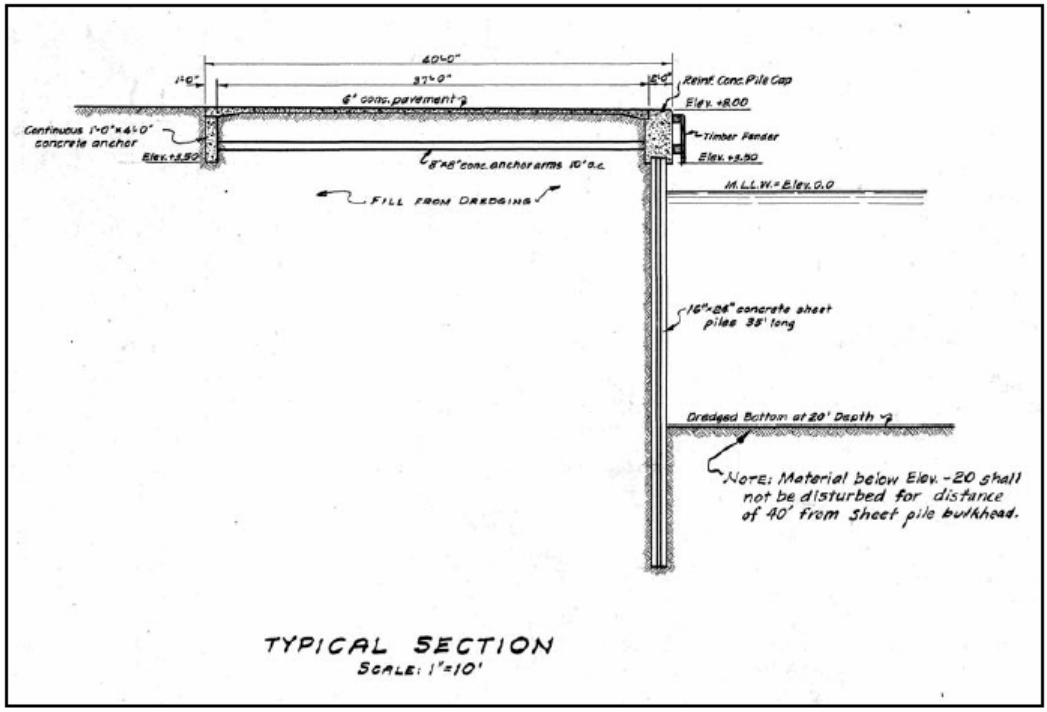


Figure 2-1 Pier 1: Typical Section

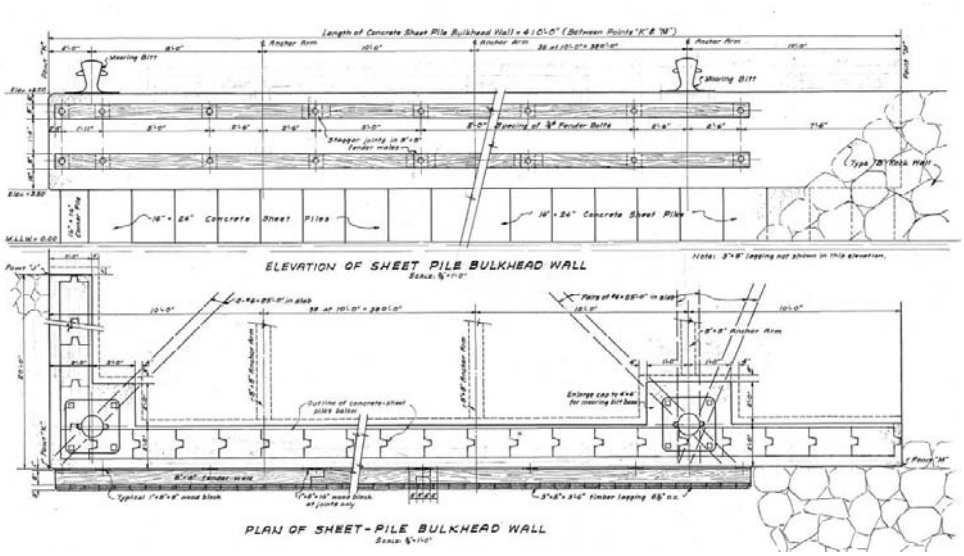


Figure 2-2 Pier 1: Typical Pile Bulkhead Wall

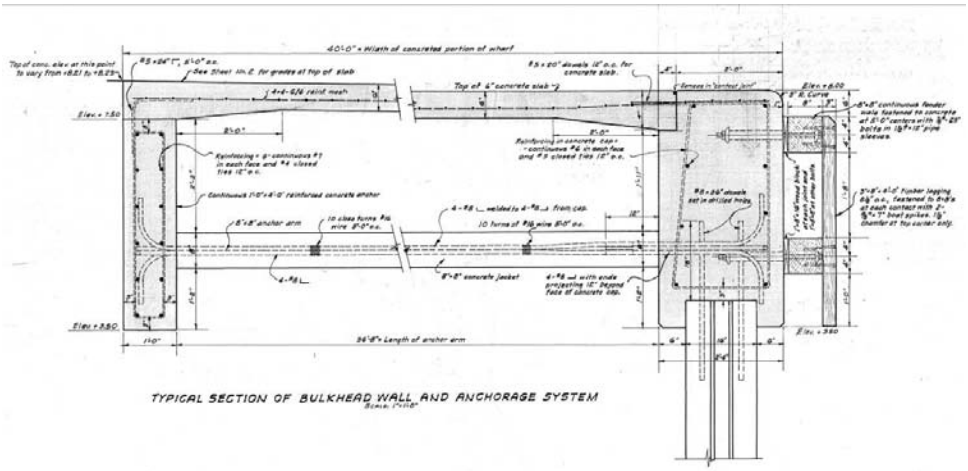


Figure 2-3 Pier 1: Typical Section of Bulkhead Wall and Anchor System

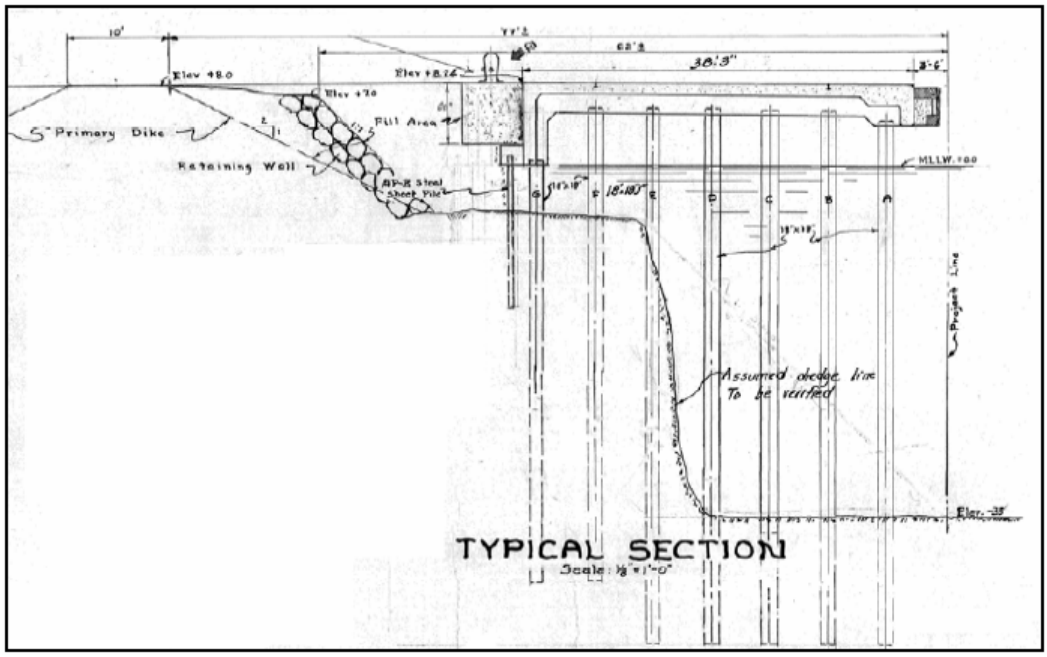


Figure 2-4 Pier 2A: Typical Section

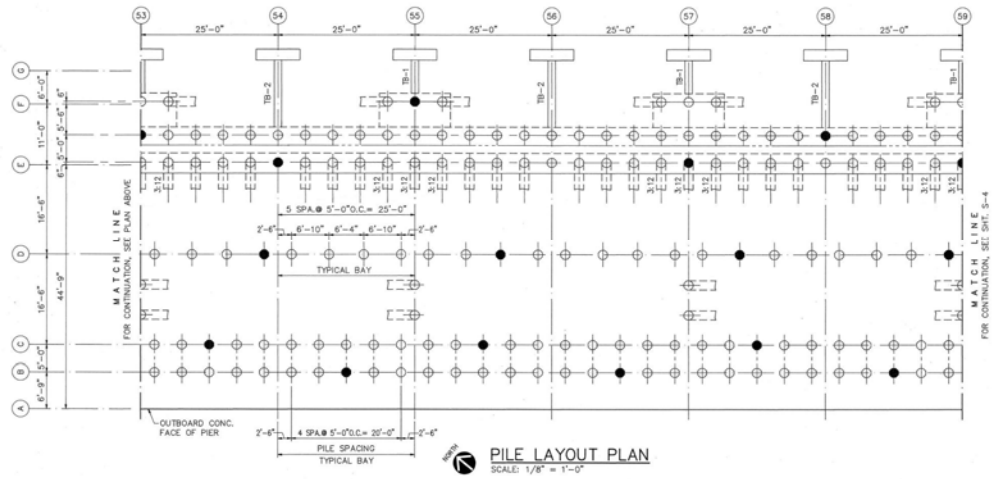


Figure 2-5 Pier 2B: Partial Pier Plan

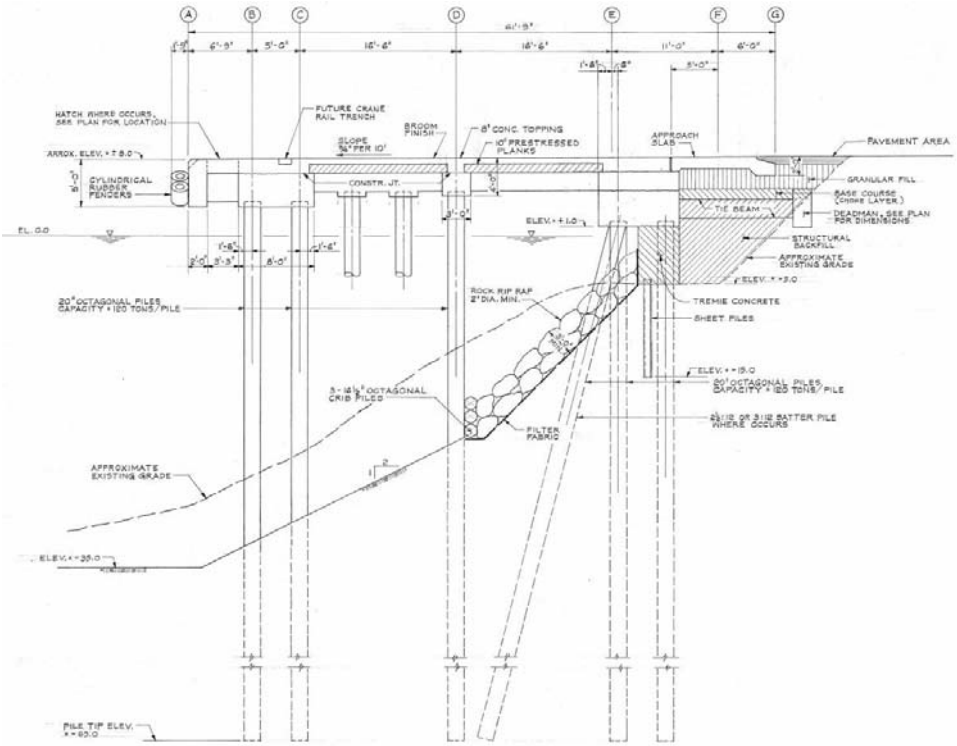


Figure 2-6 Pier 2B: Typical Section

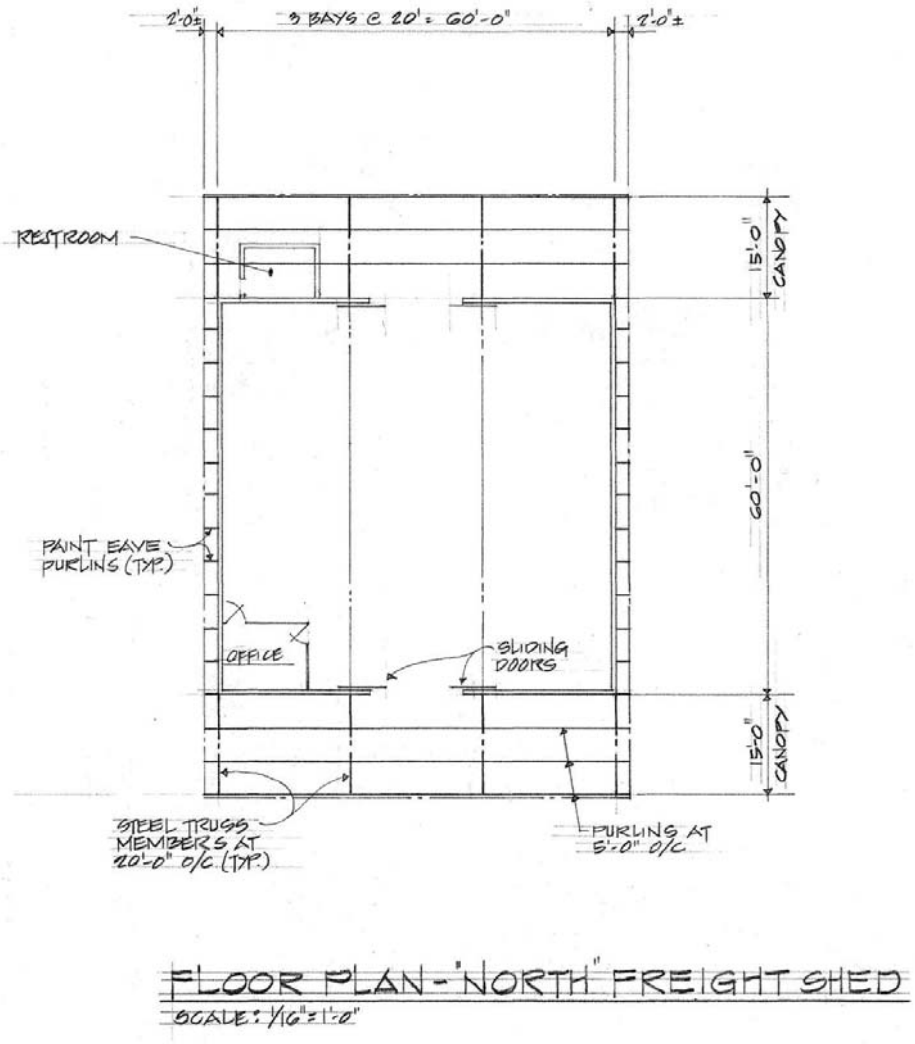


Figure 2-7 Pier 1: Floor Plan of North Metal Building

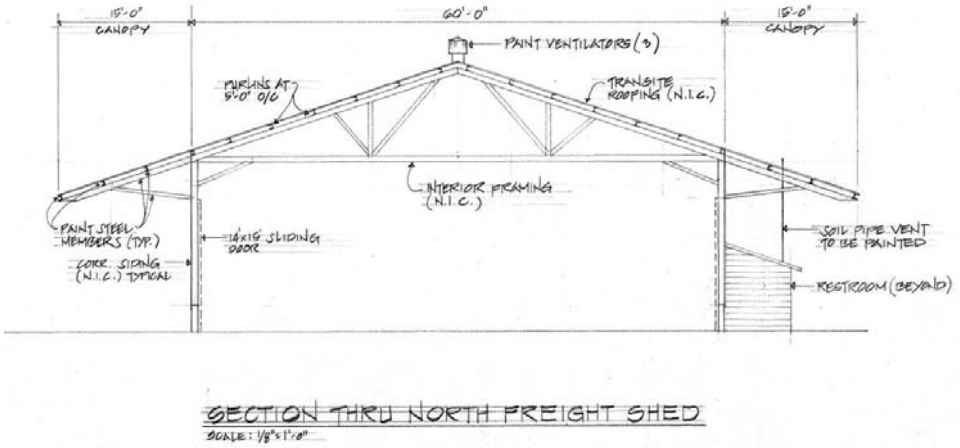


Figure 2-8 Pier 1: Section through North Metal Building

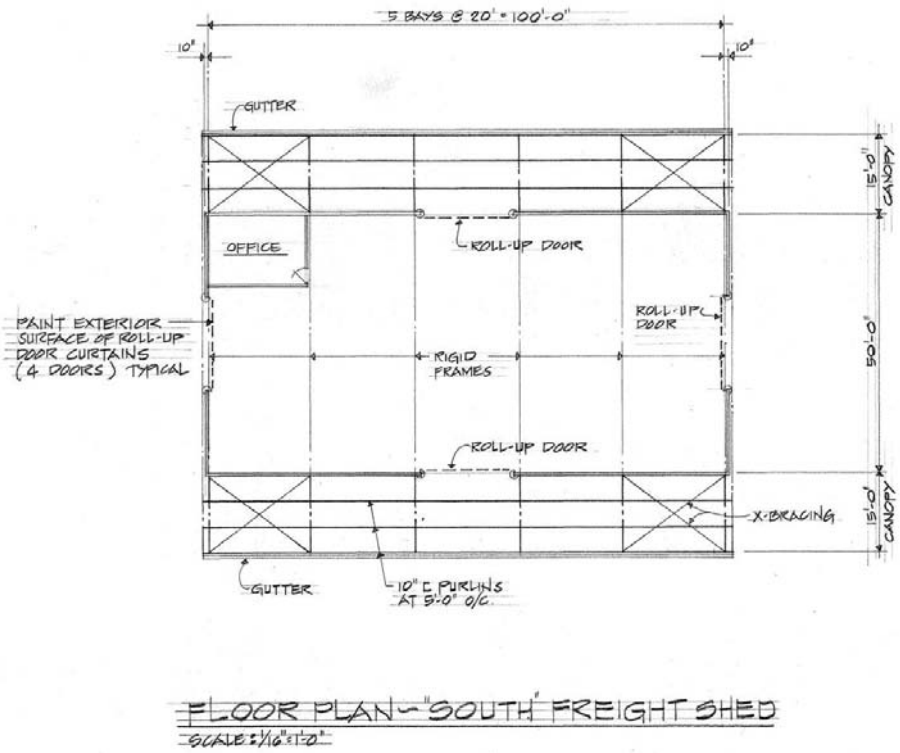


Figure 2-9 Pier 1: Floor Plan of the South Metal Building

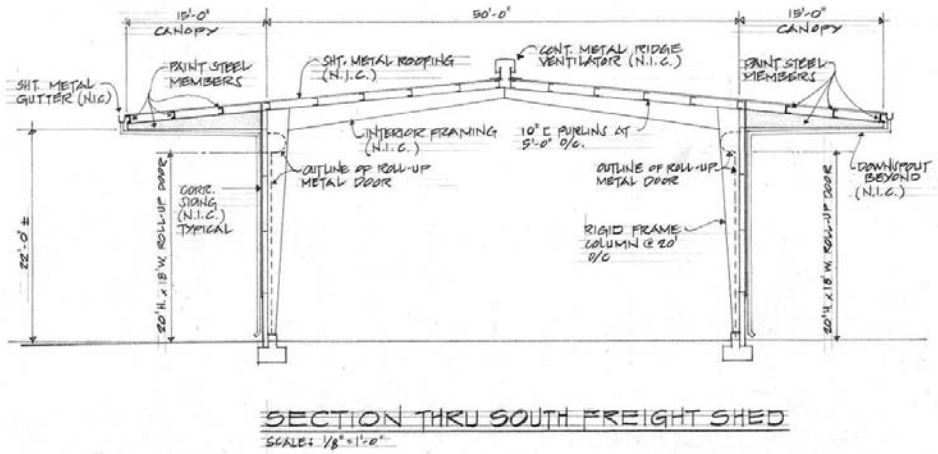


Figure 2-10 Pier 1: Section through South Metal Building

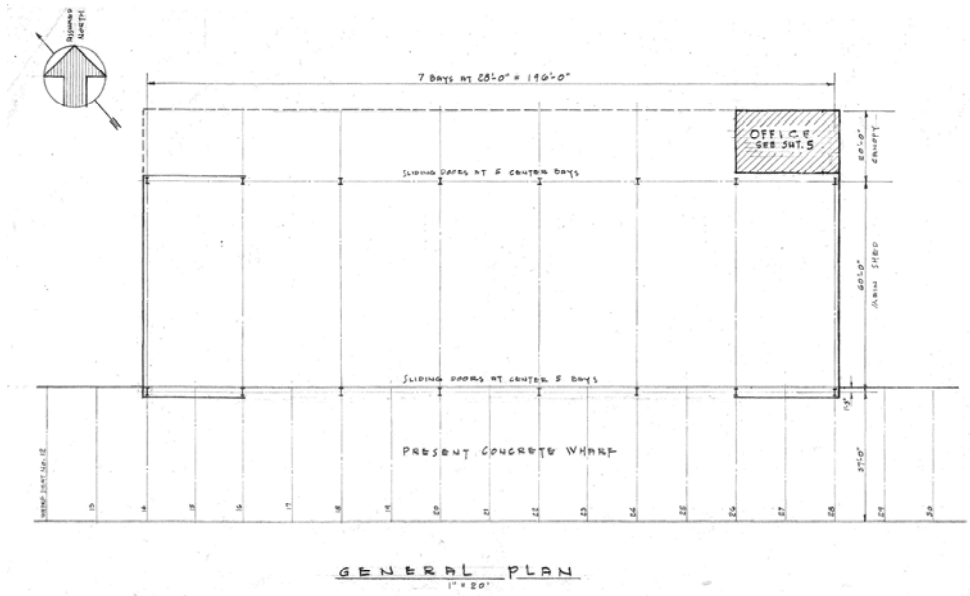


Figure 2-11 Pier 2A: Plan at Transit Shed

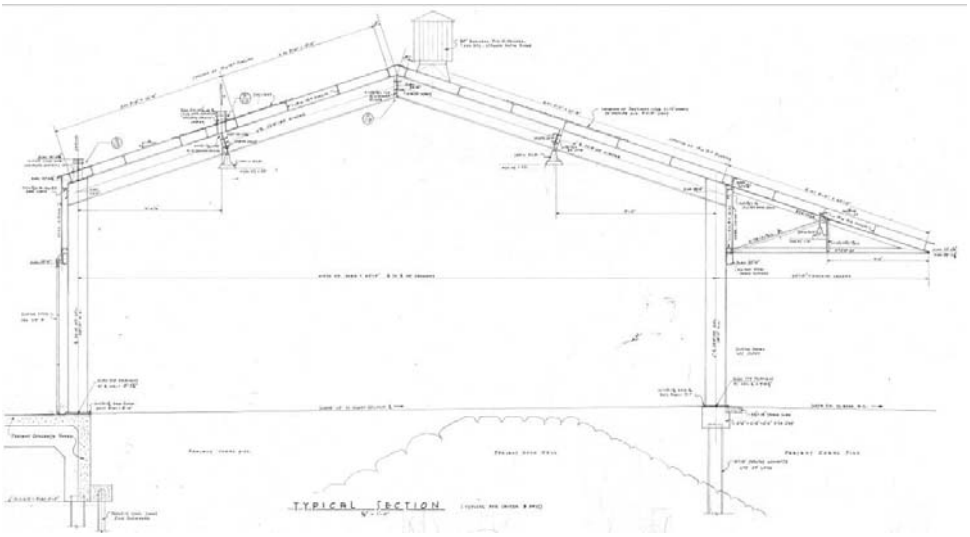


Figure 2-12 Pier 2A: Section through Transit Shed

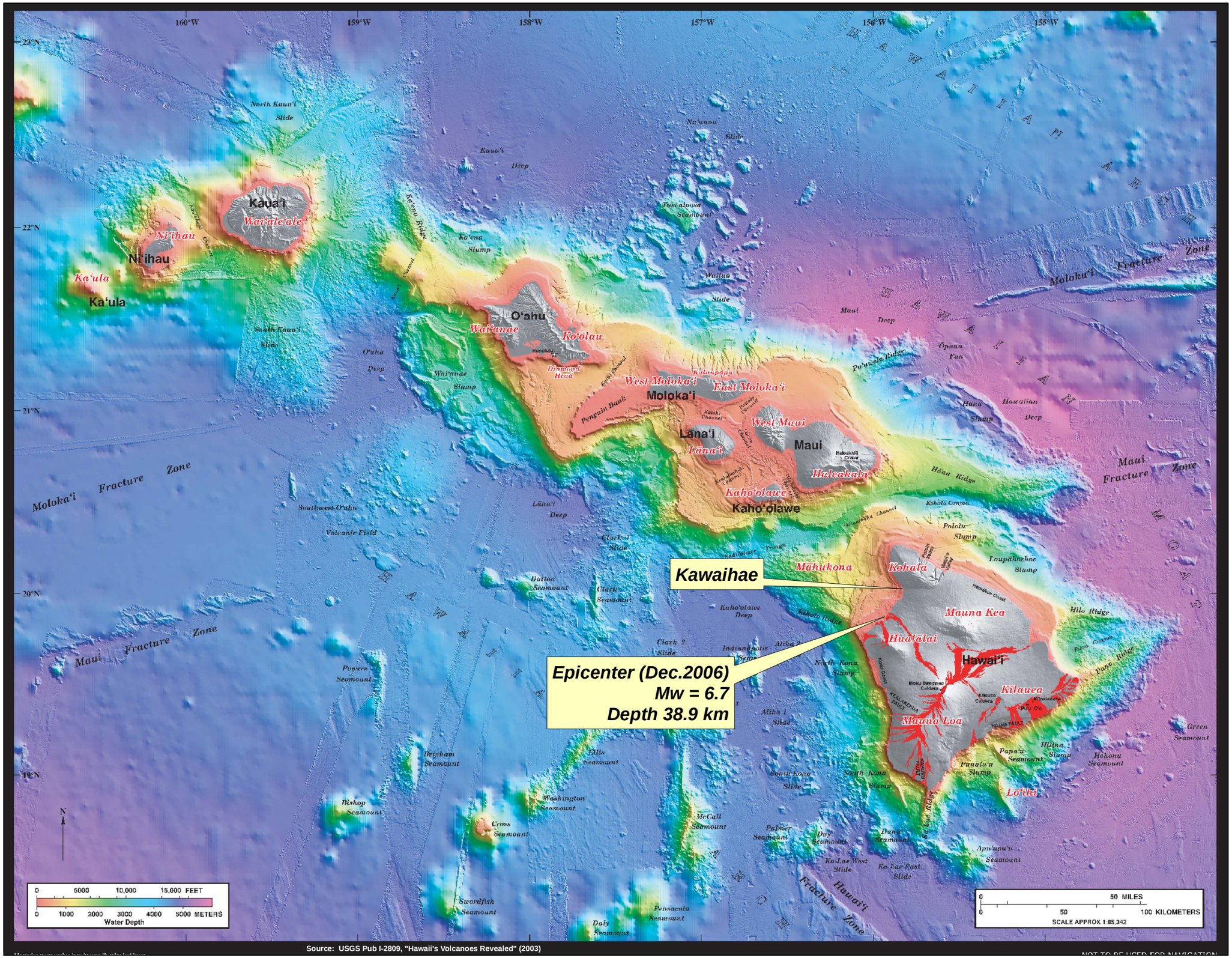


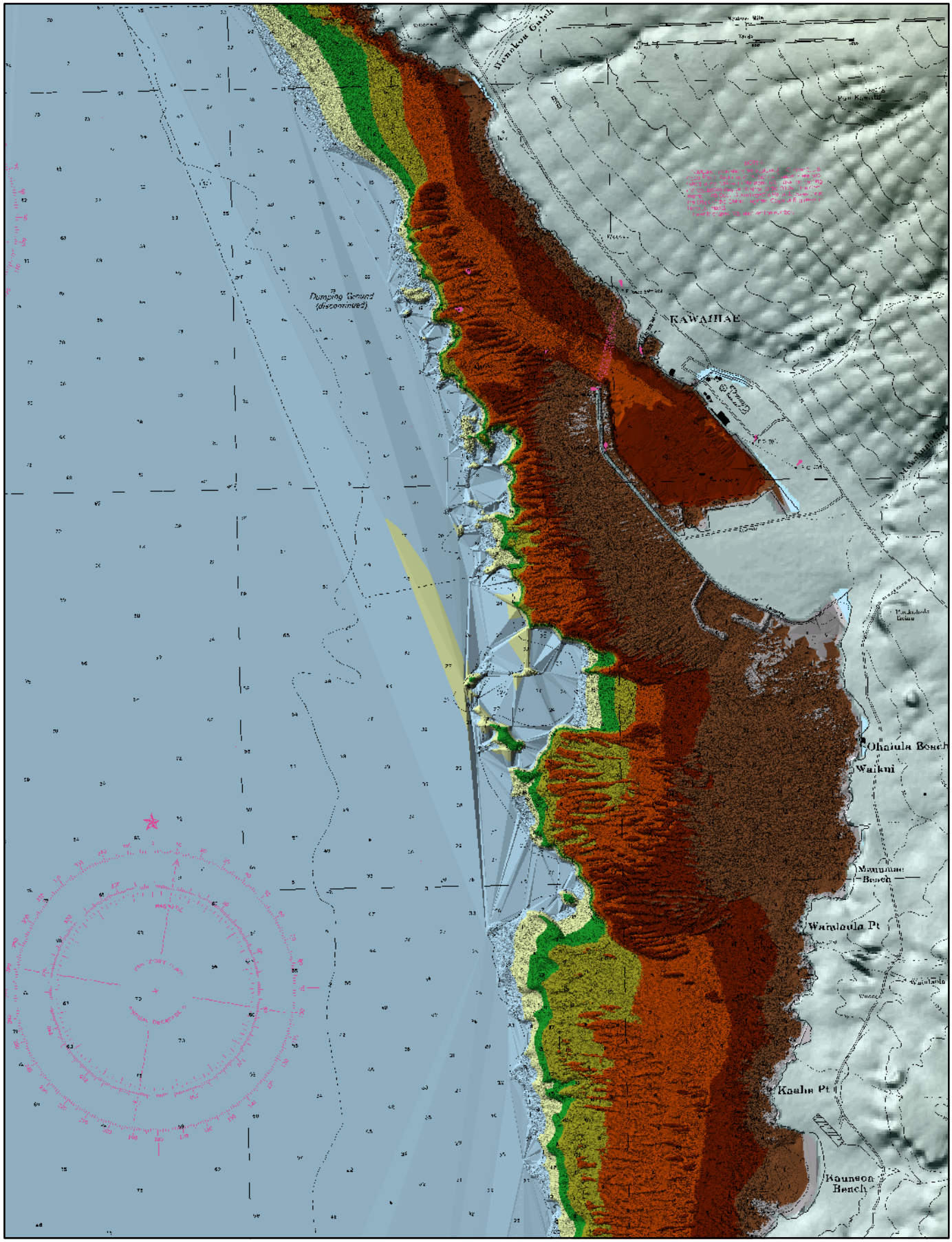
Figure 3-1.
Hawaii Bathymetry
Kawaihae Harbor
Hawaii

Source Geology: USGS, Data Series 144, "Geology of Hawaii" (1996)

Figure 3-2.
Geologic Map of Kawaiihae Vicinity
Kawaihae Harbor
Hawaii

®

Not to Scale

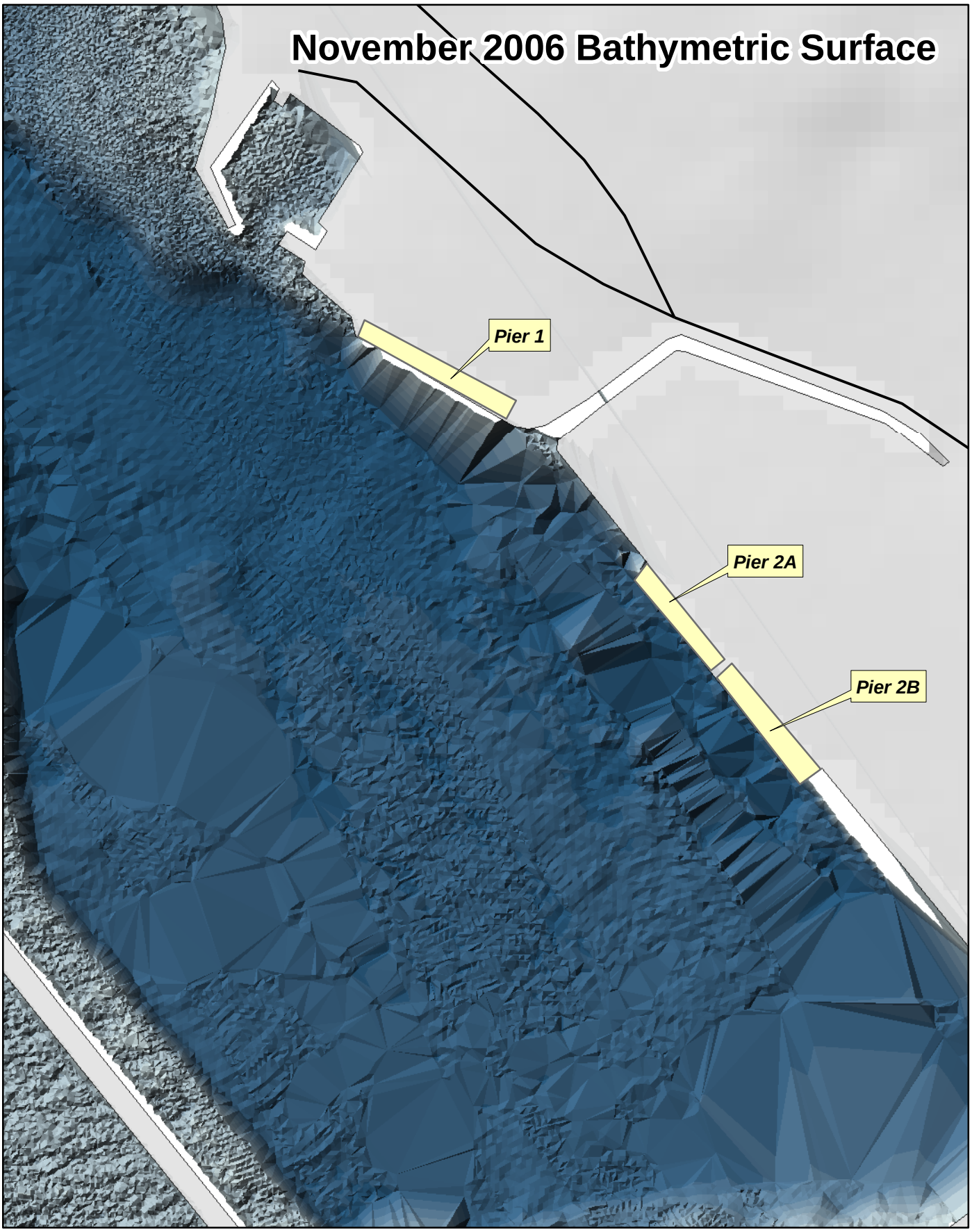


®

0 1,000 2,000 3,000

Feet

Figure 3-3.
Kawaihae Harbor Lidar
Bathymetry (1981 and 2006)
Kawaihae Harbor
Hawaii

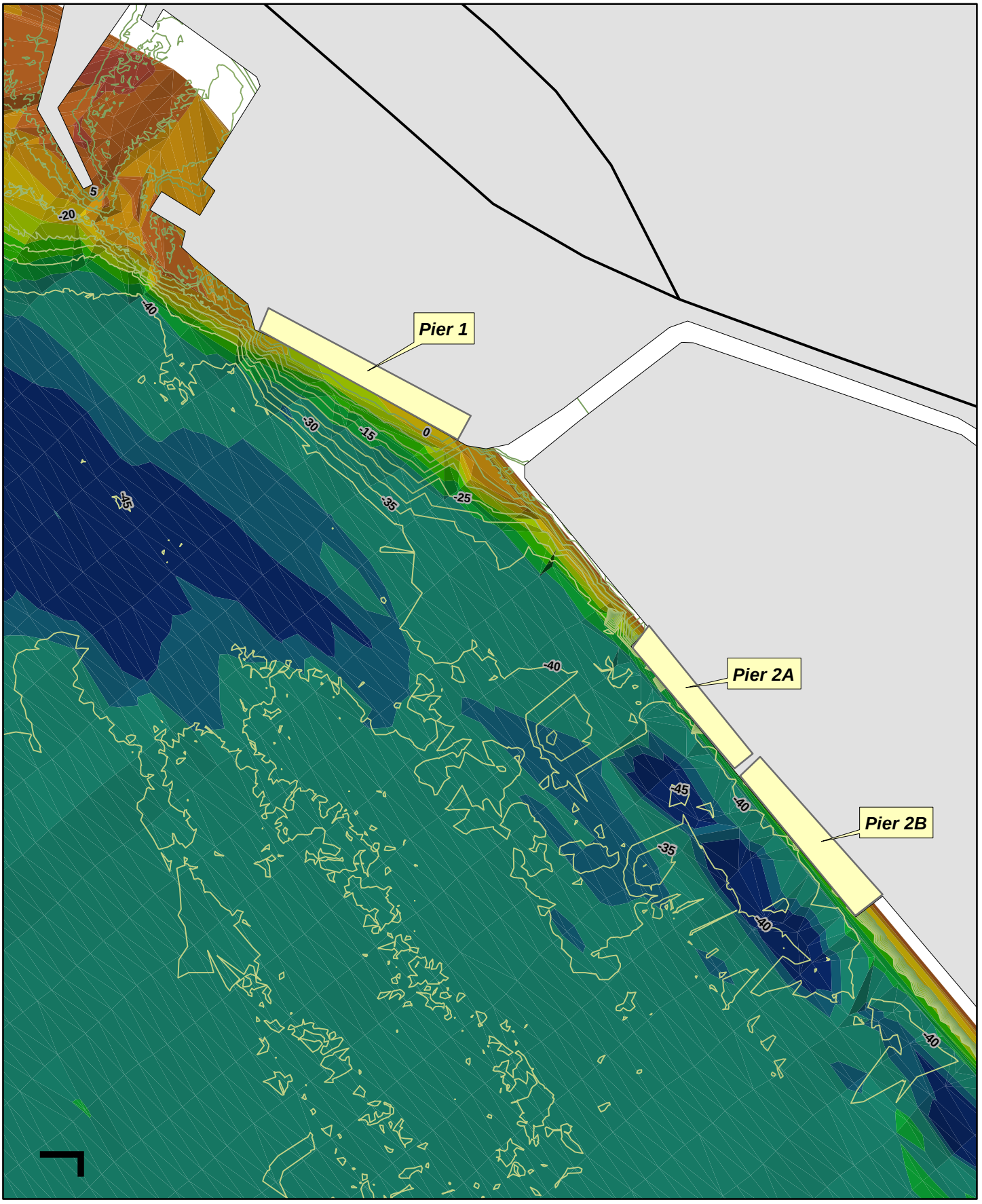


Elevation (ft mllw)

- -5 - 0
- -10 - -5
- -15 - -10
- -20 - -15
- -25 - -20
- -30 - -25
- -35 - -30
- -40 - -35
- -136.75 - -40

0 150 300 450 Feet

Figure 3-4.
Lidar Bathymetry at
Piers 1, 2A and 2B (2006)
Kawaihae Harbor
Hawaii



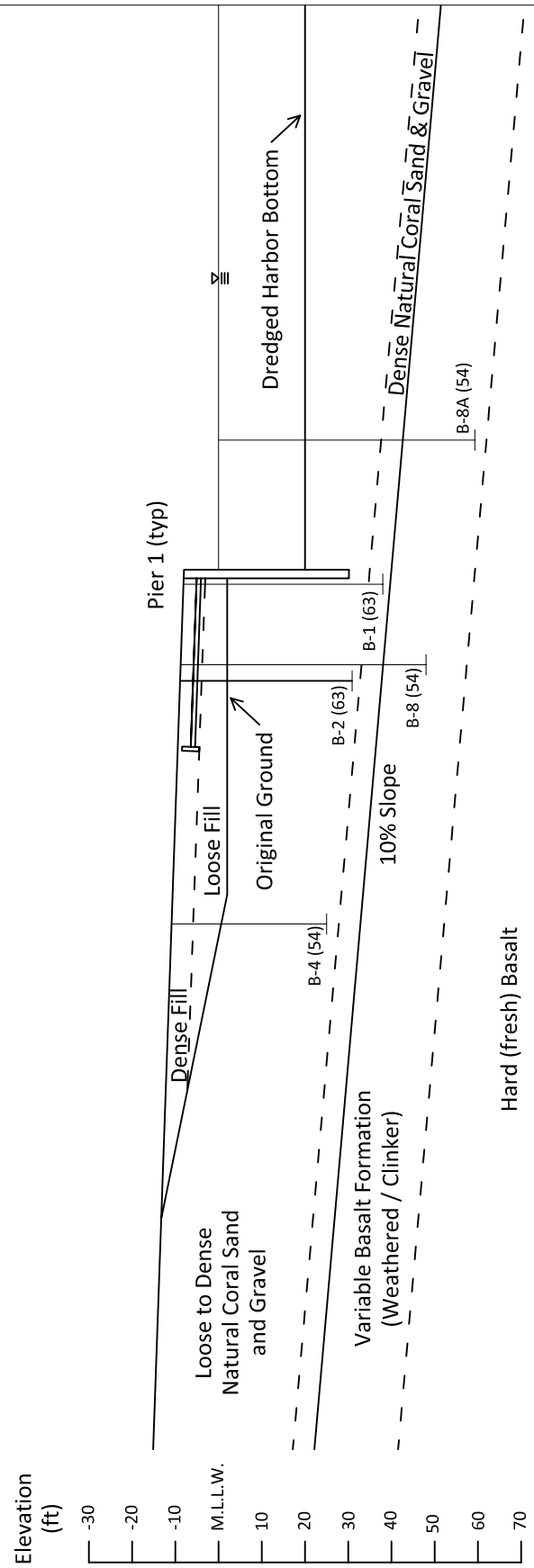
Elevation (ft mllw) (USACE data, June 2003)

- -5 - 0
- -10 - -5
- -15 - -10
- -20 - -15
- -25 - -20
- -30 - -25
- -35 - -30
- -40 - -35
- -41.68 - -40
- -109.4 - -41.68

Bathymetric Contour (Nov. 2006) SHOALS data (5 ft intervals)

0 110 220 330 Feet

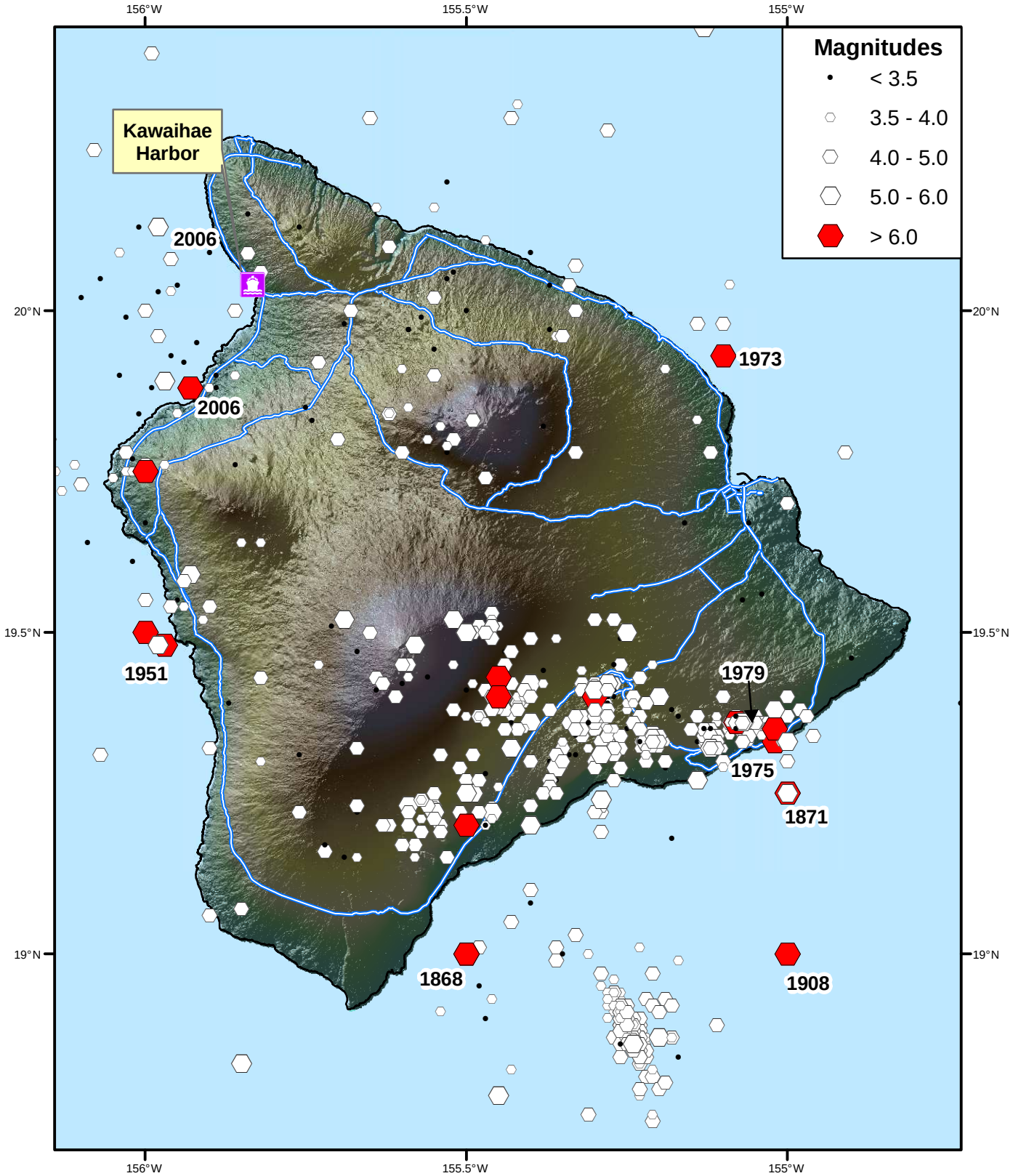
Figure 3-5.
Superimposed Lidar and Sonar Bathymetry at Piers 1, 2A, and 2B
Kawaihae Harbor
Hawaii



Scale - 1:40

Figure 3-6
Interpreted Cross-Section of Pier 1
Kawaihae Harbor
Hawaii

URS

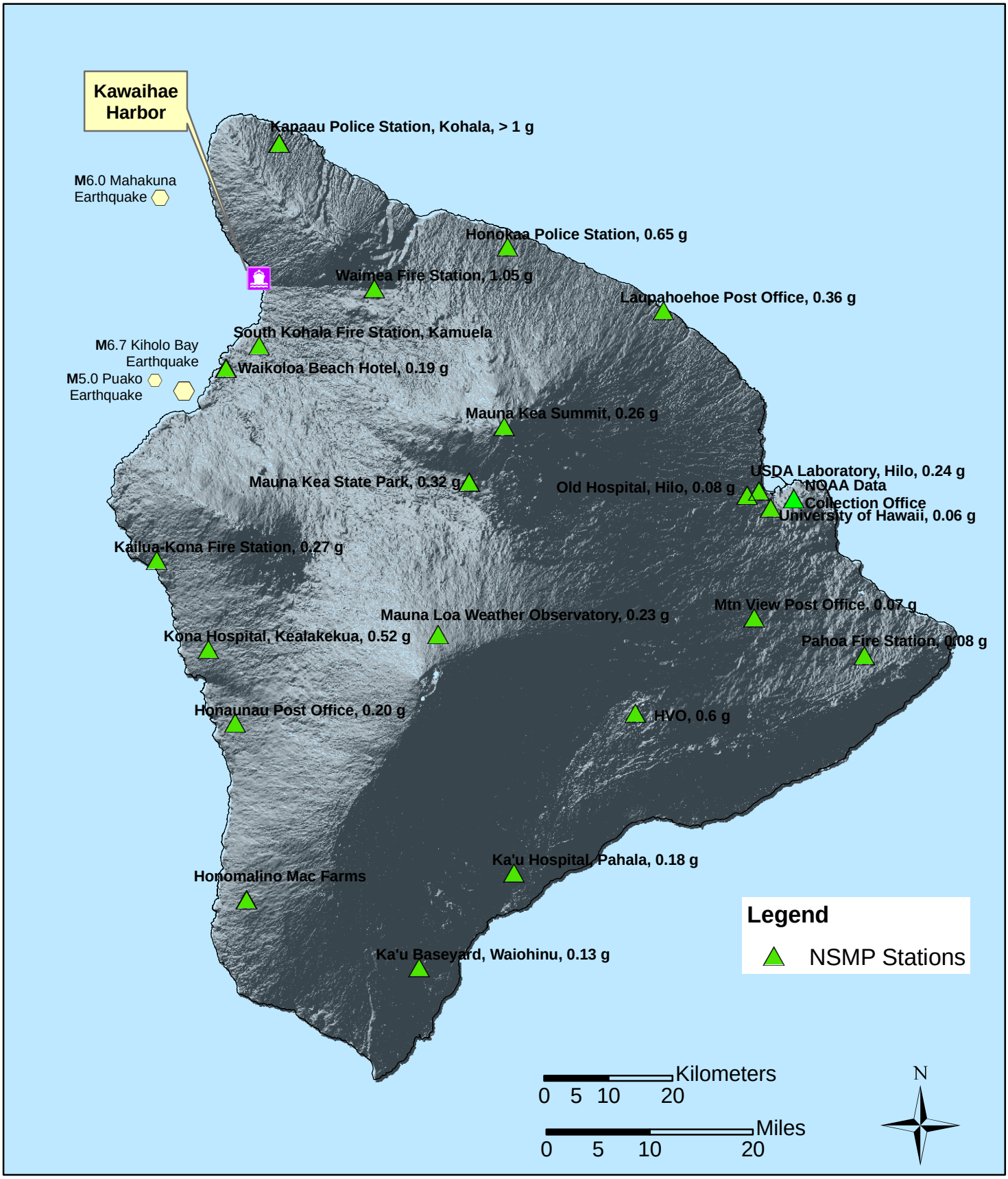


N

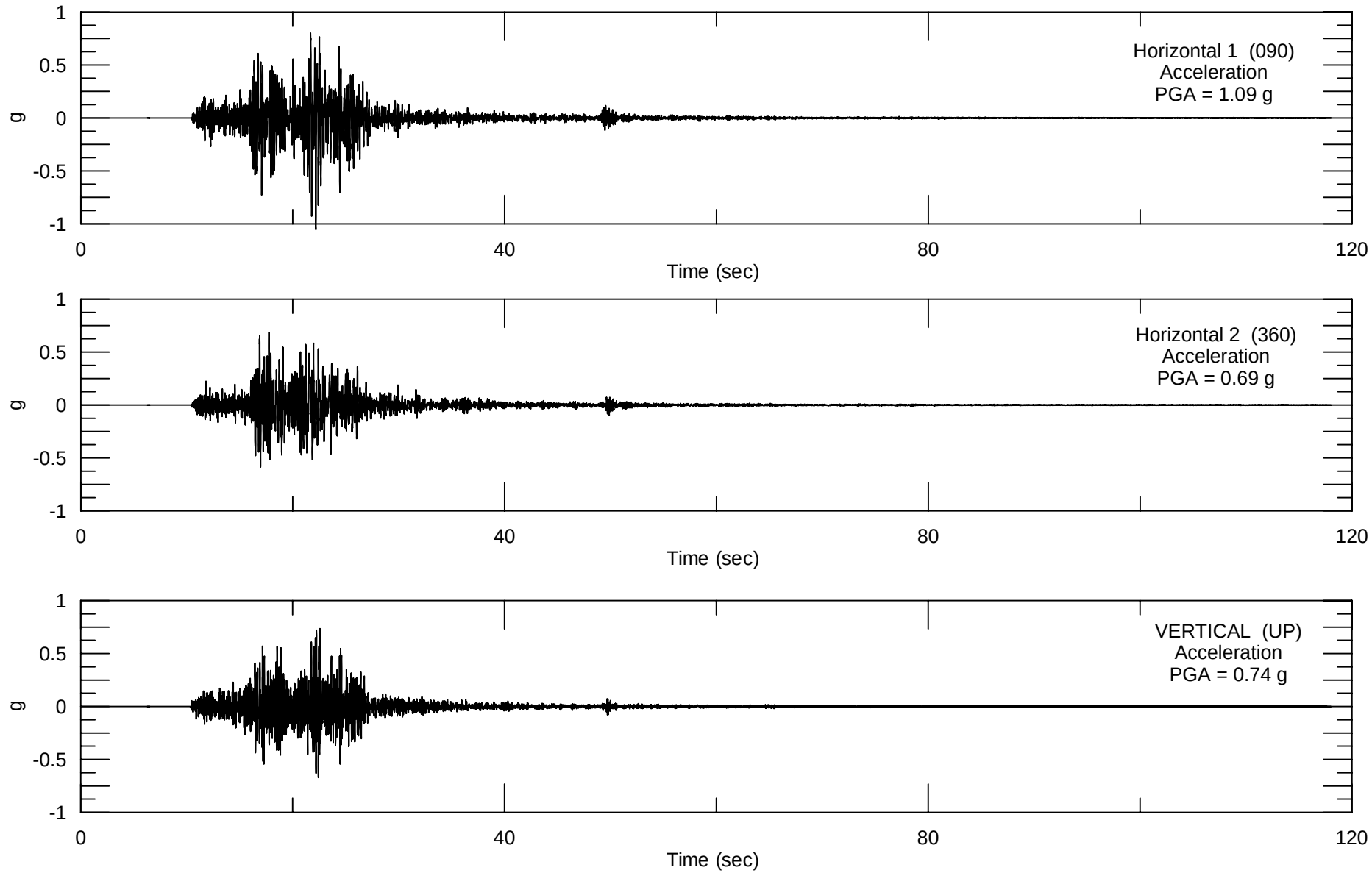
0 6 12 18 24 30 36 42 48

Kilometers

URS	Project No. 15298972 Kawaihae Harbor Hawaii	HISTORICAL SEISMICITY IN THE HAWAII REGION 1823-2007	Figure 4-1



URS	Project No. 15298972 Kawaihae Harbor Hawaii	USGS HAWAII STRONG MOTION NETWORK AND RECORDED PGAs FROM THE 2006 KIHOLO BAY EARTHQUAKE	Figure 4-2



Station-USGS 2825
Waimea Fire Station
NEHRP site class Unknown (Vs30= < 30 m/sec)
Geologic Description: Rock
Kiholo Bay, Hawaii 2006 **M**6.7
Closest Distance=50.9 km

URS

Project No. 15298972

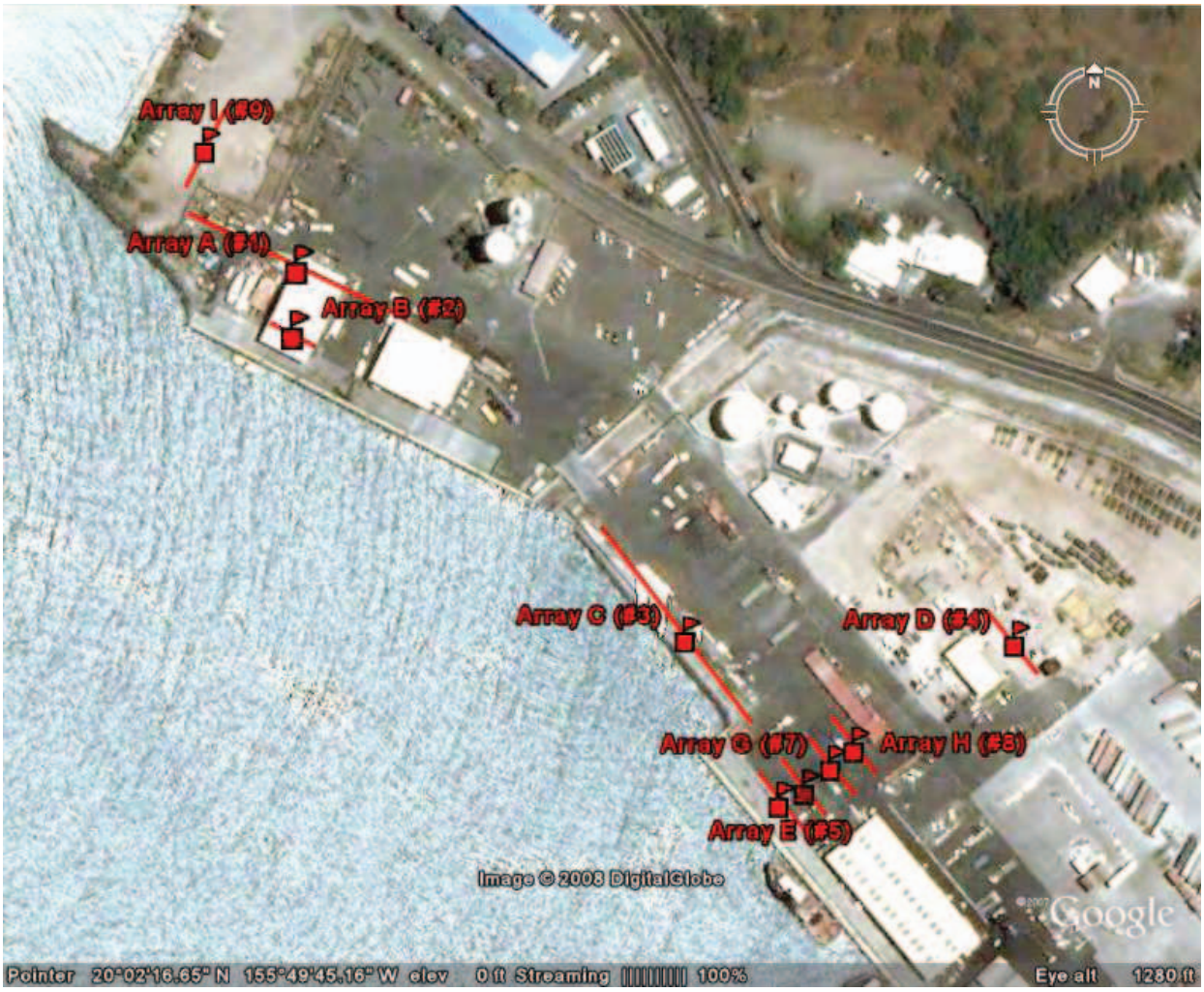
Kawaihae Harbor,
Hawaii

WAIMEA FIRE STATION STRONG
MOTION RECORDS
2006 KIHOLO BAY EARTHQUAKE

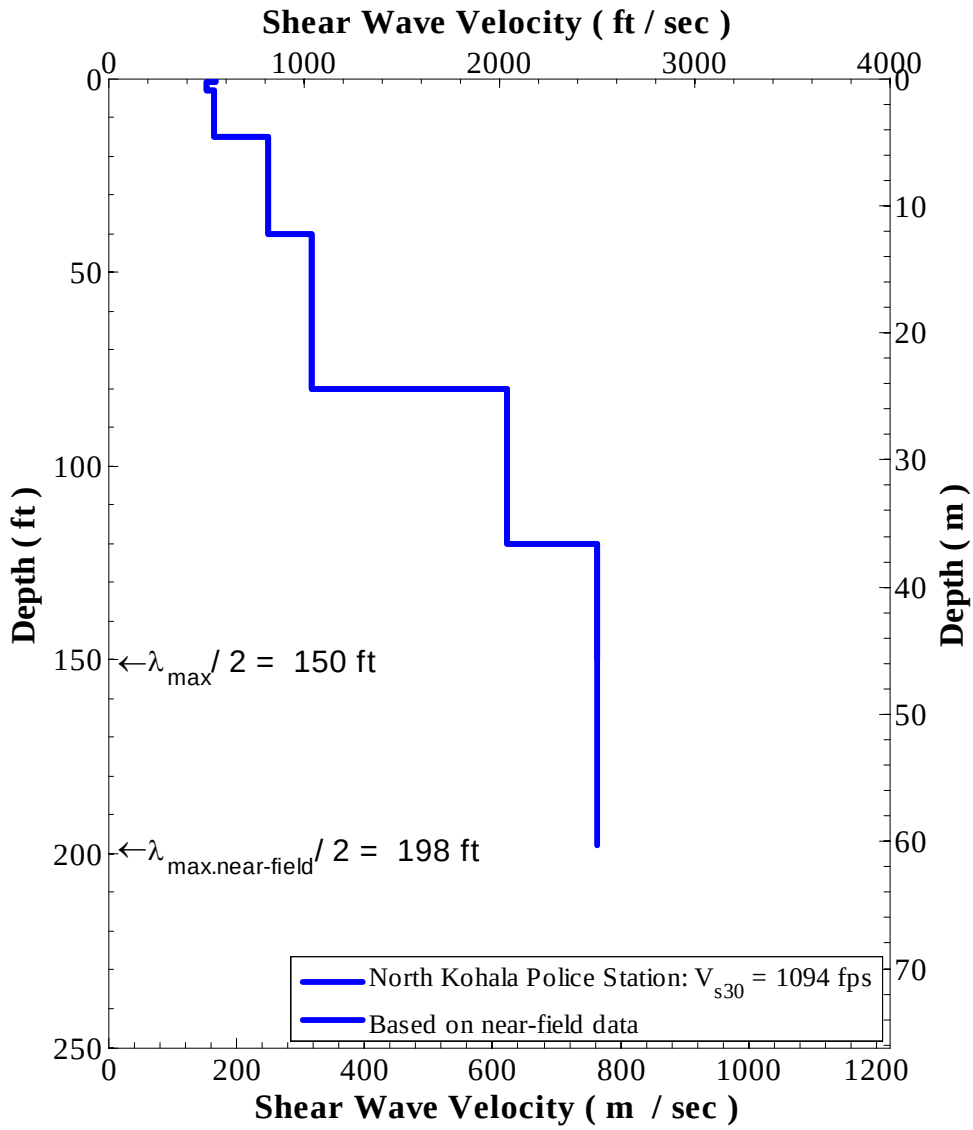
Figure
4-3

URS	Project No. 15298972 Kawaihae Harbor Hawaii	SASW SURVEYS	Figure 4-4

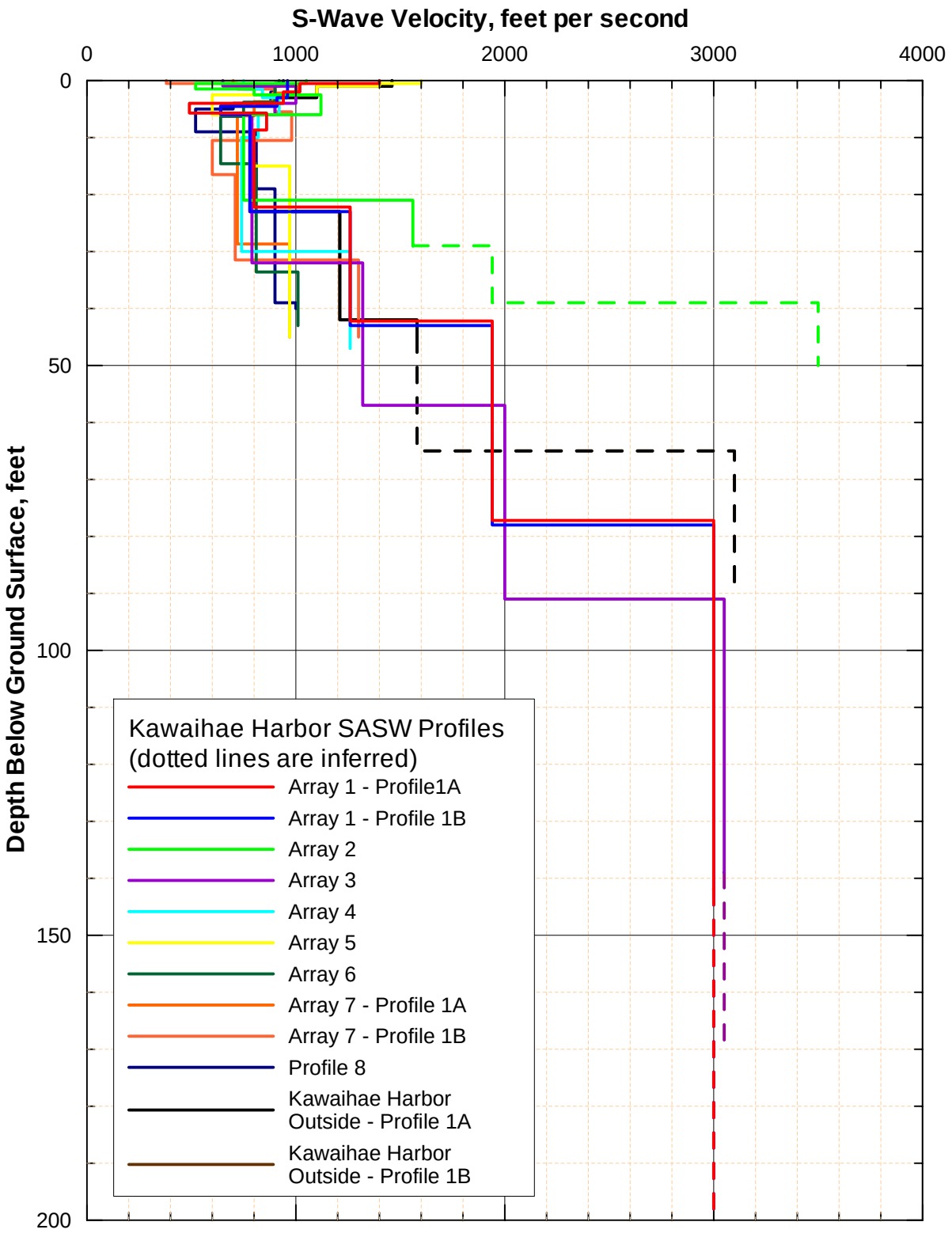
URS	Project No. 15298972 Kawaihae Harbor Hawaii	SASW SURVEYS	Figure 4-5



URS	Project No. 15298972 Kawaihae Harbor Hawaii	SASW ARRAYS USED IN KAWAIHAE HARBOR	Figure 4-6

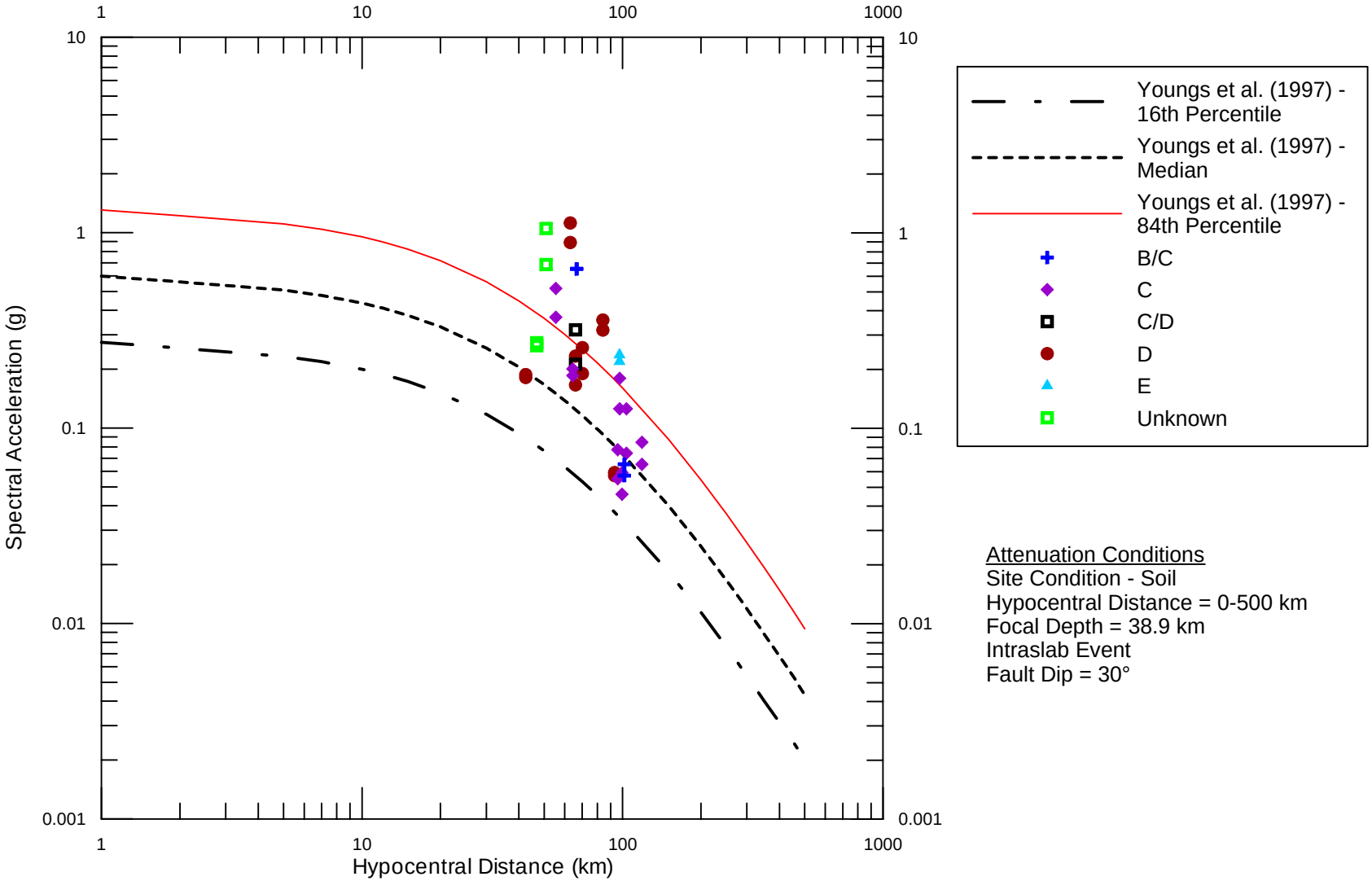


URS	Project No. 15298972 Kawaihae Harbor Hawaii	SHEAR-WAVE PROFILE FOR THE NORTH KOHALA POLICE STATION	Figure 4-7



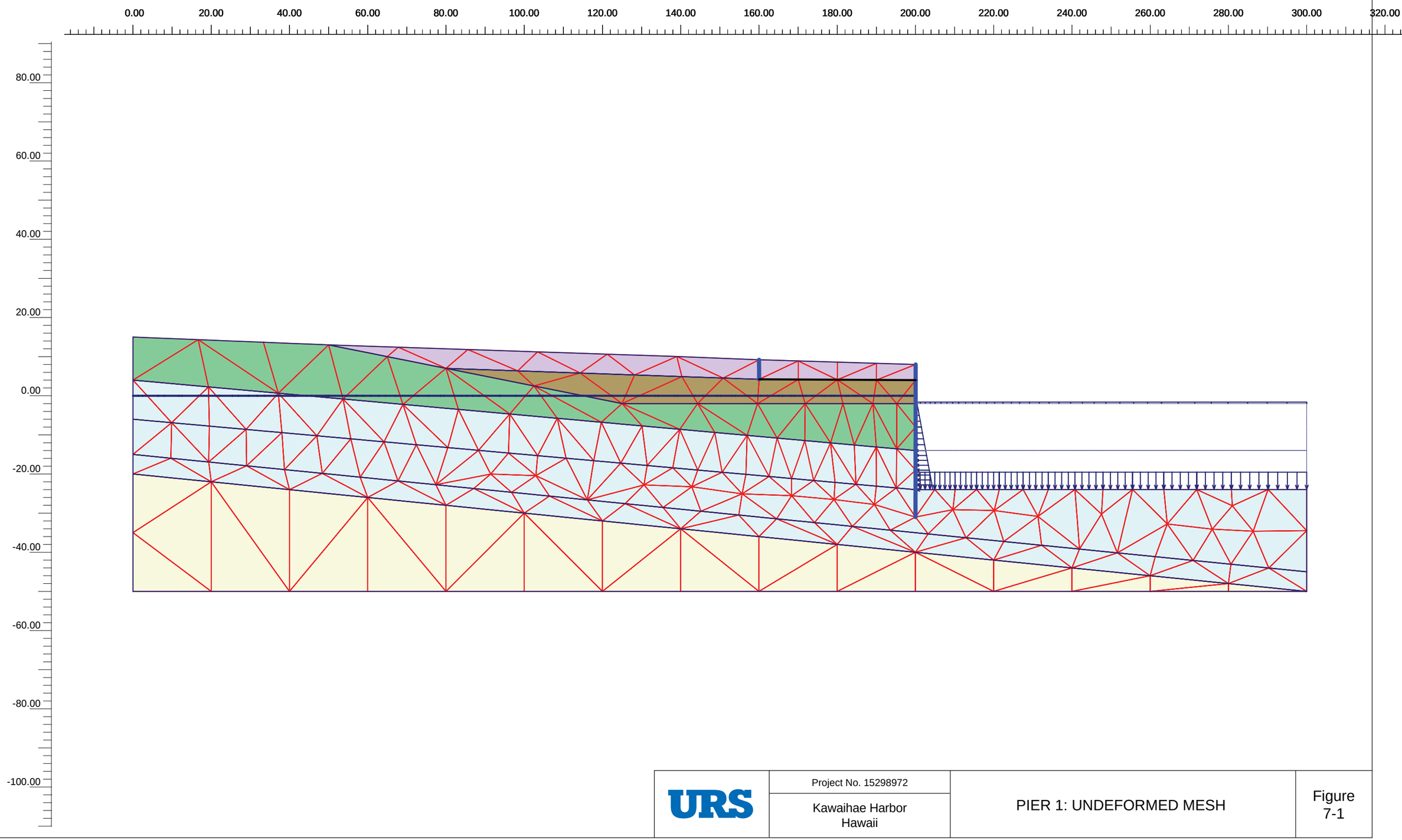
URS	Project No. 15298928 Kawaihae Harbor Hawaii	SASW PROFILES FOR KAWAIHAE HARBOR	Figure 4-8

W:\x_wcfs\projects\Kawaihae Horbor\SASW Profiles\FINAL Kawaihae Harbor SASW.grf



URS	Project No. 15298972 Kawaihae Harbor Hawaii	COMPARISON OF YOUNGS ET AL. (1997) ATTENUATION RELATIONSHIP FOR SOIL AND **M** 6.7 KIHOLO BAY EARTHQUAKE STRONG MOTION DATA AND NEHRP SITE CLASS	Figure 4-9

PLAXIS V8



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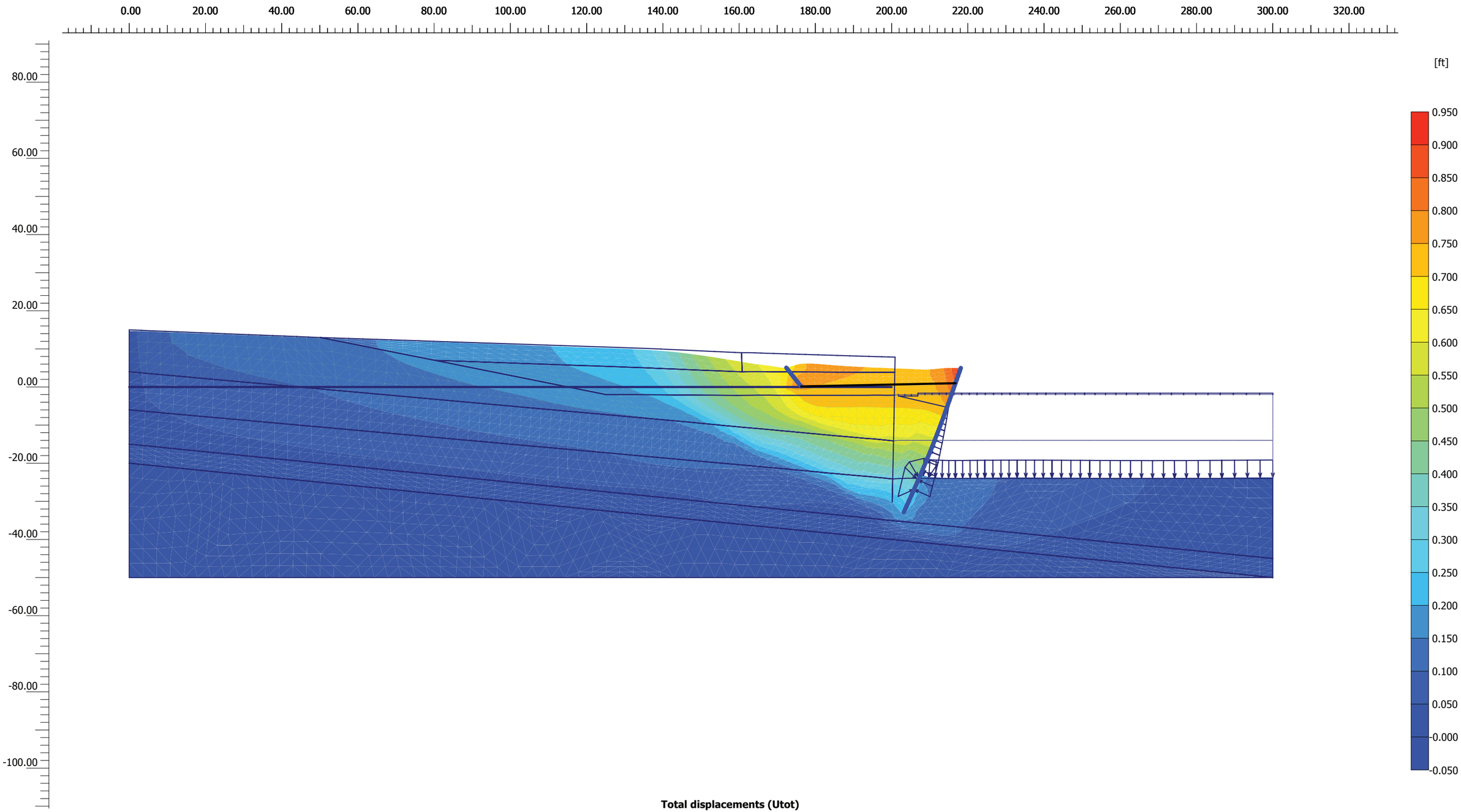
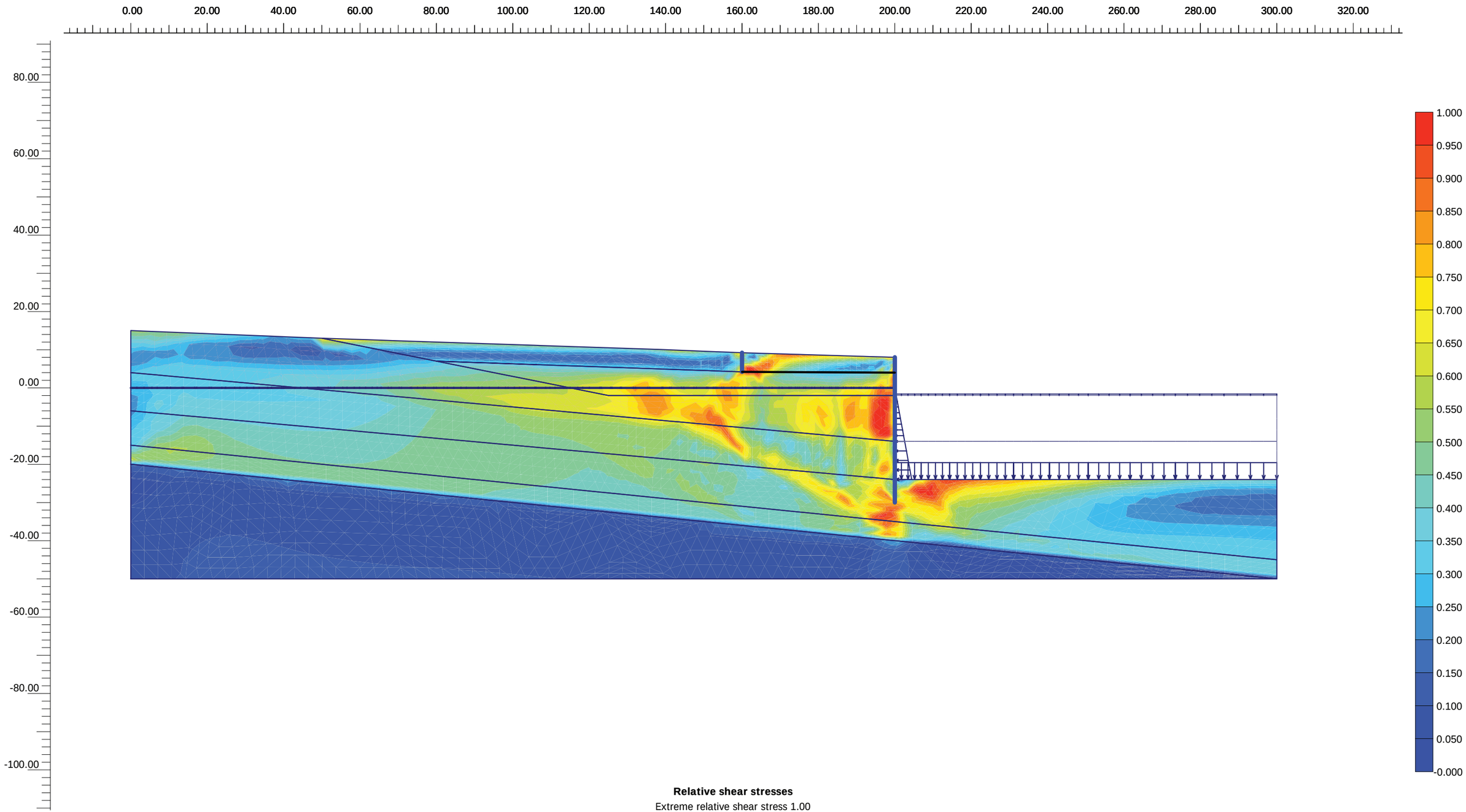


URS	Project No. 15298972 Kawaihae Harbor Hawaii	PIER 1: DYNAMIC ANALYSIS WITH UPDATED MESH	Figure 7-2

PLAXIS V8



Relative shear stresses

Extreme relative shear stress 1.00

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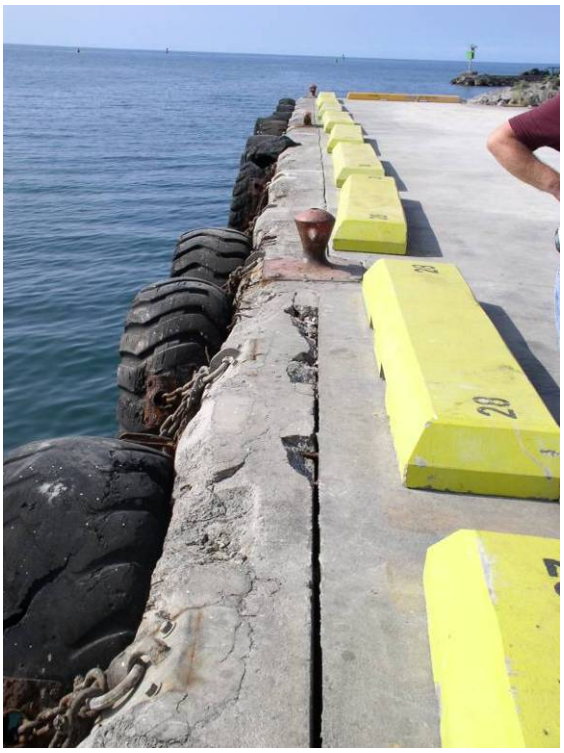

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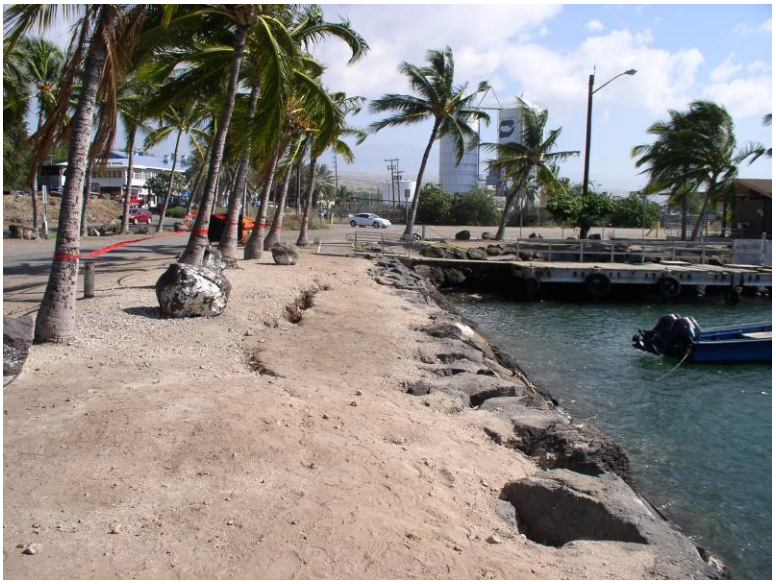

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Appendix B

Comparison of Different Codes and Design Procedures

B.1 1958 UNIFORM BUILDING CODE

The 1958 edition of the Uniform Building Code (UBC) (ICBO 1958)was the probable guideline for the design of Pier 2A, which is a concrete wharf structure with vertical concrete piles.

The horizontal force formula is as follows:

$$F = CW$$

where F = horizontal force in pounds

C = a numerical constant obtained from Table 23-C

W = total dead load tributary to the point of horizontal force F

It is assumed that Pier 2A was classified with the first group listed under "Part or Portion" in Table 23-C, and the C value is determined using the following equation:

$$C = \frac{0.15}{N + 4\frac{1}{2}}$$

N is the number of stories above the story under consideration that are contributing loads. For the one-story wharf, N = 1 and yields a C value of 0.027.

The code indicates that the C values summarized in Table 23-C are applicable only to structures located within Zone 1. Based on a 1956 Board of Supervisors Ordinance that states that the Earthquake Zone for the island of Oahu will be 1 and assuming that the island of Hawaii is considered to fall within the same zone, the calculated C value is multiplied by 1. The resulting horizontal force F is then 0.027W.

B.2 1985 UNIFORM BUILDING CODE

The 1985 edition of the UBC (ICBO 1985) is distinctly different from the simplistic approach outlined in the 1958 edition. It is clear in comparing the lateral design approach of the two editions that significant strides were made in the field of seismic engineering over the intervening 30 years. Pier 2B was designed and built under the code requirements of the 1985 edition of the UBC. Pier 2B is a concrete wharf with vertical and battered concrete piles.

The minimum earthquake forces for structures is calculated using Equation 12-1 in the code:

$$V = ZIKCSW$$

where V = total lateral base shear

Z = numerical coefficient dependent on the zone where the structure is located (as obtained from the Seismic Zone Map)

I = Occupancy Importance Factor per Table 23-K

K = numerical coefficient per Table 23-I

C = numerical coefficient as specified in Section 2312(d)

S = numerical coefficient for site-structure resonance

W = total dead load

The coefficient C is determined by the following equation:

$$C = \frac{1}{15\sqrt{T}}$$

where T is the fundamental elastic period of the structure. Insufficient detailed information is available regarding the pier's true fundamental period. Rather than calculating the C value, the maximum allowable C of 0.12, as stated in the code, will be used.

The value of S depends on the structure's period T and the characteristic site period T_s, both of which are unknown. When the value of T_s is not properly determined, the code permits the value of S to be taken as 1.5. Using S = 1.5 and C = 0.12, the C x S = 0.18. The code limits the product of C and S to a maximum value of 0.14.

The wharf is located in a region designated as Zone 3 under the 1985 code, so Z = 3/4. With an Occupancy Importance Factor *(I)* of 1.0 (from Table 23-K) and a horizontal force factor K for a general framing system of 1.00 from Table 23-I, the horizontal base shear V is equal to 0.11W.

B.3 1991 UNIFORM BUILDING CODE

The 1991 edition of the UBC (ICBO 1991) is the edition currently in use for all new design on the island of Hawaii. Under this code, a concrete wharf structure would be classified as a nonbuilding structure as defined in Section 2338. This definition describes nonbuilding structures as "…all self-supporting structures other than buildings which carry gravity loads and resist the effects of earthquake." Section 2338(b) specifies that the structural systems for a nonbuilding structure can be obtained from Table No. 23-O for structures similar to the building systems listed. Assuming that the concrete wharf is a type of concrete Special Moment Resisting Frame (SMRF), the computation of the lateral load on a nonbuilding structure is the same as for building structures. The expression for total design base shear is as follows:

$$V = \frac{ZIC}{R_w} W$$

where V = total design lateral base shear

Z = seismic zone factor

I = importance factor per Table 23-L

R_w = numerical coefficient per Tables 23-O or 23-Q

W = total dead load on structure

The coefficient C is determined from the following equation:

$$C = \frac{1.25S}{T^{2/3}}$$

Since the period and site-specific data are unknown, the maximum value of 2.75 for C for any structure and soil can be used. It is determined from Figure 23-2 of the code that the wharf falls within Zone 4. Based on Table 23-I, the seismic zone factor Z = 0.40. The importance factor I is 1.0 for standard occupancy structures.

For a concrete SMRF, R_w equals 12. The R_w factor in the 1991 UBC was a factor by which the elastic base shear was reduced to a working stress (or allowable stress) design level.

Using the values summarized above, the total base shear equation is calculated to be $V = 0.092W$. In Section 2609.3 of the 1991 code, the design of the members required applying a load factor of 1.1 x 1.3 = 1.43 to seismic loads. This load factor was to account for various uncertainties. Therefore, the horizontal base shear V would be equal to $0.131W$.

B.4 1997 UNIFORM BUILDING CODE

Studies conducted since 1992 have concluded that Hawaii is located in a region that may be at greater risk for strong ground shaking than previously assumed. Consequently, the zone designation for lateral design was upgraded from Zone 3 to Zone 4 under the 1997 UBC (ICBO 1997). The base shear calculations in this edition have been refined somewhat from those that appeared in the 1991 edition, but the same approach is used for nonbuilding structures such as wharves. A concrete special moment-resisting frame is again assumed for illustrating the procedure for computing seismic forces on the pier under the updated standards.

The total design base shear formula is as follows:

$$V = \frac{C_v I}{RT} W$$

where V = total design lateral base shear

C_v = seismic coefficient

I = importance factor per Table 16-K

R = numerical coefficient representative of the inherent overstrength and global ductility capacity of the lateral-force-resisting systems.

T = fundamental period of vibration

W = total dead load on structure

The values for C_v, I, and R are obtained from Tables 16-R, 16-K, and 16-P, respectively. Section 1629.3 states that Soil Profile Type S_D must be used in the absence of accurate soil data. From this information, it is found from Table 16-R that $C_v = 0.64Nv$. The near-source factors N_v and N_a were incorporated into the code, to recognize the amplified ground motion based on the proximity to known faults and the type of fault involved. The near-source factor for Hawaii has never been defined or mapped. Assuming a seismic source Type A that is less than 2 kilometers (1.2 miles) away, N_v would equal 2.0. Seismic source types A are faults that are capable of producing large-magnitude events and that have a high rate of seismic activity. The importance factor I is 1.0 and R is 8.5 for concrete special moment resisting frames. This new R factor is different from the R_w factor that was previously used in that the R_w factor reduced the base shear to an allowable stress level, and the R factor reduces the base shear to the strength design level.

Without knowledge of the actual period T, the maximum allowable base shear is utilized for design. This is calculated from the following formula:

$$V_{\max} = \frac{2.5 C_a I}{R} W$$

From Table 16-Q, $C_a = 0.44N_a$. Again assuming a Type A seismic source that is less than 2 kilometers (1.2 miles) away, $N_a = 1.5$. V_{max} is then calculated to be $0.194W$. The seismic load factor for concrete member design is 1.0 for this code.

Since the wharf is in seismic Zone 4, the minimum limit of base shear is the greater of:

$$V_{\min} = 0.11C_a IW$$

which yields $V_{min} = 0.073W$, or

$$V_{\min} = \frac{0.8ZN_v I}{R}W$$

which yields $V_{min} = 0.075W$.

The final base shear calculation to be used in the design of a concrete wharf under the 1997 code regulations is $0.194W$ (Table B-1).

Table B-1 Summary of Base Shear Coefficients

Code Edition	Base Shear *V*
1958	$0.027W$
1985	$0.110W$
1991	$0.131W$
1997	$0.194W$

B.5 UNITED FACILITIES CRITERIA 2005 AND MOTEMS 2003

The Unified Facility Criteria (UFC) has based its seismic design criteria on the requirements of the 2003 edition of the International Building Code (IBC) and American Society of Civil Engineers (ASCE) 7-02, Minimum Design Loads for Buildings and Other Structures.The UFC seismic design guidelines begin with determining the Seismic Use Group (SUG), which is a classification assigned to a structure based on its use. After reviewing the group descriptions listed in UFC Section 3-4.5.1, we have assumed that a wharf structure would be classified as SUG II, which refers to structures having moderate importance where the impact of being out of operation for repairs is tolerable (moderate risk). The high risk groups are reserved for structures that are considered essential facilities, which could be the case for some facilities, depending on the use of the facility.

The next step is identifying the Seismic Design Category (SDC) for the structure. The SDC is a classification assigned to a structure based on its SUG and the severity of the design earthquake ground motion at the site. This category designation is based on the 2003 edition of the IBC, Section 1616. The initial steps involve determining the mapped spectral accelerations S_S and S_1 based on the exact latitude and longitude of the site. Using these values and the site class definitions described in IBC Table 1615.1.1, the site coefficients F_a and F_v are determined. The earthquake spectral response acceleration values S_{MS} and S_{M1} are then calculated from the site coefficients and the mapped spectral accelerations. Finally, the design spectral response

accelerations in the short period and 1.0 second period S_{DS} and S_{D1}, respectively, can be computed. Table B-2 summarizes the ground motion data specific to Kawaihae Harbor.

Table B-2
Site Ground Motion Data for Kawaihae Harbor

Latitude	20.03887
Longitude	155.83167
S_S	1.262 g
S_1	0.454 g
Site Class	D
F_a	1.000
F_v	1.546
S_{MS}	1.262 g
S_{M1}	0.702 g
S_{DS}	0.841 g
S_{D1}	0.468 g

Using the procedure described above, the Seismic Design Category for the Kawaihae Harbor is Category D. It should be noted that the values utilized in determining the SDC are highly site specific, and each structure will have a unique set of data corresponding to its particular geographic location.

The Table in UFC Section 3-4.5.3 notes that a structure in Seismic Use Group II and Seismic Design Category D requires a performance-based seismic design in accordance with the provisions of the Marine Oil Terminal Engineering and Maintenance Standards (MOTEMS) (CBSC 2003) rather than determining the seismic forces utilizing the Equivalent Lateral Force procedure from ASCE 7.

The purpose of the MOTEMS is to establish the minimum engineering, inspection, and maintenance criteria for marine oil terminals to prevent oil spills and to protect the public's health and safety and the environment. The provisions of these standards have also been adopted for use on piers and wharf structures that are used for purposes other than that of a marine oil terminal. MOTEMS requires a comprehensive above-water and underwater inspection and evaluation of the structural, mooring, geotechnical, mechanical, and electrical components and systems.

Under MOTEMS, the earthquake loads are described in terms of Peak Ground Acceleration, spectral acceleration, and earthquake magnitude (CBSC 2003). These values are modified for site amplification and near-fault directivity effects.

B.6 PORT OF LOS ANGELES AND PORT OF LONG BEACH

The seismic design procedures for both the Port of Los Angeles and the Port of Long Beach are similar in that each port bases its seismic design on a performance-based design approach (Port of Los Angeles 2004a, 2004b, 2005; Port of Long Beach 2003). Two performance levels of design earthquake are considered:

- Operating-Level Earthquake: An earthquake with a 50 percent probability of being exceeded in 50 years of exposure (72-year recurrence). Under this level of excitation, the structure should satisfy the serviceability criterion of continued functionality immediately after an earthquake without structural damage. Any repairs required should be essentially cosmetic.
- Contingency-Level Earthquake: An earthquake with a 10 percent probability of being exceeded in 50 years of exposure (475-year recurrence). Under this level of excitation, the structure should be capable of resisting the earthquake without collapse and with repairable damage. A temporary loss of operations may occur, but operations are restorable within an acceptable period.

The design philosophy is to provide a ductile substructure with carefully defined and detailed plastic hinges in piles to dissipate the earthquake energy. A displacement-based design approach is used to ensure that the structural system and its individual components have enough capacity to withstand the deformation imposed by the design earthquake.

A series of nonlinear push-over analyses on two-dimensional structure models with tri-linear soil springs is performed to obtain the structure displacement capacity and shear demand. In non-regular or complicated structures, a three-dimensional structure model should be used.

Push-over analysis is a technique by which a computer model of a structure is subjected to a lateral load. The intensity of the lateral load is slowly increased and the sequence of cracks, yielding, plastic hinge formations, and failure of various components are recorded. Push-over analysis can provide significant insight into the weak links in the seismic performance of a structure. A series of iterations are usually required in which observed deficiencies are corrected until the design satisfies a pre-established performance criterion.

The effects of the up- and down-slope of the ground surface, liquefaction potential, settlement, and soil-structure interaction are considered in the analyses.

The displacement demand is established based on the equal displacement method or the substitute structure method with results from nonlinear static analysis. For two-dimensional structural analysis, a magnification factor is applied to the displacement demand to simulate orthogonal seismic excitation effects.

Time history analyses of the soil structure interaction are performed to obtain the deep-seated pile strength and displacement demand and to justify the “no deformation” assumption for the wharf structure analyses.

Material properties and strain limits for both the Operating-Level Earthquake and the Contingency-Level Earthquake are established for all the components of the wharf structure.

B.7 REFERENCES

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Appendix C3 Dive Survey Report of Damage to Piers 1 and 2 (Sea Engineering, Inc., November 2006, 10 pages) .....

Appendix C4 Emergency Structural Assessment of Piers 1 and 2 and Various Structures at Kawaihae Harbor, Island of Hawaii (Miyasato Kuniyoshi Engineers, LLC, November 13, 2006, 22 pages) .....

Appendix C5 Geotechnical Report of Harbor Damage, 7 Borings, Liquefaction Evaluation (Geolabs, Inc., January 12, 2007, 38 pages) .....

Appendix C6 Diving and Side Scan Sonar Survey Report (Sea Engineering, Inc., September 2006) .....

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Table C1-1 Mean Estimated Soil Properties of Data in Table C1-2

Mean Estimated Soil Properties

Type (-)	No. of Samples (-)	Density (-)	Unit Weight (pcf)	Moisture Content (%)	Friction Angle (degrees)	Cohesion (psf)	Relative Density (%)	Young's Modulus (ksf)	Void Ratio (-)
GP-SP / Fill	17	Dense	129	18	40	0	0.7	533	0.5
GP-SP / Fill	3	Loose (N<20)	97	31	32	0	0.3	163	0.8
SM-GM	24	Dense	131	13	42	50	0.7	528	0.3
SM-GM	28	Loose (N<20)	102	24	33	50	0.3	234	0.6

Sources: HC 5313; HC 5219; HC 5218.

Table C1-2 Estimated Soil Properties from SPT Blow Counts from Selected Borings

Estimated Soil Properties from SPT Data From Selected Borings

Boring Number	Date	Elevation	Type	N	Density	N60	N'60	Total Unit Weight	Effective Friction Angle	Void Ratio	Relative Density	Young's Modulus
(-)	(yr)	(ft)	(-)	(-)	(-)	(-)	(-)	(pcf)	(degrees)	(-)	(%)	(ksf)
BA	1990	-16.4	SM-GM	2	Loose	1	2	70	28	0.8	0.1	167
BA	1990	-14.4	SM-GM	3	Loose	2	3	70	28	0.8	0.1	178
B-8	1999	-12.8	GP-SP	27	Dense	17	29	127	40	0.4	0.7	355
B-8	1999	-11.8	GP-SP	52	Dense	33	54	135	40	0.2	0.9	568
BA	1990	-11.5	SM-GM	9	Loose	6	10	107	33	0.6	0.3	230
B-18	1988	-9.84	SM-GM	33	Dense	19	32	128	41	0.4	0.7	365
BA	1990	-9.5	SM-GM	4	Loose	3	5	83	29	0.8	0.1	188
B-8	1999	-8.9	GP-SP	35	Dense	23	37	131	43	0.3	0.7	430
B-18	1988	-7.9	SM-GM	23	Dense	13	22	120	37	0.5	0.5	303
B-36	1989	-7.5	GP-SP	60	Dense	38	61	135	40	0.2	0.9	631
BA	1990	-6.6	SM-GM	5	Loose	4	6	89	30	0.7	0.2	198
B-35	1989	-5.9	GP-SP	56	Dense	35	56	135	40	0.2	0.9	593
B-18	1988	-5.2	SM-GM	10	Loose	6	10	107	33	0.6	0.3	230
B-28	1988	-4.9	SM-GM	9	Loose	6	10	115	34	0.6	0.4	230
B-8	1999	-3.9	GP-SP	24	Dense	17	24	122	38	0.7	0.6	355
B-36	1989	-3.6	GP-SP	74	Dense	48	62	135	40	0.2	0.9	769
B-28	1988	-3.0	SM-GM	22	Dense	14	23	121	38	0.5	0.6	313
BA	1990	-2.6	SM-GM	28	Dense	21	36	131	43	0.4	0.7	397
BB	1990	-2.6	GP-SP	75	Dense	47	67	135	40	0.5	0.9	756
B-35	1989	-2.0	GP-SP	23	Dense	15	26	125	38	0.6	0.6	317
B-18	1988	-1.0	SM-GM	1	Loose	1	1	70	28	0.8	0.1	167
B-28	1988	0.0	SM-GM	16	Loose	11	18	115	35	0.5	0.5	282
B-36	1989	0.7	GP-SP	25	Dense	18	26	124	39	0.6	0.6	368
BB	1990	0.7	GP-SP	30	Dense	20	33	129	41	0.6	0.7	393
B-8	1999	1.1	GP-SP	3	Loose	2	4	70	28	0.9	0.1	100
B-35	1989	1.6	GP-SP	13	Loose	9	15	112	34	0.8	0.4	201
BA	1990	2.6	SM-GM	24	Dense	19	32	128	41	0.4	0.7	376
B-28	1988	3.0	SM-GM	5	Loose	4	6	89	30	0.7	0.2	198
BB	1990	4.6	SM-GM	18	Loose	13	22	120	37	0.5	0.5	303
B-36	1989	5.6	GP-SP	24	Dense	18	25	122	38	0.7	0.6	368
B-35	1989	5.9	GP-SP	11	Loose	8	13	109	33	0.8	0.4	188
B-8	1999	6.2	GP-SP	33	Dense	26	32	128	41	0.6	0.7	468

Table C1-2 Estimated Soil Properties from SPT Blow Counts from Selected Borings (continued)

Estimated Soil Properties from SPT Data From Selected Borings Continued

Boring Number	Date	Elevation	Type	N	Density	N60	N'60	Total Unit Weight	Effective Friction Angle	Void Ratio	Relative Density	Young's Modulus
(-)	(yr)	(ft)	(-)	(-)	(-)	(-)	(-)	(pcf)	(degrees)	(-)	(%)	(ksf)
BA	1990	7.5	SM-GM	21	Dense	17	27	125	39	0.4	0.6	355
B-28	1988	8.2	SM-GM	5	Loose	4	6	89	30	0.7	0.2	198
BB	1990	9.5	SM-GM	16	Loose	12	20	117	36	0.5	0.5	292
B-36	1989	10.5	SM-GM	51	Dense	40	47	137	47	0.3	0.8	616
B-35	1989	11.2	SM-GM	13	Loose	10	15	112	34	0.6	0.4	272
BA	1990	12.5	SM-GM	35	Dense	29	39	132	44	0.3	0.7	491
BF	1990	12.5	SM-GM	15	Loose	9	16	113	35	0.5	0.4	261
B-28	1988	14.1	GP-SP	26	Dense	21	28	126	39	0.6	0.6	405
BB	1990	14.4	GP-SP	60	Dense	47	57	135	40	0.5	0.9	756
B-36	1989	15.4	SM-GM	82	Dense	66	71	135	40	0.2	0.9	909
BF	1990	15.4	SM-GM	15	Loose	10	17	115	35	0.5	0.5	272
B-35	1989	16.1	SM-GM	28	Dense	22	29	127	40	0.4	0.7	407
BA	1990	17.4	SM-GM	62	Dense	51	61	135	40	0.2	0.9	741
B-28	1988	19.0	GP-SP	23	Dense	19	25	122	38	0.7	0.6	380
BF	1990	19.4	SM-GM	31	Dense	22	37	131	43	0.3	0.7	407
B-36	1989	20.7	SM-GM	97	Dense	80	82	135	40	0.2	0.9	1065
B-35	1989	21.0	SM-GM	53	Dense	43	49	138	48	0.3	0.8	647
BA	1990	22.6	SM-GM	15	Loose	12	19	116	36	0.5	0.5	292
BE	1990	24.6	SM-GM	48	Dense	30	51	139	49	0.3	0.8	501
BF	1990	24.6	SM-GM	26	Dense	20	34	129	42	0.4	0.7	386
B-36	1989	25.6	SM-GM	86	Dense	71	73	135	40	0.2	0.9	961
B-35	1989	25.9	SM-GM	31	Dense	26	29	127	39	0.4	0.6	449
BE	1990	27.6	SM-GM	6	Loose	4	7	95	31	0.7	0.2	198
BF	1990	29.5	SM-GM	25	Dense	20	32	128	41	0.4	0.7	386
B-35	1989	31.2	SM-GM	50	Dense	41	44	135	46	0.3	0.8	627
BE	1990	31.5	SM-GM	5	Loose	4	6	89	30	0.7	0.2	198
BF	1990	34.4	SM-GM	19	Loose	15	25	122	38	0.4	0.6	324
B-35	1989	36.1	GP-SP	91	Dense	75	77	135	40	0.5	0.9	1145
BE	1990	36.4	SM-GM	5	Loose	4	7	95	31	0.7	0.2	198
BE	1990	43.6	SM-GM	12	Loose	10	16	113	35	0.5	0.4	272
BE	1990	47.6	SM-GM	8	Loose	7	11	115	34	0.6	0.4	240
BE	1990	53.5	SM-GM	6	Loose	5	8	102	32	0.7	0.3	209
BE	1990	57.4	SM-GM	8	Loose	7	11	115	34	0.6	0.4	240
BE	1990	63.6	SM-GM	24	Dense	20	29	127	40	0.4	0.7	386

Sources: HC 5313; HC 5219; HC 5218.

BORING LOG W.O. 89-1861

BORING NO. BA DRIVING WT. 140 lb. DATE OF DRILLING 11-14-89
SURFACE ELEV. -17± DROP 30 in. WATER LEVEL N.A.

DEPTH (FEET)	GRAPH SYMBOL	UNIFIED SOIL CLASSIFICATION	BLOWS/FT.	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	RELATIVE COMPACTION (%)	DESCRIPTION
0		SM	2	84	38		Sandy SILT- Dark grayish brown, soft, with coral fragments.
			3	No Recovery			
5			9	No Recovery			
			4	84	28		
10			5	89	34		Grading to mottled brown color from 10 feet.
15			28	85	40		Grading firm to medium stiff from 14 feet.
20			24	90	33		
25			21	81	44		
30			35	97	33		
							Grading with gravel from 32 feet.
35			62	No Recovery			
40			15	75	49		
45			50/1"	No Penetration			BASALT- Gray, hard, vesicular. Begin NX coring from 44 feet. 86% Recovery from 44 to 49 feet. RQD = 78%
50							100% Recovery from 49 to 54 feet. RQD = 80%
55							76% Recovery from 54 to 59 feet. RQD = 20%
							End boring at 59 feet.
60							

Figure C1-1 Geotechnical Boring BA

Sources: HC 5313; HC 5219: HC 5218; HC 1226; HC 916A.

BORING LOG

W.O. 89-1861

BORING NO. BB | DRIVING WT. 140 lb. | DATE OF DRILLING 11-16-89
SURFACE ELEV. 8.5+ | DROP 30 in. | WATER LEVEL @ 6 ft.

DEPTH (FEET)	GRAPH SYMBOL	UNIFIED SOIL CLASSI-FICATION	BLOWS/FT.	DRY DENSITY (PCF.)	MOISTURE CONTENT (%)	RELATIVE COMPAC-TION (%)	DESCRIPTION
0		SM					Silty SAND - Light brown, moist, medium dense to dense, with coral fragments.
			75	84	9		
5			15/6"	84	12		
		SM	50/1"				Sandy SILT - Dark brown, loose to medium dense. Grading with basalt cobbles and gravel from 5.5 to 9 feet.
10			18	71	47		
15			16	95	27		
		SM					Silty SAND - Gray, medium dense.
20			60	106	25		
							End boring at 21.5 feet.
25							
30							

Figure C1-2 Geotechnical Boring BB

Sources: HC 5313; HC 5219; HC 5218; HC 1226; HC 916A.

BORING LOG W.O. 89-1861

BORING NO. BE DRIVING WT. 140 lb. DATE OF DRILLING 11-28-89
SURFACE ELEV. -24+ DROP 30 in. WATER LEVEL N.A.

DEPTH (FEET)	GRAPH SYMBOL	UNIFIED SOIL CLASSI-FICATION	BLOWS/FT.	DRY DENSITY (PCF.)	MOISTURE CONTENT (%)	RELATIVE COMPAC-TION (%)	DESCRIPTION
0		SM					Sandy SILT- Mottled light brown, medium dense, with basalt gravel and coral fragments.
			48	72	26		
							Grading soft from 4 feet.
5			6	90	35		
10			5	No Recovery			
15			5	79	46		
20			12	73	49		
25			8	80	45		
30			6	87	39		
							Grading to light gray color from 32 feet.
35			8	94	35		
40			24	83	41		
							BASALT- Gray, medium hard, vesicular.
45							Begin NX coring from 42 feet. 80% Recovery from 42 to 47 feet. RQD = 36%
							100% Recovery from 47 to 52 feet. RQD = 78%
50							
							End boring at 52 feet.
55							
60							

Figure C1-3 Geotechnical Boring BE

Sources: HC 5313; HC 5219; HC 5218; HC 1226; HC 916A.

BORING LOG

W.O. 89-1861

BORING NO. BF DRIVING WT. 140 lb. DATE OF DRILLING 12-1-89
SURFACE ELEV. -10+ DROP 30 in. WATER LEVEL N.A.

DEPTH (FEET)	GRAPH SYMBOL	UNIFIED SOIL CLASSI-FICATION	BLOWS/FT.	DRY DENSITY (PCF.)	MOISTURE CONTENT (%)	RELATIVE COMPAC-TION (%)	DESCRIPTION
0		SM					Silty SAND - Light gray, medium dense, with coral fragments.
			15	86	36		
5			15	79	30		
10			31	95	28		
15			26	101	27		
20			25	78	31		
25			19	92	30		
							End boring at 25.5 feet.
30							

Figure C1-4 Geotechnical Boring BF

Sources: HC 5313; HC 5219; HC 5218; HC 1226; HC 916A.

BORING LOG W.O. 89-1879

BORING NO. B 35 DRIVING WT. 140 lb. DATE OF DRILLING 10-11-89
SURFACE ELEV. 8.5+ DROP 30 in. WATER LEVEL @ 7.7 ft

DEPTH (FEET)	GRAPH SYMBOL	UNIFIED SOIL CLASSIFICATION	BLOWS/FT.	DRY DENSITY (PCF.)	MOISTURE CONTENT (%)	RELATIVE COMPACTION (%)	DESCRIPTION
0		GW-GM	56	90	29		Sandy Coral GRAVEL Mottled tan, moist, medium dense, silty. (Fill)
5		GW	23	98	27		Sandy Coral GRAVEL Mottled tan, wet, loose.
10			13	Tip Recovery			Grading to light gray color from 10 feet.
15			11	73	36		
20		SM	13	88	36		Silty SAND - Mottled gray, loose.
25			28	93	32		Grading medium dense from 24 feet.
30			53	92	32		Plate A1

BORING LOG W.O. 89-1879

BORING NO. B35(Cont.) DRIVING WT. 140 lb. DATE OF DRILLING 10-11-89
SURFACE ELEV. 8.5+ DROP 30 in. WATER LEVEL @ 7.7 ft

DEPTH (FEET)	GRAPH SYMBOL	UNIFIED SOIL CLASSIFICATION	BLOWS/FT.	DRY DENSITY (PCF.)	MOISTURE CONTENT (%)	RELATIVE COMPACTION (%)	DESCRIPTION
30			31	100	27		
35			50	108	26		Grading dense from 39 feet.
40		GW	91	99	29		Sandy GRAVEL Mottled gray, dense.
45							BASALT- Gray, medium hard to hard. Begin NX coring from 49 feet. 94% Recovery from 49 to 54 feet. RQD = 56%
50							94% Recovery from 54 to 59 feet. RQD = 48%
55							
60							End boring at 59 feet. Plate A2

Figure C1-5 Geotechnical Boring B-35

Sources: HC 5313; HC 5219; HC 5218; HC 1226; HC 916A.

BORING LOG W.O. 89-1879

BORING NO. B36 DRIVING WT. 140 lb. DATE OF DRILLING 10-12-89
SURFACE ELEV. 9.0+ DROP 30 in. WATER LEVEL @ 7.5 ft.

DEPTH (FEET)	GRAPH SYMBOL	UNIFIED SOIL CLASSI-FICATION	BLOWS/FT.	DRY DENSITY (PCF.)	MOISTURE CONTENT (%)	RELATIVE COMPAC-TION (%)	DESCRIPTION
0		GW-GM	60/6"				Sandy Coral GRAVEL Mottled tan, moist, dense, silty. (Fill) Covered by 3 inches of asphaltic concrete.
5			74	101	18		
		GW					Sandy Coral GRAVEL Mottled tan, loose to medium dense.
10			25	85	27		
15			24	97	30		
		SM					Silty SAND - Dark brown, medium dense to dense.
20			51	103	27		
25			82	96	31		
30			97	95	29		Plate A3

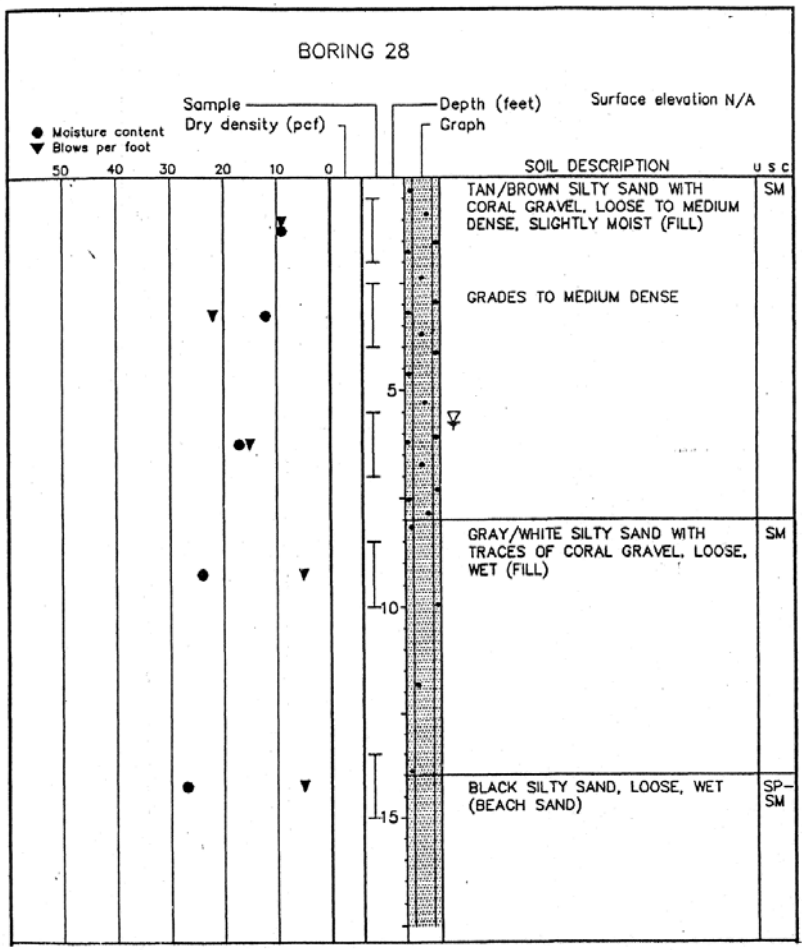
BORING LOG W.O. 89-1879

BORING NO. B36 (Cont.) DRIVING WT. 140 lb. DATE OF DRILLING 10-12-89
SURFACE ELEV. 9.0+ DROP 30 in. WATER LEVEL @ 7.5 ft.

DEPTH (FEET)	GRAPH SYMBOL	UNIFIED SOIL CLASSI-FICATION	BLOWS/FT.	DRY DENSITY (PCF.)	MOISTURE CONTENT (%)	RELATIVE COMPAC-TION (%)	DESCRIPTION
30			86	106	25		
35		GW					Sandy GRAVEL Grayish brown, dense to very dense.
40							BASALT- Gray, hard. Begin NX coring from 39 feet. 78% Recovery from 39 to 44 feet. RQD = 68%
45							66% Recovery from 44 to 49 feet. RQD = 34%
50							End boring at 49 feet.
55							
60							Plate A4

Figure C1-6 Geotechnical Boring B-36

Sources: HC 5313; HC 5219; HC 5218; HC 1226; HC 916A.



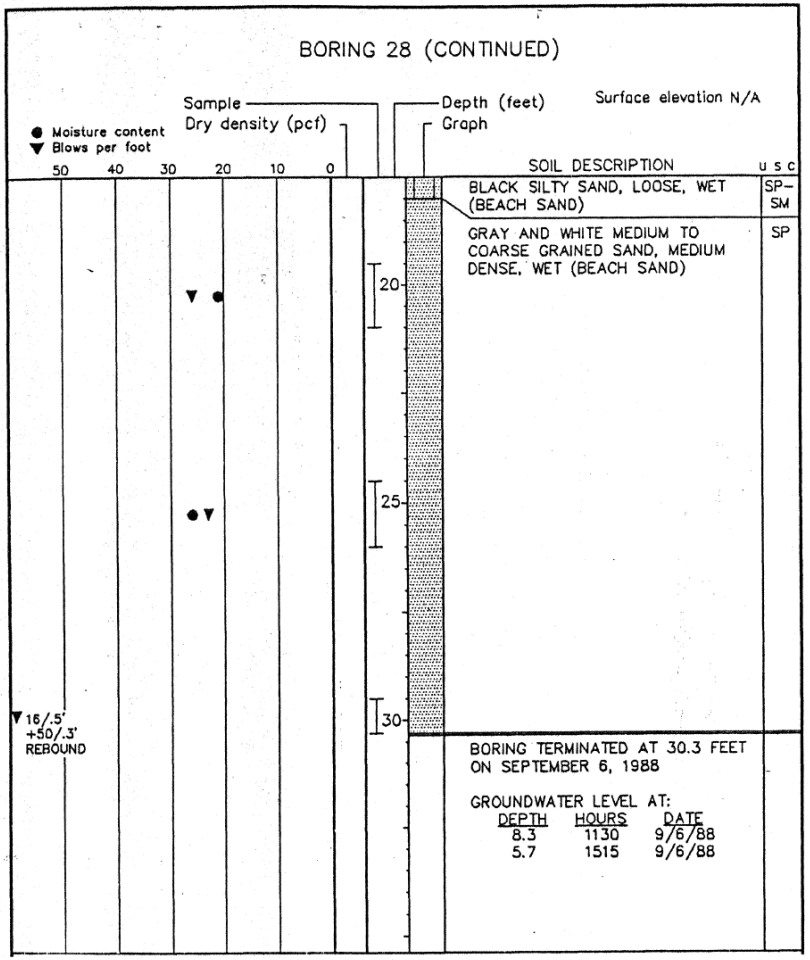


Figure C1-7 Geotechnical Boring B-28

Sources: HC 5313; HC 5219; HC 5218; HC 1226; HC 916A.

Date Started:	11/13/98	Drill Rig:	Mobile B-53
Date Completed:	11/13/98	Drilling Method:	H.S. Auger (8.0")
Logged By:	J. Chen	Driving Energy:	140 lb. wt., 30 in. drop
Total Depth:	21.5 feet		

Depth, ft	FIELD: Sample	FIELD: Penetra. Resist. Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength ksf	LABORATORY: Other Data	Pen, tsf	DESCRIPTION (Approximate Surface Elevation (ft): 14.4*)
		27						Tannish brown SANDY CORAL GRAVEL (GW), medium dense, damp
		52						Tannish brown CORAL GRAVELLY SAND (SP), dense, damp
5		35						
10		24						grades to medium dense
								Black SAND (SM) with traces of silt, very loose
15		3						
20		33						grades to medium dense
								Boring terminated at 21.5 feet
25								Groundwater level at: Depth 12 ft. Hours 1000 Date 11/13/98
30								

tk ILKHN.D/TSKJF

CW ASSOCIATES, INC. dba GEOLABS-HAWAII Geology Soils and Foundation Engineering	LOG OF BORING 8 PAVE ADDITIONAL BARGE TERMINAL AREA JOB H.C. 5313 KAWAIHAE, SOUTH KOHALA, HAWAII	PLATE A-8
WORK ORDER NO. 4086-00 KHN Dec 98		

Figure C1-8 Geotechnical Boring B-8

Sources: HC 5313; HC 5219; HC 5218; HC 1226; HC 916A.

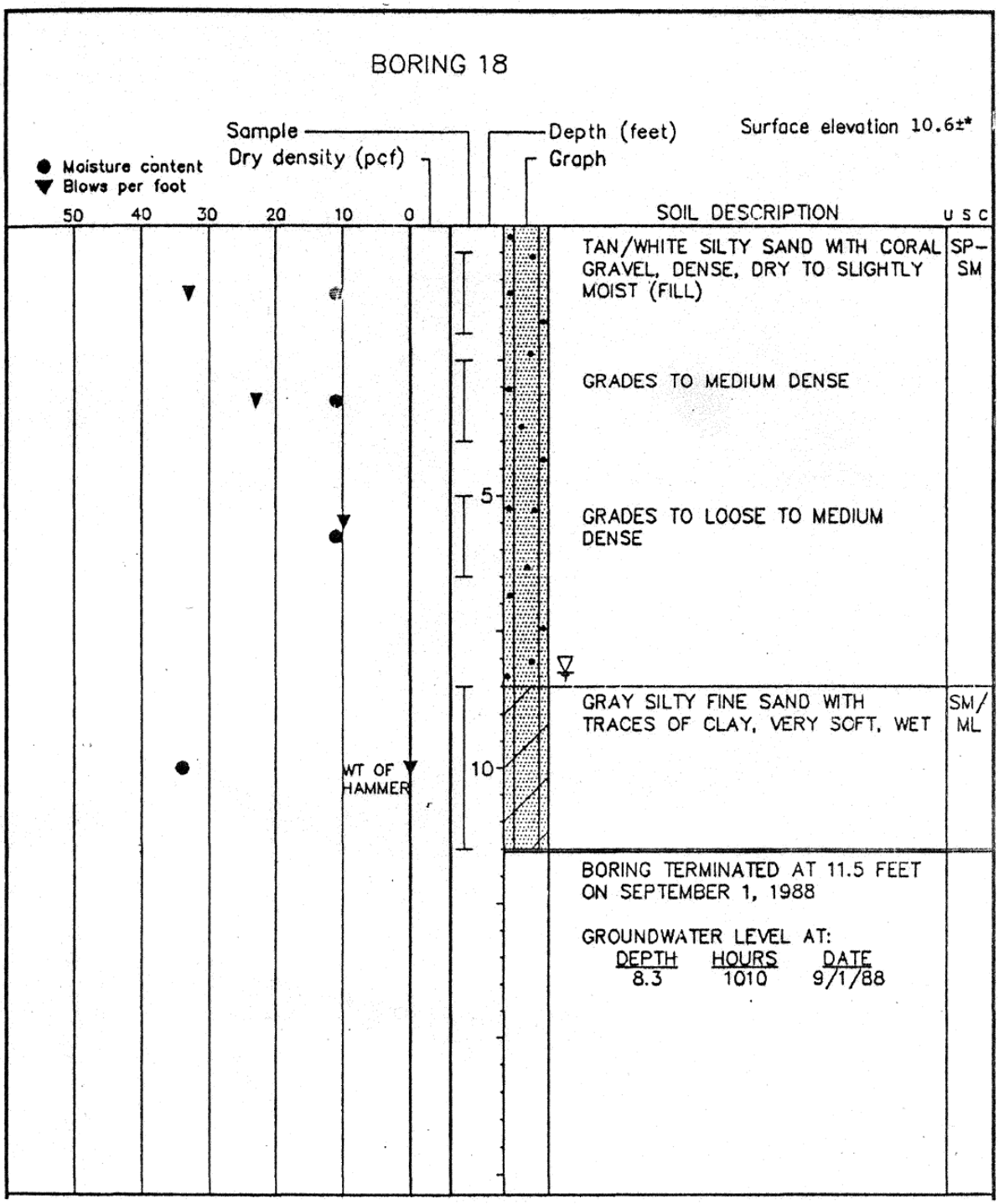


Figure C1-9 Geotechnical Boring B-18

Sources: HC 5313; HC 5219; HC 5218; HC 1226; HC 916A.

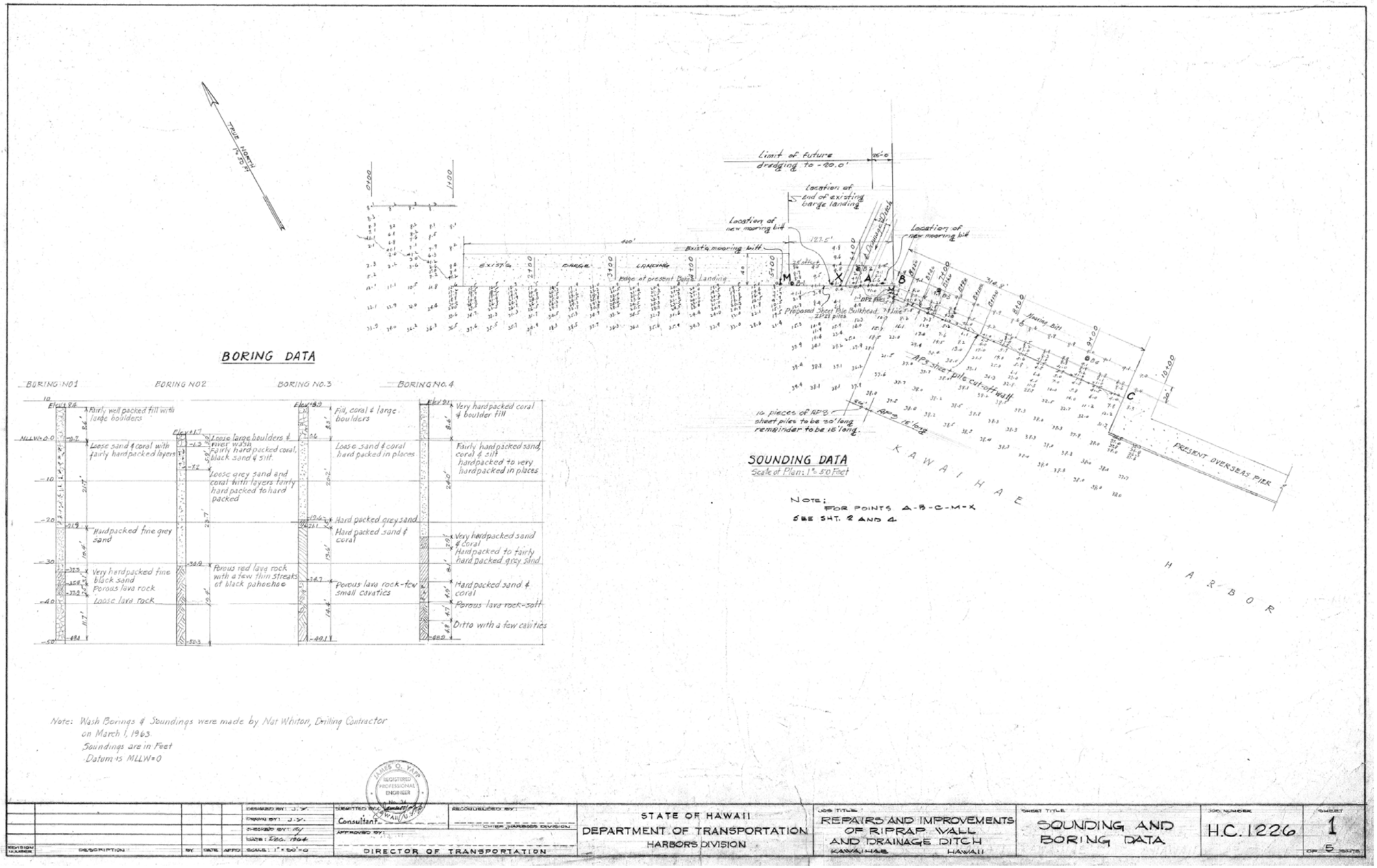


Figure C1-10 Geotechnical Borings (1)

Sources: HC 5313; HC 5219; HC 5218; HC 1226; HC 916A.

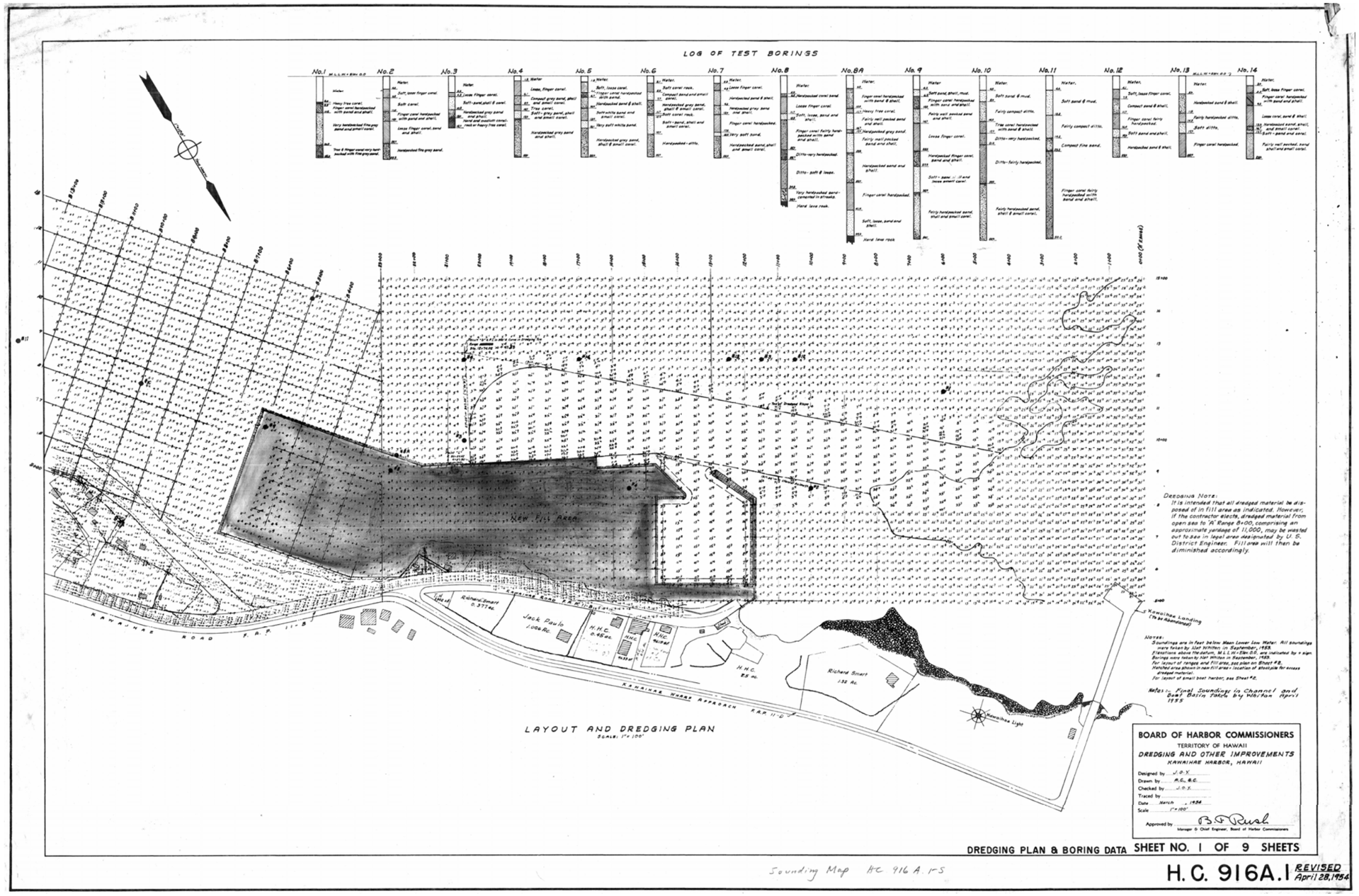


Figure C1-11 Geotechnical Borings (2)

Sources: HC 5313; HC 5219; HC 5218; HC 1226; HC 916A.

References

Drawings

Drawing No.	Project Number	Location	Description
1	HC 916A	Pier 1	Dredging and other improvements, Kawaihae Harbor, Hawaii (includes boring logs and Pier 1 construction drawings) (March 1954).
7	HC 1226	Pier 1 and revetment	Repair and improvements of riprap wall and drainage ditch (December 1964, including soundings in front of revetment and Pier 1 and borings).
10	HC 5218	Pier 2A/2B	Drawings for landside improvements at Kawaihae Harbor, Hawaii (1990). (Includes drawings for heavy load bridge over drainage channel.)
11	HC 5219	Pier 2B	Drawings for dredging and overseas pier extension at Kawaihae Harbor by Nishimura, Katayama, Oki & Santo, Inc., Okahara & Associates, Inc. (May 1990).
14	HC 5313	Behind Pier 2A	Pave additional barge terminal area, Kawaihae Harbor, Hawaii (October 1999).
16	HC 5218	Pier 2A/2B	Pile driving logs; landside improvements at Kawaihae Harbor (June 1993). (8½- x 11-inch drawings)
17	HC 5219	Pier 2B	Ernest K. Hirata & Associates, Inc., Test Pile Driving Report, Extended Overseas Pier, Kawaihae Harbor Improvements. Summary of Test Pile Logs, Pile Driving Logs (May 1991). (8½- x 11-inch drawings)
18	HC 5219	Pier 2B	Dredging and Overseas Pier Extension at Kawaihae Harbor, Hawaii, Pile Driving Logs, Volumes 1, 2, and 3 (1991) (8½- x 11-inch drawings)
19	HC 5219	Pier 2B	Nishimura, Katayama, Oki & Santo, Inc., and Okahara & Associates, Inc., Structural Calculations for Dredging and Overseas Pier Extension at Kawaihae Harbor (May 1990). (8½- x 11-inch drawings)

Appendix C2
Test Pile Driving Report for Pier 2
(Ernest K. Hirata & Associates, May 1991, 6 pages)

ERNEST K. HIRATA & ASSOCIATES, INC.

Soils and Foundation Engineering

Mailing Address: P.O. Box 1028, Aiea, Hawaii 96701-1028
99-1433 Koaha Place * Aiea, Hawaii 96701 * Phone (808) 486-0787 * FAX (808) 486-0870

ERNEST K. HIRATA P.E.
PAUL S. MORIMOTO P.E.
DAVID M. KITAMURA P.E.
JUNG K. KIM P.E.

May 14, 1991
W.O. 91-1861.1

Hawaiian Dredging & Construction Company
P.O. Box 4088
Honolulu, Hawaii 96812

Attention: Mr. Cedric Ota

Subject: Test Pile Driving Report
Extended Overseas Pier
Kawaihae Harbor Improvements
Kawaihae, Hawaii
Project No. H.C. 5219

Gentlemen:

Attached is our test pile driving summary for the above referenced project. A total of 58 test piles were driven by the Contractor from April 10th to May 3rd, 1991. All test piles were spudded prior to driving.

The piles were designed for a load capacity of 120 tons. All test piles were driven to depths ranging from 66.3 to 78.2 feet measured from the template, which corresponds to pile tip elevations of -54.4 and -66.3, respectively.

Based on the test pile driving, we recommend that a casting length for production piles be 78 feet in length.

Should you have any questions concerning this report, please feel free to call on us.

Very truly yours,

Ernest K. Hirata & Associates, Inc.

Jung K. Kim, P.E.

Enc: Summary of Test Pile Log
Pile Driving Logs (59 sheets)

TEST PILE DRIVING SUMMARY

PROJECT: DREDGING AND OVERSEAS PIER EXTENSION AT KAWAIHAE HARBOR W.O. 91-1861.1

PILE TYPE : 20″ OCTAGONAL PRESTRESSED CONCRETE PILE HAMMER TYPE : DELMAG DIESEL D30-23

COLUMN LINE	PILE NO.	PILE LENGTH (FEET)	DRIVEN LENGTH (FEET)	TEMPLET ELEV.	PILE TIP ELEV.	PILE CUT-OFF ELEVATION	PAYMENT LENGTH (FEET)	FINAL BLOWS/ LAST IN.	PILE CAPACITY (TONS)	REMARKS
E&F-45	40	90	74.5	14.5	-60	1.5	61.5	10	120	TEST PILE TP1
E&F-46	45	90	74.5	14.5	-60	1.5	61.5	10	120	TEST PILE TP2
E&F-47	50	90	68.7	11.9	-56.8	1.5	58.3	10	120	TEST PILE TP3
E&F-48	48	90	72.1	11.9	-60.2	1.5	61.7	10	120	TEST PILE TP4
E&F-49	49	90	70.6	11.9	-58.7	1.5	60.2	10	120	TEST PILE TP5
E&F-50	65	90	68.3	11.9	-56.4	1.5	57.9	10	120	TEST PILE TP6
E&F-51	70	90	69.5	11.9	-57.6	1.5	59.1	10	120	TEST PILE TP7
E&F-51&52	75	90	68.2	11.9	-56.3	1.5	57.8	10	120	TEST PILE TP8
E&F-53	81	90	67	11.9	-55.1	1.5	56.6	10	120	TEST PILE TP9
E&F-54	86	90	71.3	11.9	-59.4	1.5	60.9	10	120	TEST PILE TP10
E&F-57	91	90	71.8	11.9	-59.9	1.5	61.4	10	120	TEST PILE TP11
E&F-56	96	90	69.8	11.9	-57.9	1.5	59.4	10	120	TEST PILE TP12
E&F-57	101	90	69.3	11.9	-57.4	1.5	58.9	10	120	TEST PILE TP13
E&F-58	106	90	68.8	11.9	-56.9	1.5	58.4	10	120	TEST PILE TP14
E&F-59	111	90	66.3	11.9	-54.4	1.5	55.9	20	120	TEST PILE TP15
E&F-60&61	117	90	69.5	11.9	-57.6	1.5	59.1	10	120	TEST PILE TP16
E&F-61	122	90	71	11.9	-59.1	1.5	60.6	10	120	TEST PILE TP17

TEST PILE DRIVING SUMMARY

PROJECT: DREDGING AND OVERSEAS PIER EXTENSION AT KAWAIHAE HARBOR W.O. 91-1861.1

PILE TYPE : 20″ OCTAGONAL PRESTRESSED CONCRETE PILE HAMMER TYPE : DELMAG DIESEL D30-23

COLUMN LINE	PILE NO.	PILE LENGTH (FEET)	DRIVEN LENGTH (FEET)	TEMPLET ELEV.	PILE TIP ELEV.	PILE CUT-OFF ELEVATION	PAYMENT LENGTH (FEET)	FINAL BLOWS/ LAST IN.	PILE CAPACITY (TONS)	REMARKS
E&F-62	127	90	72.1	11.9	-60.2	1.5	61.7	12	120	TEST PILE TP18
E&F-63&64	133	90	70.7	11.9	-58.8	1.5	60.3	11	120	TEST PILE TP19
E&F-64	137	90	69.5	11.9	-57.6	1.5	59.1	15	120	TEST PILE TP20
E&F-65	42	90	70.6	11.9	-58.7	1.5	60.2	16	120	TEST PILE TP21
E&F-65&68	44	90	70.3	11.9	-58.4	1.5	59.9	15	120	TEST PILE TP22
D-44&45	262	90	73.5	12	-61.5	4.5	66	10	120	TEST PILE TP23
D-46	267	90	73.7	12	-61.7	4.5	66.2	15	120	TEST PILE TP24
D-47&48	274	90	74.7	12	-62.7	4.5	67.2	10	120	TEST PILE TP25
D-49&50	281	90	79	11.9	-67.1	4.5	71.6	10	120	TEST PILE TP26
D-50&51	286	90	69.8	11.9	-57.9	4.5	62.4	10	120	TEST PILE TP27
D-52&53	292	90	70.1	11.9	-58.2	4.5	62.7	10	120	TEST PILE TP28
D-53&54	299	90	77.5	11.9	-65.6	4.5	70.1	10	120	TEST PILE TP29
D-55&56	306	90	73.8	11.9	-61.9	4.5	66.4	10	120	TEST PILE TP30
D-57&58	313	90	73.7	11.9	-61.8	4.5	66.3	10	120	TEST PILE TP31
D-58&59	319	90	70.6	11.9	-58.7	4.5	63.2	10	120	TEST PILE TP32
D-60&61	324	90	69.9	11.9	-58	4.5	62.5	12	120	TEST PILE TP33
D-61&62	330	90	73.6	11.9	-61.7	4.5	66.2	12	120	TEST PILE TP34

TEST PILE DRIVING SUMMARY

PROJECT: DREDGING AND OVERSEAS PIER EXTENSION AT KAWAIHAE HARBOR W.O. 91-1861.1

PILE TYPE : 20″ OCTAGONAL PRESTRESSED CONCRETE PILE HAMMER TYPE : DELMAG DIESEL D30-23

COLUMN LINE	PILE NO.	PILE LENGTH (FEET)	DRIVEN LENGTH (FEET)	TEMPLET ELEV.	PILE TIP ELEV.	PILE CUT-OFF ELEVATION	PAYMENT LENGTH (FEET)	FINAL BLOWS/ LAST IN.	PILE CAPACITY (TONS)	REMARKS
D-63&64	475	90	74.2	11.9	-62.3	4.5	66.8	24	120	TEST PILE TP35
D-64&65	342	90	74.2	11.9	-62.3	4.5	66.8	12	120	TEST PILE TP36
D-65&68	348	90	72.9	11.9	-61	4.5	65.5	17	120	TEST PILE TP37
C-44&45	380	90	77.8	12	-65.8	3.5	69.3	10	120	TEST PILE TP38
C-45&46	385	90	76.8	12	-64.8	3.5	68.3	30/0.5″	120	TEST PILE TP39
C-46&47	391	90	77.3	12	-65.3	3.5	68.8	11	120	TEST PILE TP40
C-47&48	395	90	74.5	12.9	-61.6	3.5	65.1	10	120	TEST PILE TP41
C-48&49	399	90	75.9	13.5	-62.4	3.5	65.9	10	120	TEST PILE TP42
C-49&50	405	90	76	11.9	-64.1	3.5	67.6	10	120	TEST PILE TP43
C-50&51	410	90	73	11.9	-61.1	3.5	64.6	10	120	TEST PILE TP44
C-51&52	415	90	74.5	12.6	-61.9	3.5	65.4	10	120	TEST PILE TP45
B-52&53	420	90	73	11.9	-61.1	3.5	64.6	10	120	TEST PILE TP46
C-53&54	425	90	72.7	11.9	-60.8	3.5	64.3	10	120	TEST PILE TP47
C-45&46	430	90	75.3	11.9	-63.4	3.5	66.9	10	120	TEST PILE TP48
C-57&58	435	90	75.3	11.9	-63.4	3.5	66.9	10	120	TEST PILE TP49
C-56&57	440	90	73.3	11.9	-61.4	3.5	64.9	10	120	TEST PILE TP50
C-57&58	445	90	72.3	11.9	-60.4	3.5	63.9	10	120	TEST PILE TP51

TEST PILE DRIVING SUMMARY

PROJECT: DREDGING AND OVERSEAS PIER EXTENSION AT KAWAIHAE HARBOR W.O. 91-1861.1

PILE TYPE : 20″ OCTAGONAL PRESTRESSED CONCRETE PILE HAMMER TYPE : DELMAG DIESEL D30-23

COLUMN LINE	PILE NO.	PILE LENGTH (FEET)	DRIVEN LENGTH (FEET)	TEMPLET ELEV.	PILE TIP ELEV.	PILE CUT-OFF ELEVATION	PAYMENT LENGTH (FEET)	FINAL BLOWS/ LAST IN.	PILE CAPACITY (TONS)	REMARKS
C-58&59	450	90	73	11.9	-61.1	3.5	64.6	10	120	TEST PILE TP52
C-59&60	455	90	71.5	11.9	-59.6	3.5	63.1	12	120	TEST PILE TP53
C-60&61	460	90	71	11.9	-59.1	3.5	62.6	13	120	TEST PILE TP54
C-61&62	465	90	78.2	11.9	-66.3	3.5	69.8	12	120	TEST PILE TP55
C-62&63	470	90	76.1	11.9	-64.2	3.5	67.7	10	120	TEST PILE TP56
C-64&65	480	90	74.5	11.9	-62.6	3.5	66.1	10	120	TEST PILE TP57
C-65&68	485	90	75.4	11.9	-63.5	3.5	67	14	120	TEST PILE TP58
TOTAL		5220	4213				3687.7			
A+17'-51	EX-1	90	60.7	8.5	69.2	10.5			120	TEST LOAD

CONSTRUCTION SERVICES
29-1437 AIA PLACE
AIEA HAWAII 96701
TELEPHONE 486-0270
486-0737

PILE DRIVING LOG

JOB NO. 91-1061.1
Col. No.
Pile No. EX 1 - LOAD TEST

Project: KAWAIHAE HARBOR IMPROVEMENTS

Pile Type: 20" OCT. CONC
Hammer: DELMAG 30-32
Energy: 56,630 FT-LBS

SECTION	LENGTH	CAST DATE
I	90'	3-13-91
II		
III		

NOTES

DATE	START	STOP	SECT	SPLICE	CAUSE
5-10-91	11:10 AM	12:00			

DEPTH	BLOWS	DEPTH	BLOWS	DEPTH	BLOWS
			38		
			35		
			29		
			24		
5'		55'	22	105'	
			20		
			25		
			30		
			43		
10'		60"	24+50	110'	
			82/10"		
15'		65'		115'	
20'	7	70'		120'	
	6				
	7				
	10				
	13				
25'	13	75'		125'	
	15				
	17				
	21				
	20				
30'	22	80'		130'	
	24				
	17				
	16				
	14				
35'	11	85'		135'	
	16				
	13				
	14				
	18				
40'	16	90'		140'	
	18				
	20				
	22				
	22				
45'	20	95'		145'	
	22				
	21				
	19				

$S+C_2+C_3$ = set = 10/1"

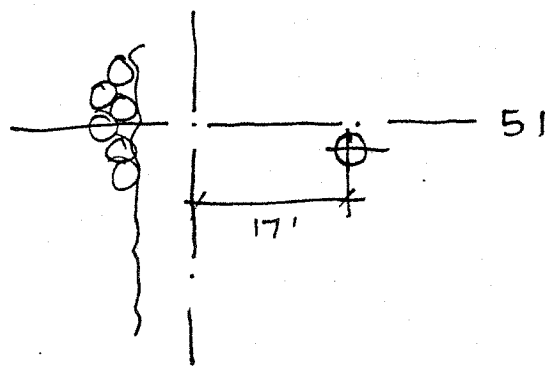
predrill 15 ft. (40' - SPUDDING DEPTH)

driven length 60.7 ft.

grd. elevation 8.5 ft.

SPUDDING DEPTH 40
TIDE 1.5 10:50 AM

BENT 51



Inspected by: BRAD

DEPTH	BLOWS
1"	
2"	
3"	
4"	
5"	
6"	9
7"	7
8"	6
9"	6
10"	7
11"	8
	7
1"	8
2"	7
3"	9
4"	9
5"	9
6"	10
7"	10
8"	10
9"	10
10"	
11"	
1"	
2"	
3"	
4"	
5"	
6"	
7"	
8"	
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10"	
11"	
1"	
2"	
3"	
4"	
5"	
6"	
7"	
8"	
9"	
10"	

Appendix C3
Dive Survey Report of Damage to Piers 1 and 2
(Sea Engineering, Inc., November 2006, 10 pages)

Kawaihae Harbor Earthquake Damage
Dive Survey Report

November, 2006

Prepared for:

Miyasato Kuniyoshi Engineers, LLC
99-205 Moanalua Road., Suite 205
Aiea, Hawaii 96701

Submitted by:

Sea Engineering, Inc.
Makai Research Pier
Waimanalo, Hawaii 96795

#06-59

KAWAIHAE HARBOR EARTHQUAKE DAMAGE
DIVE SURVEY REPORT

INTRODUCTION

Sea Engineering, Inc., conducted diving inspections of Pier 2 and Pier 1 substructures at Kawaihae Harbor on October 16 and 19, 2006. The diving inspection was also attempted on October 17, 2006, but the visibility in the water was so poor due to the rains from the previous night that the divers could see nothing, and the inspection was aborted. The purpose of the inspections was to identify any structural damage to the pier caused by an earthquake that occurred off the Kona coast of Hawaii on the morning of October 15, 2006. Bottom substrate conditions were also inspected in front of the east end of Pier 1 and in the area between Pier 1 and Pier 2.

All diving operations were performed by a 3-man dive team, consisting of a dive supervisor, a diver, and a tender/standby diver. A P.E. diver performed all underwater inspection work. The inspection team utilized SCUBA and conducted all diving operations from shore.

PIER 1 SURVEY

Pier 1 was surveyed on October 19, 2006. A diving inspection was also attempted on October 17, 2006 at Pier 1, but it was aborted because of poor water visibility. Pier 1 is constructed as a 475-ft long sheet pile bulkhead. The pier layout, survey stationing, and some damage locations are shown on Figure 1. Approximately 60 feet of the east end of the wharf is constructed with steel sheet pile, the remainder is constructed from concrete sheet pile. Two 200-ft survey tapes were laid at the mud line along the face of the pier for a total stationing of 400 feet. Station 0+00 was located at the east end of the wharf. Two separate dives were made, one to lay the tape and inspect for damage, and a second to record findings on video. The video record is submitted under separate cover. The upper one-half of the bulkhead face is obscured by heavy coral growth, but the lower half is relatively clean. Water conditions were poor, with 2 to 5 feet nominal visibility. Visibility decreased to about 1 foot or less near the mudline in some places.

Minor cracks observed in encrusting coral on the face of the sheet pile were the most common indicator of the earthquake event. On the steel sheet pile section, fresh rust-colored surfaces showed where sections of corrosion scale had flaked off the sheet pile surface. This was most common on the rounded interlock joints, and mostly occurred in small sections approximately 1-in by 6-in or less. Figure 2 is a photograph showing cracks in the encrusting coral. Figure 3 is another photograph of cracks in the encrusting coral.

Two small areas of concrete sheet pile damage were found. At station 0+94, the concrete sheet pile was cracked across in a "Z" step-wise fashion (vertical and horizontal) for a length of approximately 18 inches (Figure 4). Portions of the damaged sheet pile were obscured by

coral growth. The damage was midway in the water column, approximately 8.5 feet below the MLLW water surface. Crack separation was approximately 0.75 inches. A corner spall was also observed at this location, and broken pieces had fallen to the base of the bulkhead.

A corner spall was also seen at the mud line at approximately station 1+47 (Figure 5). The spall was about 4 feet in vertical height, and had flaked off the sheet pile.

Pier 1 had previously been recently inspected by SEI for the Super Ferry operations, allowing some comparisons to be made. Concrete sheet pile separation near the mudline was compared at three locations, and no measurable differences were found.

Gaps between concrete sheet piles and between the base of the piles and the bottom were noted in the previous survey. These areas were inspected for possible loss of fill material from behind the sheet pile. Particularly wide bottom gaps appeared at station 4+00, and geotextile bags were observed behind the sheet pile. By feel it appeared that the geotextile encapsulated large gravel-sized material. A particularly large mound of gravel is evident at a gap between sheet piles at about station 1+75. Much of the material appears to be fresh, judging from the lack of marine growth. No other areas showing apparent significant loss of fill were found.

Pier 1 Bottom Substrate Conditions

At the request of State Harbors personnel, the bottom substrate was examined at the west end of Pier1. This area had also previously been examined for the Super Ferry. A large pile of boulders exists in front of the culvert east of and adjacent to Pier 1. However, the substrate at the mudline of the Pier1 bulkhead is native bottom consisting of firm coralline limestone. The sheet piles are embedded in this material. The native bottom is irregular, with up to 2 feet of relief, and extends approximately 20 feet from the bulkhead before boulders are encountered. Boulders are both coralline limestone and basalt in composition.

Some changes appeared to have occurred to the bottom substrate in the area east of Pier 1 directly in front of the drainage culvert. Many of the rocks showed what appeared to be fresh surfaces, as if they had been turned over. Figures 6 and 7 show freshly exposed surfaces of coral and basalt. Some settling or slumping appears to have taken place. The approximate location of this area is shown in Figure 1.

BOTTOM SURVEY, PIER 1 TO PIER 2

The area between Pier 1 and Pier 2 was also surveyed at the request of State Harbors. The shoreline here consists of a sloping Cemented Rock Masonry (CRM) wall with a fronting concrete apron. A steel sheet pile bulkhead contains the concrete apron, rising approximately 10 feet in height to the water surface. The bathymetry seaward of the sheet pile is steep, at approximately 1.5V to 2H. A survey tape was laid on the CRM wall for positioning, with stationing beginning at the edge of Pier 2, and divers swam the length of the bulkhead. The survey extended seaward from the bulkhead to about the 20-foot depth, where water visibility

decreased substantially. Only one area (station 1+74) was found where the bottom was obviously affected by the earthquake event. A piece of native limestone shelf had apparently broken and collapsed. Figure 8 shows the freshly sheared escarpment. The area was approximately 8 feet in length, with a vertical relief of about 4 feet.

PIER 2 SURVEY

The Pier 2 substructure was inspected on October 16, 2006. Pier 2 is a concrete structure consisting of a deck slab supported by concrete piles and beams. At the inboard edge of the pier there is a concrete seawall and a lowered portion of deck supported by either one or two piles per bent. Inboard of the lowered deck section is a bulkhead. Bent numbering convention has Bent 0 at the north side (i.e. the Pier 1 side) of the structure, with bent numbering increasing to the south. Piles are labeled alphabetically, with Pile A being the outboard pile. There are two distinct sections, Bents 0 to 43 and Bents 44 to 66, which appear to have been built separately as each section is configured differently. The north section (Bents 0 to 43) appears to be older than the south section (Bents 44 to 66).

The inspection was conducted by making a surface swim of the pier and noting any defects above water. There is no visual indication of damage to the Pier 2 substructure as a result of the earthquake. Particular attention was paid to the interface of the piles and the beams, inspecting for separations, cracks or any other signs of movement or displacement. Following the surface swim, a dive was made to inspect the submerged portion of the piles and the bulkhead, looking for cracked piles, signs of movement at the bulkhead or evidence of loss of fill at the bulkhead.

In Bents 0 to 43, there were a large number of spalls and delaminations observed on the slab soffit, beams, and concrete piles above water. Additionally, many of the piles have concrete repair jackets, several of which are deteriorated. All of the observed damage appears to be corrosion based, as evidenced by the corroded reinforcing steel visible within the spalled areas. Numerous pieces of freshly fallen concrete were encountered along the mudline in Bents 0 to 43. These pieces of concrete may have fallen off due to movement from the earthquake, however, as all of the exposed steel was corroded, the concrete appears to have already been delaminated. There were no signs of movement or loss of fill at the bulkhead.

Two pipes, one used for offloading diesel and the other used for offloading gasoline, are suspended beneath Pier 2 in Bents 0 to 43. As a result of the earthquake both of the pipes suffered damaged to their hangers, and a significant length of each pipe was in the water. In some locations failure was apparently caused by corrosion of the hangers, and at other locations deck spalls at the hanger locations caused the hanger anchors to fail. At the time of the inspection, the owner of the pipes was performing inspections to assess damage to the pipes and hangers, and to formulate a plan for repair.

In Bents 44 to 66 only a few spalls were observed above water, typically located on the lower outboard face of the longitudinal beam in the D-pile row. At each of these locations corroded reinforcing steel was exposed within the spalled area indicating that the damage

was corrosion based and not a result of the earthquake. There were no visible defects to the piles or bulkhead below water.

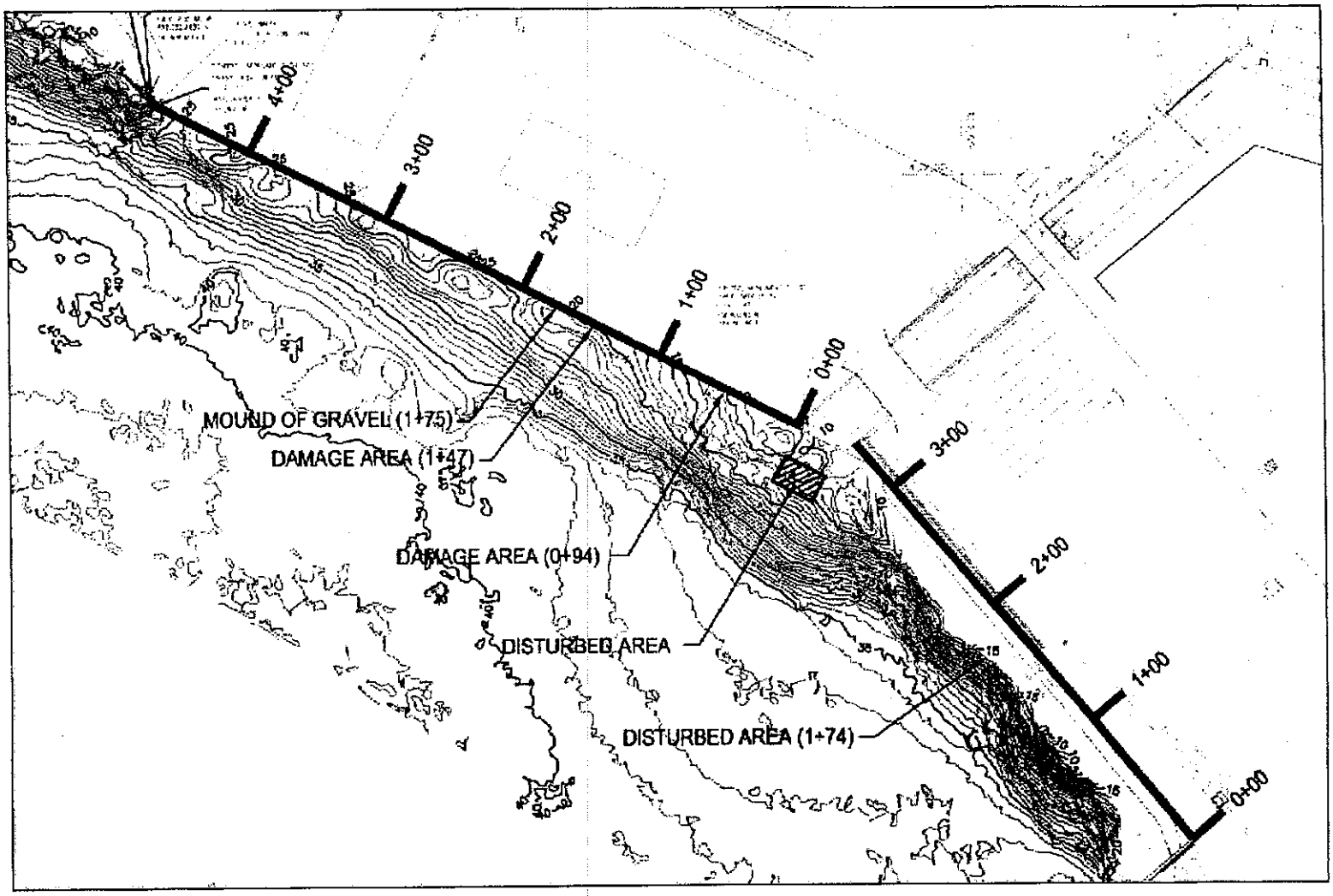

Figure 1. Stationing and areas of damage or disturbance, Pier 1

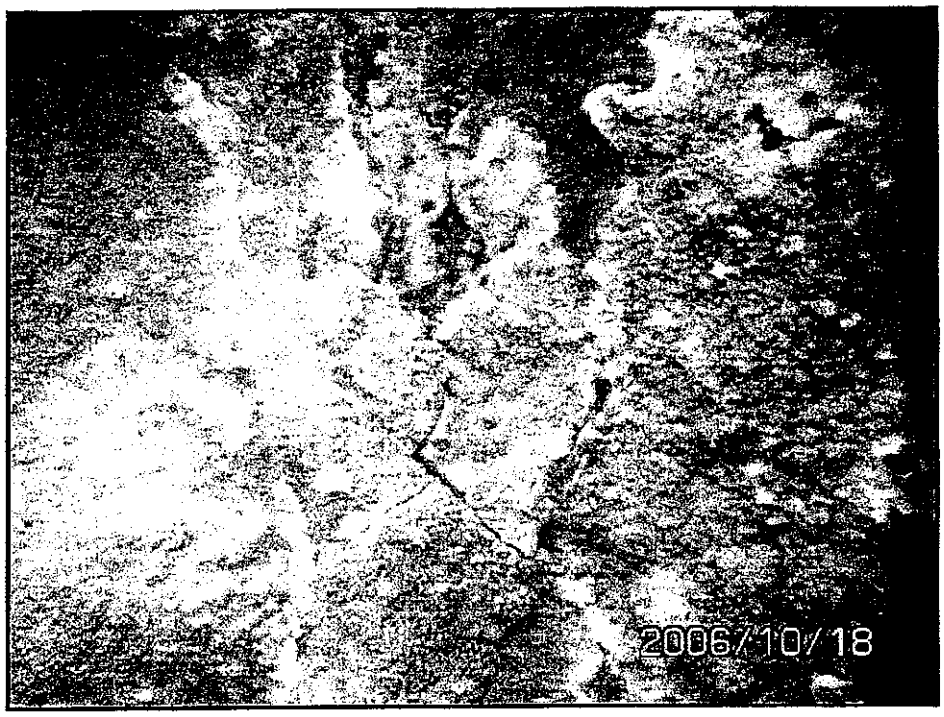

Figure 2. Typical conditions showing cracks in encrusting coral (Pier 1)

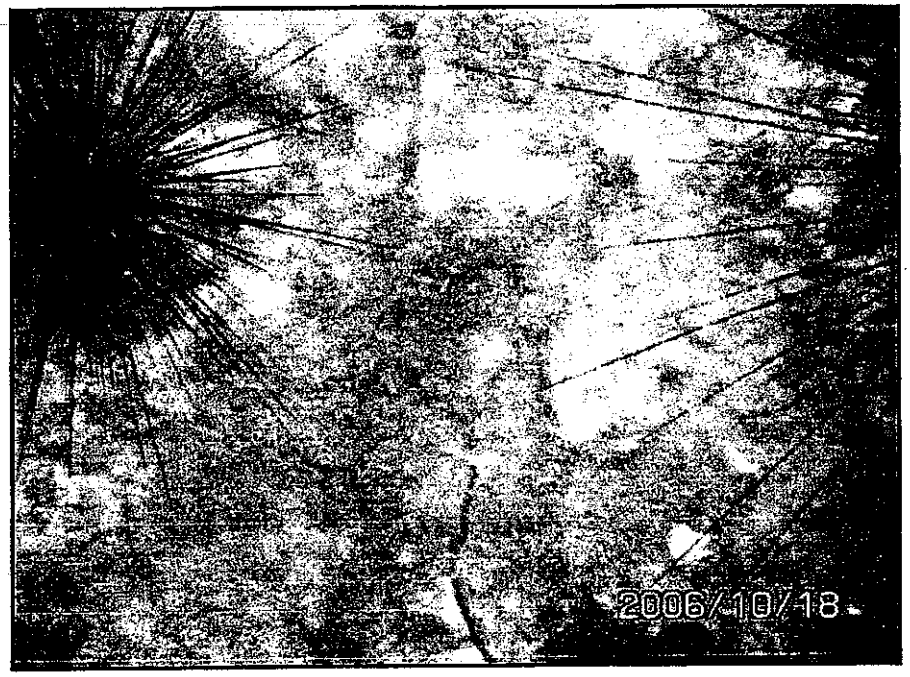

Figure 3. Cracks in encrusting coral (Pier 1)

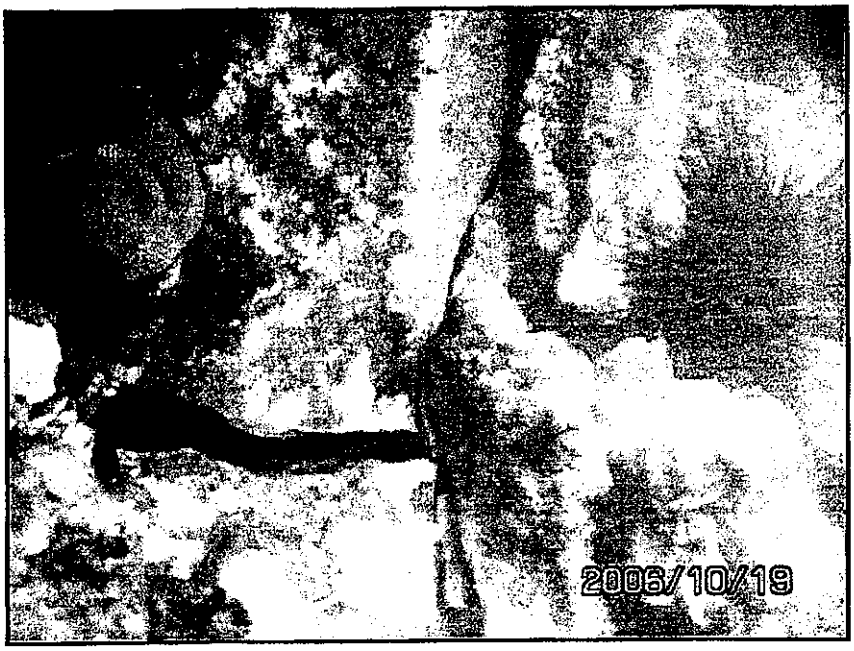

Figure 4. Station 0+94, damage to concrete sheet pile: crack and corner spall (Pier 1)

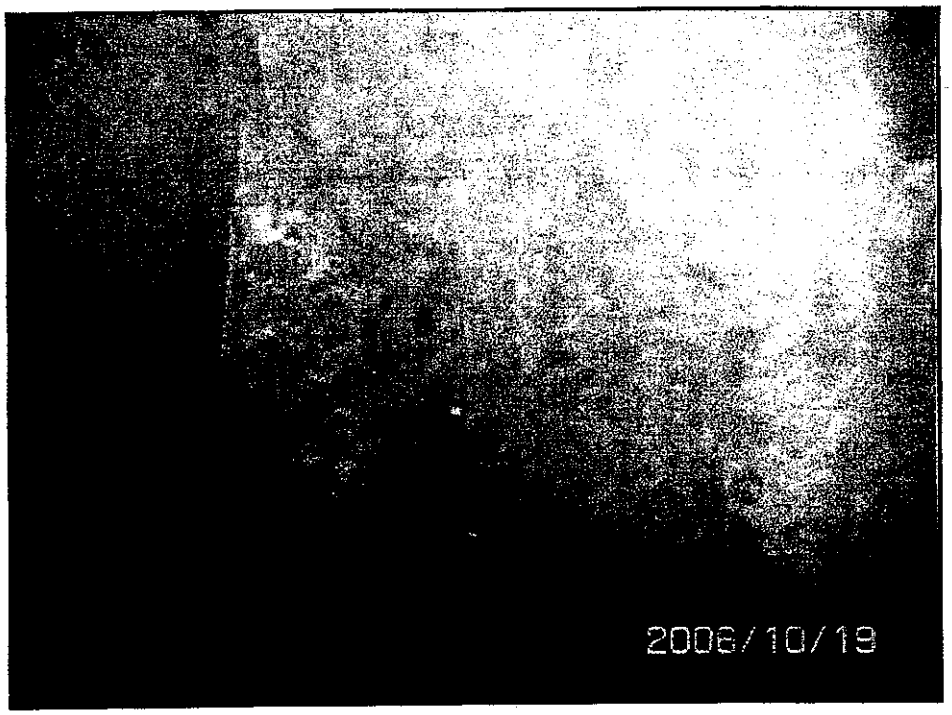

Figure 5. Station 1+47, corner spall showing freshly exposed surface (Pier 1)

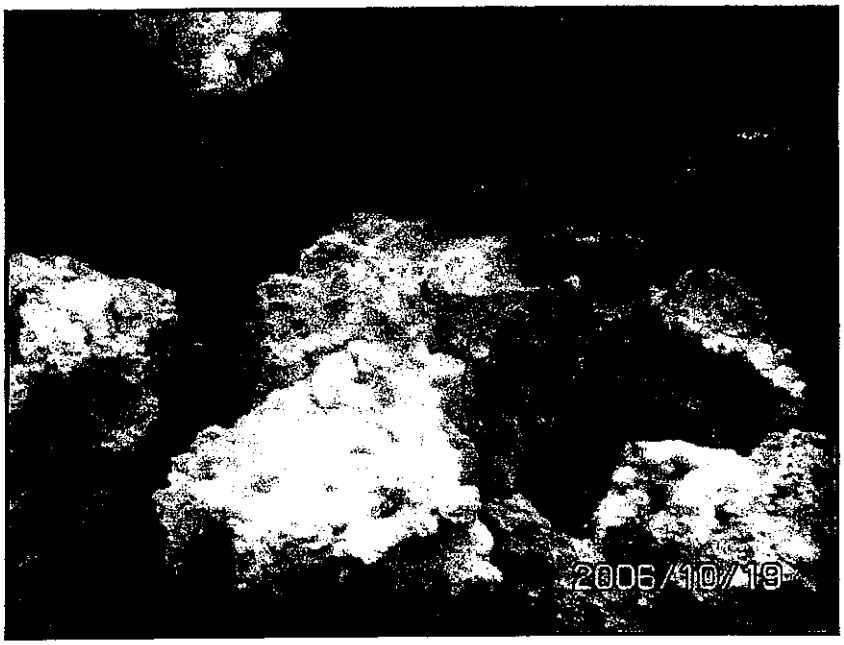

Figure 6. Freshly exposed surfaces of coralline limestone

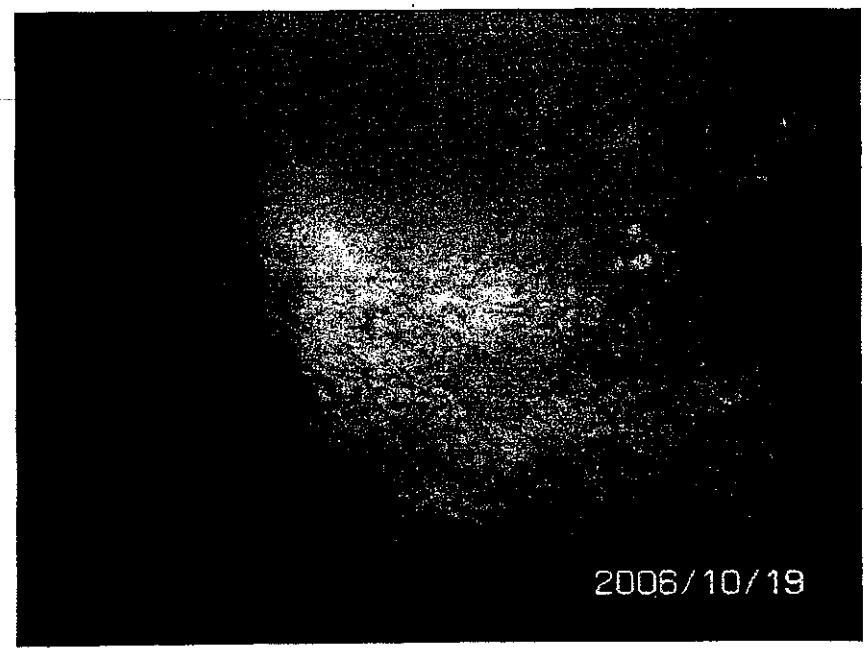

Figure 7. Freshly exposed surface, basalt boulder

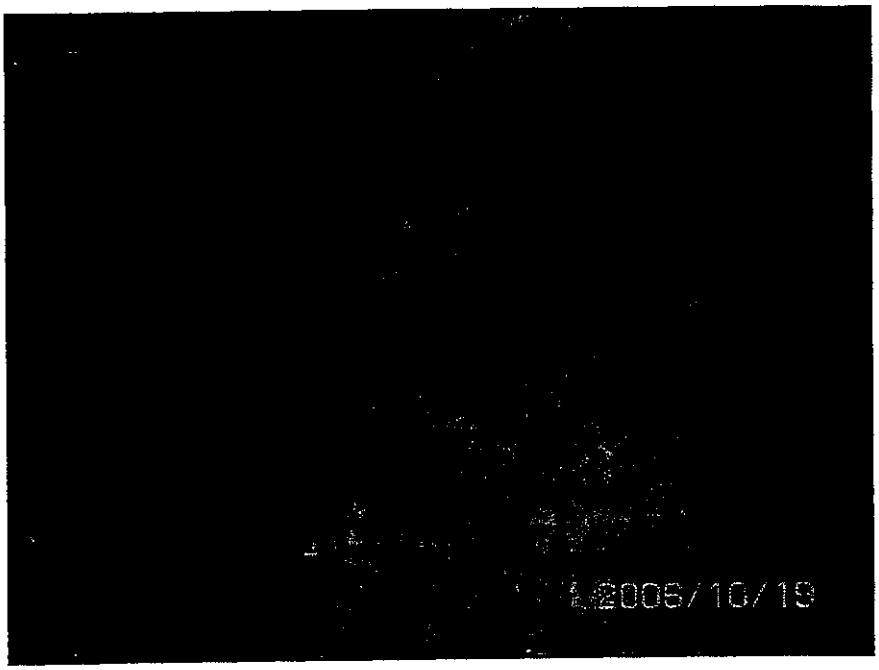

Figure 8. Shear zone of coralline limestone (native ground) between Pier 1 and Pier 2

Appendix C4
Emergency Structural Assessment of Piers 1 and 2 and Various Structures at Kawaihae Harbor, Island of Hawaii
(Miyasato Kuniyoshi Engineers, LLC, November 13, 2006, 22 pages)

Miyasato • Kuniyoshi
ENGINEERS LLC

Structural Engineering Services

Glenn H. Miyasato, Ph.D., P.E.
Susan Y. Kuniyoshi, P.E.
Brian D. Kung, P.E.

November 13, 2006

Mr. Carter Luke
State of Hawaii
Department of Transportation- Harbors Division
79 South Nimitz Highway
Honolulu, HI 96813

RE: **Emergency Structural Assessment of Piers 1 and 2 and Various Structures at Kawaihae Harbor, Island of Hawaii**

Dear Mr. Luke:

At your request, Miyasato Kuniyoshi Engineers LLC visited the Kawaihae Harbor site to assess various damages observed after the Puako earthquake that occurred on October 15, 2006. The following is a summary of our findings based on visual observations noted during our field visits on October 16th and 17th, 2006. A small boat was made available to assist us in observing conditions at the underside of Pier 2 and along the bulkhead at Pier 1. In addition, Pat Ross and Jeremy Pope of Sea Engineering, Inc. assisted us in performing the underwater condition assessment of the pier substructures and bulkhead. A follow up underwater condition assessment of Pier 1 was performed on October 19, 2006. The primary focus of this assessment was to determine whether damaged structures are safe for continued use, or if use should be restricted and/or prohibited.

GENERAL DESCRIPTION

Pier 1 consists of a wharf bulkhead, approximately 411 feet long, and is built of precast concrete sheet piles at the bulkhead anchored by a concrete dead-man located approximately 40 feet inboard from the bulkhead face. Two steel-framed warehouse-type structures are located along the pier. Some pavement repair work at the north and south ends of the Pier has recently been completed to accommodate the future use of the pier by the Super Ferry. Pavement repair work for the section between the repaired ends is pending.

Pier 2 consists a pile-supported, suspended concrete wharf deck section with a steel sheet pile bulkhead inboard of the concrete wharf deck. Pier 2 is comprised of two separate sections: Pier 2A and Pier 2B. Pier 2A, which was originally built around 1958, is built of a concrete wharf deck spanning between transverse concrete bent-beams supported by vertical concrete piles and a concrete cut-off wall and steel sheet pile bulkhead. It is approximately 600 feet long by 38 feet wide. Pier 2B is an extension of Pier 2A at the south end and was built in the early 1990s. It is comprised of a concrete wharf deck spanning between longitudinal concrete girders supported by vertical and battered concrete piles (in longitudinal and transverse directions) and a concrete cut-off wall and steel sheet pile bulkhead. The Pier 2B wharf includes a concrete

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dead-man system to provide lateral support for the concrete wharf structure. Pier 2B is approximately 550 feet long by 45 feet wide. A steel-framed warehouse structure (approximately 80 feet by 196 feet) is located along the wharf at Pier 2A. This structure is currently used for storage and office space by Young Brothers, Ltd.

Other structures that were assessed included a short, pile-supported concrete bridge that provides access from Pier 2 to Pier 1 and the Brewer Building, a one-story concrete framed building with concrete masonry unit (CMU) exterior infill walls.

OBSERVATIONS AND ASSESSMENTS

Pier 1 (Wharf)

A separation has opened up in the ground along the joint between the concrete pavement and A/C pavement (for the entire length of the pier), directly over the dead-man wall location (See Photo 1). The width of the separation varies from 2 inches at the south end to over 12 inches at the north end, but some of the wider separations probably resulted from the A/C pavement and soil crumbling into the resulting void. The average separation width was 5 inches and the depth of the void varied from 3 feet to 4 feet. At the bulkhead, a separation gap has occurred between the concrete pavement and the concrete sheet pile cap (See Photo 2). The gap was approximately ½ inch wide at the north section of the pier and 1 inch wide at the center section (where the pavement has not been repaired). There was no gap at the south section where the concrete pavement has recently been replaced. Some repairs to the tiebacks at the south section were performed when the concrete pavement in this area was replaced. No continuous parallel surface rupture, buckling, or significant crack along the concrete pavement outboard of the dead-man was noticed that would suggest severe passive soil failure fronting the dead-man.

At the southern end of the pier, beyond the concrete pavement, the bulkhead consists of cantilevered steel sheet piles with a concrete pile cap. This section of the pier, from the south end of the concrete pavement to where the bulkhead intersects the adjacent drainage canal, an approximately one-inch separation gap has occurred between the bulkhead pile cap and the adjacent pavement.

An underwater investigation of the sheet piles at the bulkhead was performed on October 17th, but due to poor visibility from heavy rains on the previous night, the condition of the bulkhead structure could not be confirmed. A subsequent underwater investigation of Pier 1 noted a crack, approximately ¾ inch in width, has occurred in one concrete sheet pile for the full width of the pile at mid-height. A couple of spalls, one at the ¾" inch crack and one near the mudline of another sheet pile were also observed. At the center of the pier, a "large talus cone of gravel" was observed at the base of the sheet piles. Some soil materials have seeped through gaps in the sheet pile bulkhead. A few slope measurements of the vertical face of the pile cap at the bulkhead indicate a slope of 1% to 2.5% from plumb, leaning towards the ocean.

Based on these observations, it is our opinion that the entire pier structural system, including the bulkhead wall, pavement, and dead-man, displaced outboard at least a few inches at the surface as evidenced by the observed separation/void along the entire dead-man wall. In addition, higher lateral pressures associated with earthquake loading could have caused the horizontal crack and spalls in the concrete sheet pile noted in the underwater investigation and may have resulted in yielding/fracture of the tiebacks.

Pier 1 (Shed 1)
This shed, nearest to the water, is framed with steel moment frames that are oriented perpendicular to the wharf. The inboard columns of the moment frames are probably supported on shallow spread footings embedded in soil while the outboard columns are supported on the wharf dead-man wall. As a result of the displacement of the dead-man wall, the bases of the outboard steel columns have been pulled away laterally with respect to the inboard columns, thus inducing loads on the structure beyond its original design intent. The exterior CMU wainscot wall has also cracked and separated at the outboard corners of the building due to this differential displacement (See Photo 3). Some wide cracks in the A/C pavement at the interior and exterior of the building have also occurred. Although no large buckling was observed in the building's primary steel framing, an initial slope measurement of one outboard column indicated a 2.5% slope from plumb, with the column leaning inboard. Therefore, it is believed that this building has experienced moderate damage associated with the displacement of the wharf structure.

Pier 1 (Shed 2)
This shed, located south-east of Shed 1, is similarly framed to Shed 1; however all of the foundations for this shed are located inboard of the wharf dead-man, and the damages noted for Shed 1 were not evident for Shed 2. Some minor cracks in the A/C pavement that do not appear to be a structural concern were observed at the perimeter of the building.

Bridge between Pier 1 and Pier 2 (See Photo 4)
A minor spall has occurred at the northeast approach slab curb, possibly due to some pounding between the superstructure and the abutments. Minor separations have occurred between the abutment wing walls and the surrounding pavement. No major signs of structural distress or excessive displacements were noted.

Pier 2A (Wharf) (See Photo 5)
Significant ground settlement has occurred in the A/C pavement behind the wharf bulkhead. The settlement appears to be fairly uniform along the entire length of Pier 2A and the A/C pavement is approximately 3 to 4 inches lower than the pile supported concrete wharf deck, thus creating a step at this joint (See Photo 6). A minor gap, up to one-inch, has occurred at this step at the portion of Pier 2A south of the existing adjacent shed. At the underside of the concrete wharf deck, extensive spalling to the slab soffit, concrete beams and struts, and pile caps were noted through-out the pier. These conditions appear to be the result of long-term corrosion of the embedded reinforcing steel. However, newer concrete cover sections were observed by the divers at the bottom of the harbor below the pier. Therefore, it appears that the earthquake induced displacements to the pier has caused loosening and detachment of the delaminated concrete, exposing additional reinforcing steel and creating more open spalls. These newly created open spalls directly expose the concrete wharf reinforcing to the environment and increase the potential for accelerated corrosion.

Some horizontal cracks were noted at the outboard side of the tops of the some of the piles (See Photo 7). The cracks were less than 1/8" wide and located at random piles, and are likely to be flexural cracks that were created by the lateral displacement of the wharf superstructure during the earthquake. These cracks do not appear to be a structural concern at this time, but are a long-term durability concern.

Some valves along water lines below the wharf deck are leaking water. The support brackets for the fuel line below the wharf deck have become detached from the wharf deck and a large section of fuel line is below water (See Photo 8). No diagonal support struts were installed to resist lateral loads on the various utility lines.

Pier 2A (Warehouse)
The steel framed warehouse along Pier 2A appears similar to Shed 1 at Pier 1. The rigid steel frames are oriented perpendicular to the wharf, with the outboard columns supported by the concrete wharf structure and the inboard columns supported by pile foundations. Although the bases of the columns may have experienced some differential lateral displacements similar to those of Shed 1 at Pier 1, the magnitudes of these displacements appear to be much less based on the smaller separation widths between the A/C pavement and the concrete wharf deck. Initial slope measurements of the columns at the outboard side indicated slopes from 0.1% to 0.8% from plumb, with the columns leaning inboard. At the north end of the warehouse, one threaded rod of the vertical x-brace at the exterior end frame pulled out of the turnbuckle, probably due to lack of adequate thread engagement (See Photo 9). No other significant damage was observed in the building's primary steel framing. The foundations at the north end frame also showed signs of some movement, since there is a small separation gap between the interior column foundations of the end frame and the surrounding A/C pavement. The exterior concrete wainscot wall has wide vertical cracks at the outboard corners of the building, but these cracks do not appear to be a structural concern at this time.

The framing of the office area consists of wood ceiling joists supported by wood studs on a continuous shallow concrete footing. At the office area, the settlement in the A/C pavement has rendered the entry door inoperable. Numerous cracks in the interior wall sheathing are found throughout, though many appear to be only cracks in the paint. Some glass windows are also broken.

Pier 2B (Wharf)
Some minor ground settlement has occurred in the A/C pavement behind the wharf bulkhead. The settlement appears to be somewhat uniform and is evidenced by some minor cracks in the pavement along most of the Pier length and water ponding in this area (See Photo 10). No signs of structural distress of the concrete wharf structure from earthquake loading were noticed. The underwater condition assessment did not uncover any signs of structural distress.

Brewer Building
The damages to the concrete framed building are concentrated at the north-east exterior wall of the building where a partial height CMU infill wall is located. A large portion of the CMU infill wall at the north corner has spalled off at the top, probably due to concentrated compressive loads at this location. At the top of each of the three interior columns, severe shear cracks have developed leading to significant loss of concrete section (See Photos 11 & 12). This type of failure to short, stubby columns resulting from partial height CMU infill walls has been well-documented in the past. No other significant damage was observed in the building's concrete framing.

General Soil Conditions
Several workers at the harbor site noticed water and sand had seeped up through some of the A/C pavement fissure cracks for a few minutes after the earthquake event. These observations,

in conjunction with the resulting ground settlements noted, suggest that some liquefaction of the subsurface soils has occurred. Currently, a geotechnical investigation of the area, including an evaluation of the liquefaction potential, has been undertaken and some soil borings have been taken at the site as of this date.

CONCLUSIONS AND RECOMMENDATIONS FOR IMMEDIATE ACTION

Based on our observations and follow-up discussions, we have the following conclusions and preliminary recommendations concerning the immediate use of various pier structures:

Pier 1 (Wharf)

Due to the outboard displacements and separations experienced by the entire pier structural system and resulting damage to bulkhead wall, tiebacks, dead-man, pavement and bollard reinforcing, the mooring/berthing capacity of the pier has been adversely affected. Therefore, Pier 1 should not be used to berth or moor large vessels until items #1 and #2 noted below are performed.

1. Infill void inboard of the concrete pavement with Controlled Low Slump Material or sand/cement grout mix and repave as required to provide temporary support for local traffic
2. Temporary critical off-loading operations of smaller vessels may be allowed under the following conditions:
    a. Tug boats are used to actively minimize mooring forces on the pier.
    b. A system/program is set up to monitor the lateral displacements of the pier bulkhead. If lateral displacements greater than 1 inch are observed, then off-loading operations should be halted immediately.
    c. Temporary mooring locations are installed inboard of the dead-man to provide redundancy to the bollards or cleats.
    d. To the greatest extent possible, berth the vessel at the repaired south section of the concrete pavement where no separation gap between the concrete pavement on bulkhead pile cap was noted.
    e. Do not moor/berth any vessels to the pier when wind gust speeds exceed 40 miles per hour

Pier 1 (Shed 1)

Due to the relatively large permanent displacements and associated damage experienced by the shed, its current structural integrity is in question. Therefore, the following actions are warranted.

1. Evacuate occupants from the building. Entry into the building should be restricted to temporary access only for moving personal items from the building.

Pier 1 (Shed 2)

As no significant permanent displacement or damage was observed, it does not appear the structural integrity of this shed has been adversely affected and continued use of the building is acceptable.

Bridge between Pier 1 and Pier 2

As no significant distress was observed, it does not appear the structural integrity of the bridge or support abutments and wingwalls has been adversely affected and continued use of the bridge is acceptable.

Pier 2A (Wharf)
The damages to the pile supported wharf structure from earthquake loading appear to be minimal and no significant change in structural strength or stiffness is anticipated from the pre-earthquake condition. While pre-earthquake operations may resume based on the assessment that there is no significant loss of structural strength or stiffness, the earthquake has caused more open spalled areas with exposed reinforcing at the underside of the wharf deck. Based on current observations, at a minimum, the area between Bents 13 and 14 should be barricaded to prevent vehicular traffic. Items #1 and #2 should be performed prior to resuming operations.

1. Repave depressed A/C pavement inboard of concrete wharf deck to eliminate the step at the joint. The pavement should include proof rolling to ensure proper compaction.
2. Repair damaged utility lines. Repair details should be stamped by a structural engineered licensed in the State of Hawaii.

Pier 2A (Warehouse)
As no significant permanent displacement or damage was observed, it does not appear the structural integrity of this shed has been adversely affected. Therefore, continued use of the building is acceptable subject to execution of items #1, #2, and #3.

1. Repave depressed A/C pavement inboard of concrete wharf deck to eliminate the step at the joint. The pavement should include proof rolling to ensure proper compaction.
2. Verify that bases of the outboard columns do not have permanent spreading deformation with respect to the interior columns. Horizontal measurements between the inboard and outboard columns can be taken at the base and top of each column at each frame to determine if the bases have spread apart significantly.
3. Reattach the threaded rod vertical x-bracing at the north end frame and ensure fully engaged threads at the turnbuckle. Ensure that x-bracing is taut.

Pier 2B (Wharf)
Due to the lack of observed distress likely resulting from the more recent design and construction of this pier, the structural integrity of the pier does not appear to have been adversely affected by the earthquake. Therefore, resumption of pre-earthquake operations at this pier is acceptable subject to the execution of item #1.

1. Repave A/C pavement inboard of concrete wharf deck to provide a smooth transition between the concrete deck and the A/C pavement for harbor operations. The pavement should include proof rolling to ensure proper compaction.

Brewer Building
Due to the damage experienced by the north-east exterior concrete columns and CMU infill walls, the structural integrity of this building has been adversely affected. Therefore, the following actions are warranted.

1. Evacuate occupants from the building. Entry into the interior space within 30 feet of the north-east exterior wall should be prohibited. Entry into the rest of the building should be restricted to temporary access only for moving personal items from the building.
2. Temporary shoring should be installed immediately on each side of each damaged column at the exterior north-east wall.

REPAIR RECOMMENDATIONS

The following are preliminary recommendations for permanent repair of various pier structures from earthquake related damage:

Pier 1 (Wharf)

1. Probe for voids below the concrete pavement that may have occurred as a result of consolidation of loose soils. Remove pavement, fill and compact all voids and install new concrete pavement.
2. Evaluate asphalt pavement near the edge of concrete pavement for significant voids. Repair additional discovered voids as necessary.
3. Excavate to expose and determine the extent of damage to tie backs and dead man below the north and center pavement sections where separation between the bulkhead pile cap and pavement has occurred. Repair or replace damaged tie backs and dead man as necessary.
4. Replace the failed bulkhead pile cap, bollard, and cleat reinforcing steel connection to the concrete pavement where separation between the bulkhead pile cap and pavement has occurred.
5. Repair cracks and spalls in concrete sheet pile.
6. Seal gaps in the concrete sheet piles to prevent further erosion of the retained soils.
7. Infill separation gap between concrete bulkhead pile cap and pavement at southern end of pier.
8. Evaluate remaining pier lateral displacement capacity verses future earthquakes. In reference to the Naval Facilities Engineering Service Technical Report TR-2069-SHR, "Design Criteria for Earthquake Hazard Mitigation of Navy Piers and Wharves", the performance criteria for wharves subjected to earthquake induced liquefied soils loads is that lateral displacements be limited to 3 inches or less for a moderate earthquake and 6 to 12 inches or less for a major earthquake. Since at least 3 inches of lateral displacement at the top of bulkhead sheet piles has already occurred, the lateral load resisting system may be considered to have experienced a significant portion of its life cycle loading and displacement and its future earthquake displacement capacity is compromised. Restoration of the lateral load resisting system displacement capacity may require installation of additional bulkhead sheet piles and/or tie backs and may not be feasible or practical.
9. Perform geotechnical investigation to verify the global bulkhead soil stability and to determine liquefaction potential of supporting soils. Retrofit the pier structure as necessary to address concerns.

The above recommendations point to the relatively large effort and inherent difficulty associated with performing repairs to damage and permanent displacements to all major elements of an existing structure. In addition, this structure has demonstrated poor performance during an earthquake. Therefore, replacement of the structure should be considered as a prudent option. Replacement costs could be partially offset by savings on mass excavation necessary to expose the existing bulkhead system and higher costs associated with specialized repair. If a new bulkhead system is installed, the excavation could be limited to trench excavation for the tie backs and dead man in lieu of mass excavation.

Pier 1 (Shed 1)

1. Perform comprehensive structural investigation and analysis to determine condition of primary structural frame elements after undergoing large deformations observed. Repair and/or strengthen frame elements as determined from the structural investigation.
2. Reattach detached wall girts at the north and south exterior walls
3. Reconstruct outboard CMU wainscot wall
4. Restore structural frames such that columns are plumb. Restoration of the structural frames to a plumb condition may require modifications to the pier dead man foundation, which has permanently shifted outboard. Should a new foundation for the columns of the building frames be required, the foundation should be isolated from the pier dead man to prevent the same thing from reoccurring.
5. Infill cracks in the A/C pavement to prevent tripping hazard

Pier 1 (Shed 2)

1. Infill cracks in pavement around perimeter of building.

Bridge between Pier 1 and Pier 2

1. Infill small separation gap between wingwalls and adjacent pavement.

Pier 2A (Wharf)

1. Repair the flexural cracks at the tops of the piles, by epoxy injection or other methods, to protect the pile from deterioration. A more comprehensive survey of the substructure damages should be performed to identify similar type earthquake induced cracks that may not have been observed during the initial site visit.
2. Repair remaining spalling at the wharf substructure, particularly at open spalls that have lost delaminated cover due to earthquake movement. As a priority to allow reopening of the area between Bents 13 and 14 to normal operations, repairs to this area may be performed first.
3. Due to the presence of flexural cracks in the piles and lack of tie backs or battered piles, further study of the lateral load capacity of the wharf may be warranted.

Pier 2A (Warehouse)

1. Repair cracks in the concrete wainscot wall at the exterior corners.
2. Repair office area damages.

Brewer Building

1. Repair the damaged concrete columns and CMU infill walls. As part of the repairs, further investigation of the interaction between the infill walls and columns should be undertaken so that more effective repairs or retrofit of the affected elements and the building as a whole can be performed to mitigate future earthquake damage concerns.

LIMITATIONS

The opinions stated in this report are based on limited visual observations of the readily accessible and observable portions of the various structures and a review of the record drawings made available to Miyasato Kuniyoshi Engineers LLC. It is assumed that the record drawings accurately reflect the as-built conditions of the harbor structures. The observations were conducted during a brief walk-through. No physical testing or hazardous material testing was performed, and no calculations were performed to determine the adequacy of the structural systems or their compliance with any current code requirements.

Discovery of additional earthquake damage or design of permanent structural repairs or retrofits may require further structural investigation.

If you should have any questions, please feel free to call.

Sincerely,
Miyasato Kuniyoshi Engineers, LLC

Brian Kung

Brian Kung, P.E.
Its Member

Encl.- 1. Figures
2. Dive Survey Report dated October 2006 by Sea Engineering, Inc.

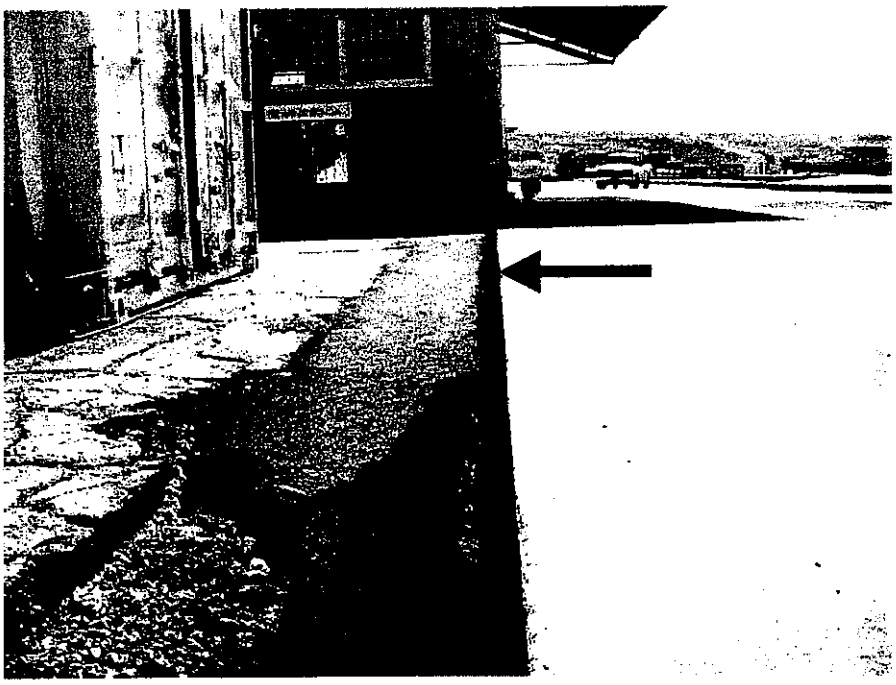


Photo 1- Separation gap along Dead-Man wall at Pier 1

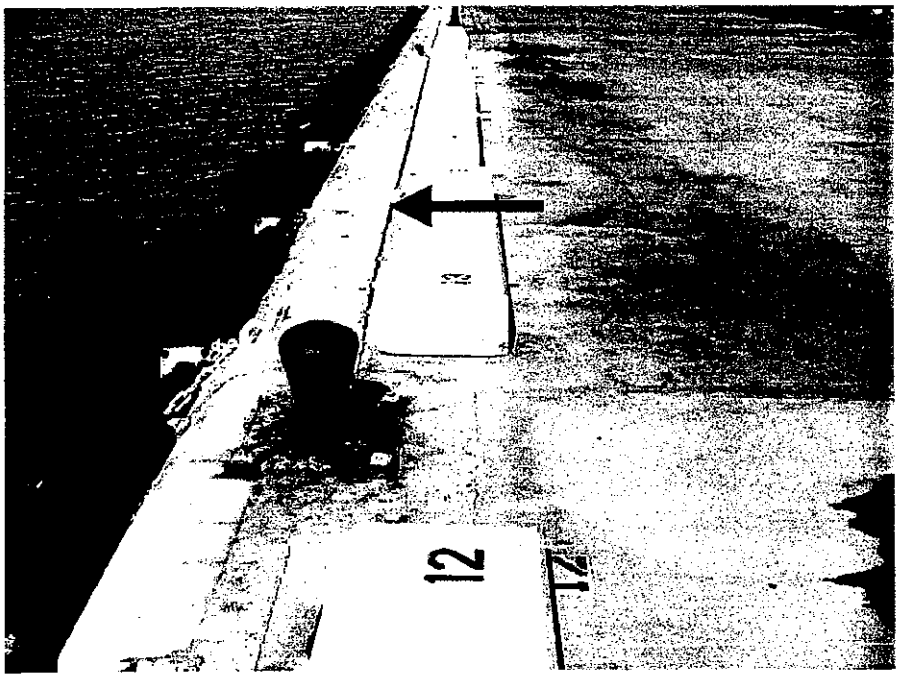


Photo 2- Separation gap at Bulkhead Pilecap/Pavement joint at Pier 1

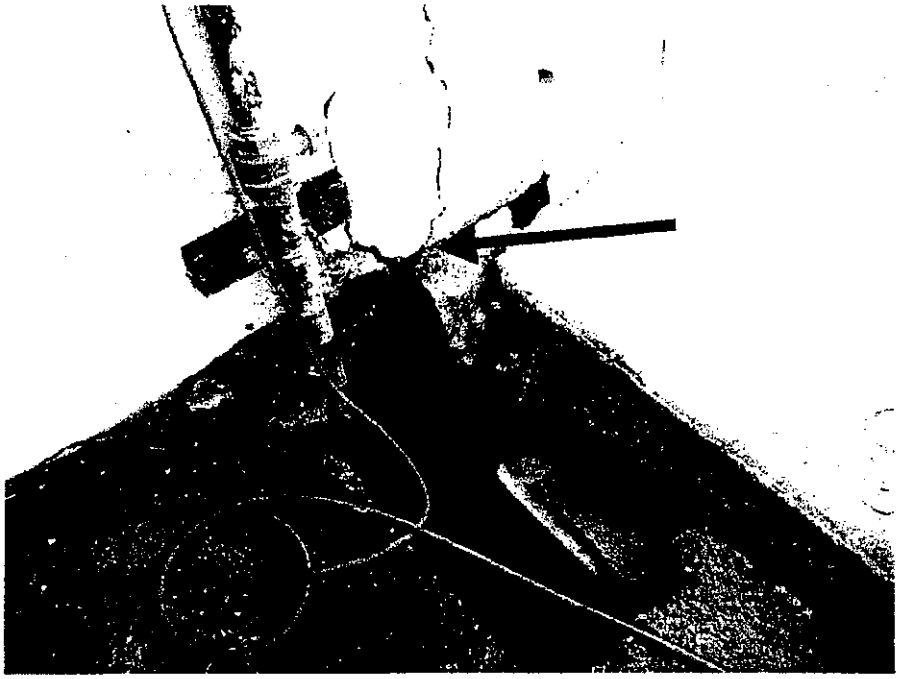


Photo 3- Crack in exterior CMU wall and foundation of Shed 1 at Pier 1

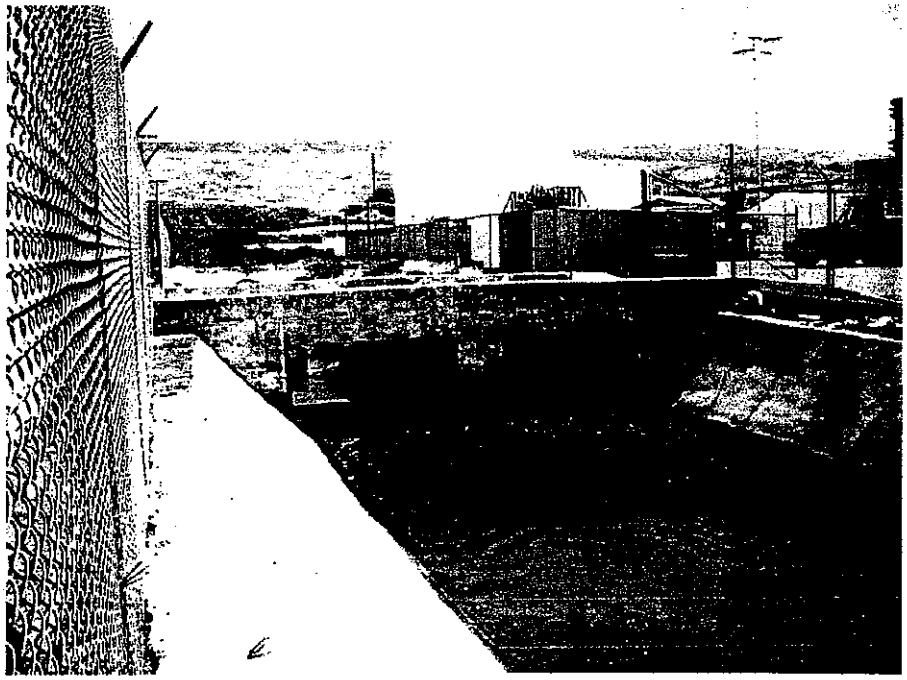


Photo 4- Bridge between Pier 1 & Pier 2

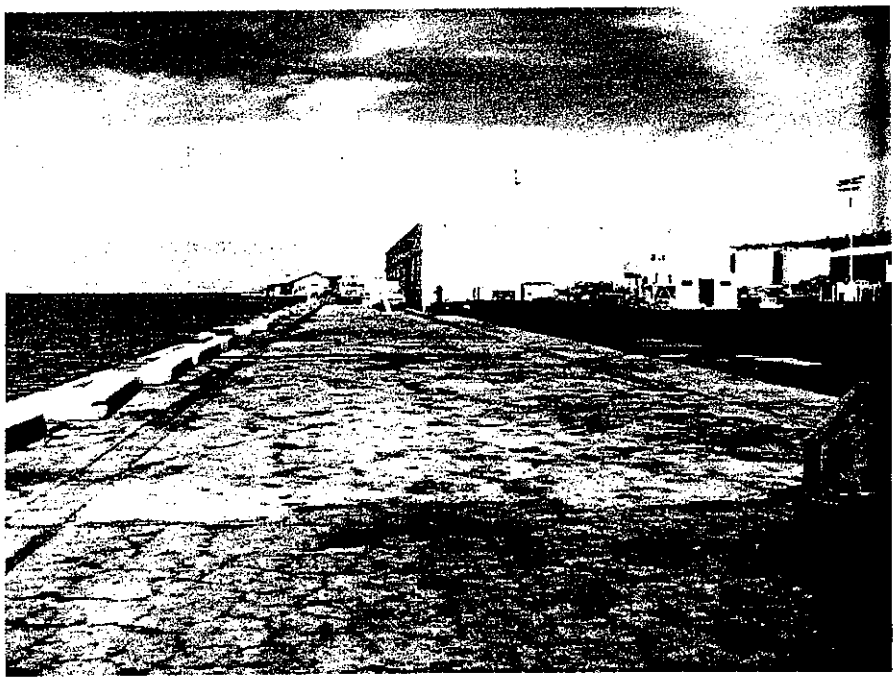


Photo 5- Pier 2A and Young Brothers, Ltd Warehouse

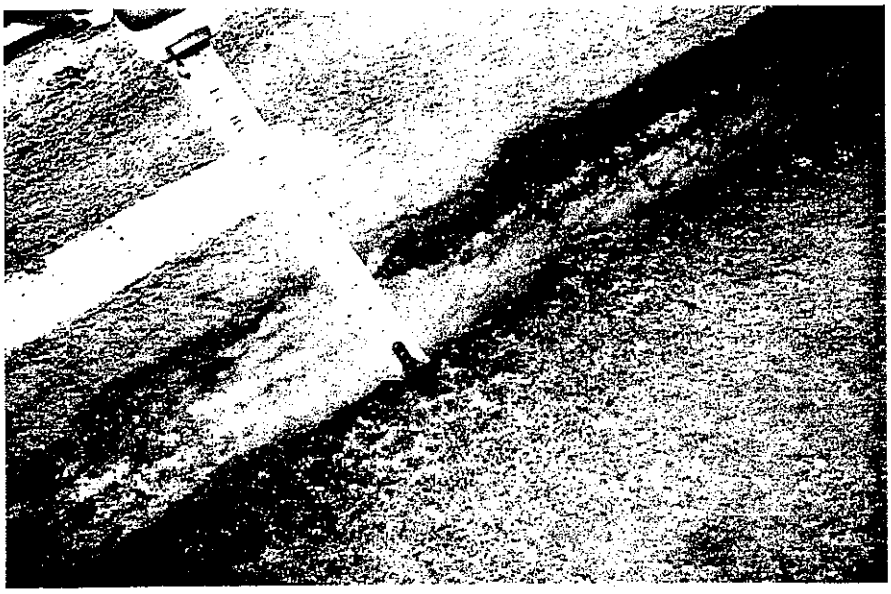


Photo 6- Ground Settlement at Pier 2A Bulkhead

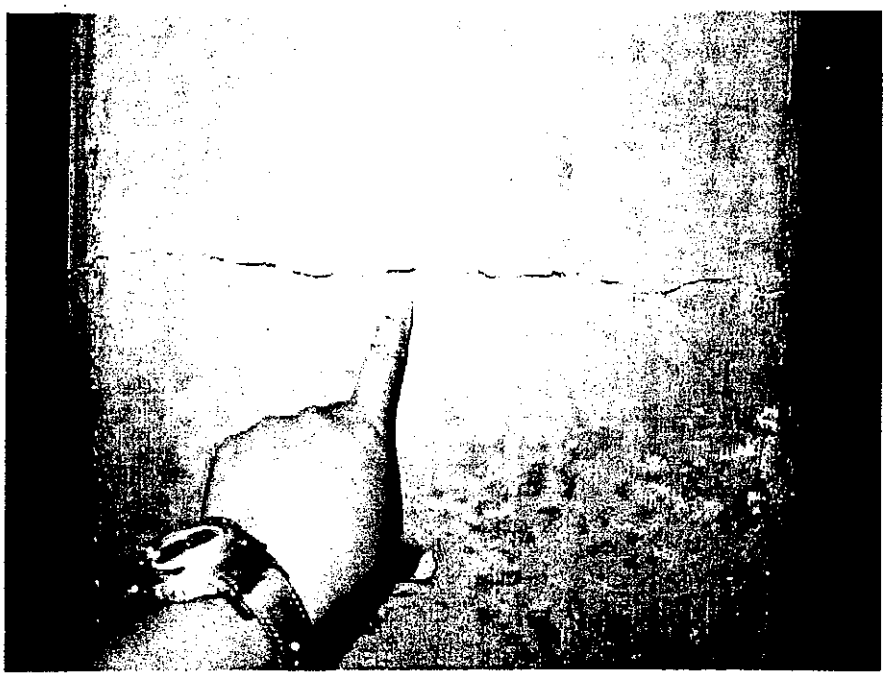


Photo 7- Flexural Crack at Top of Pile at Pier 2A

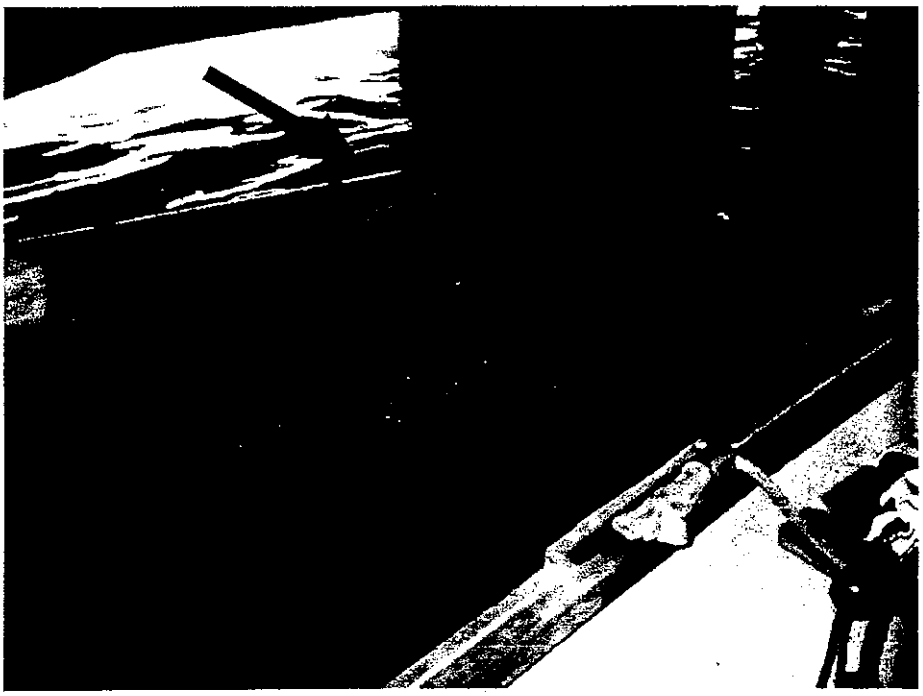


Photo 8- Collapsed Fuel Line below Pier 2A

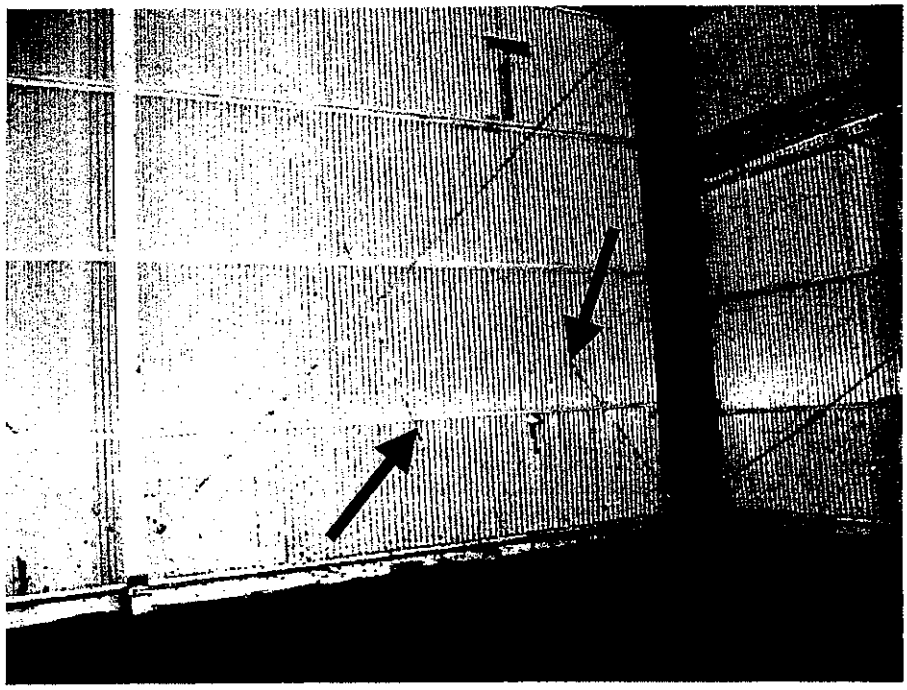


Photo 9- Disengaged X-Bracing at Young Brothers, Ltd. Warehouse

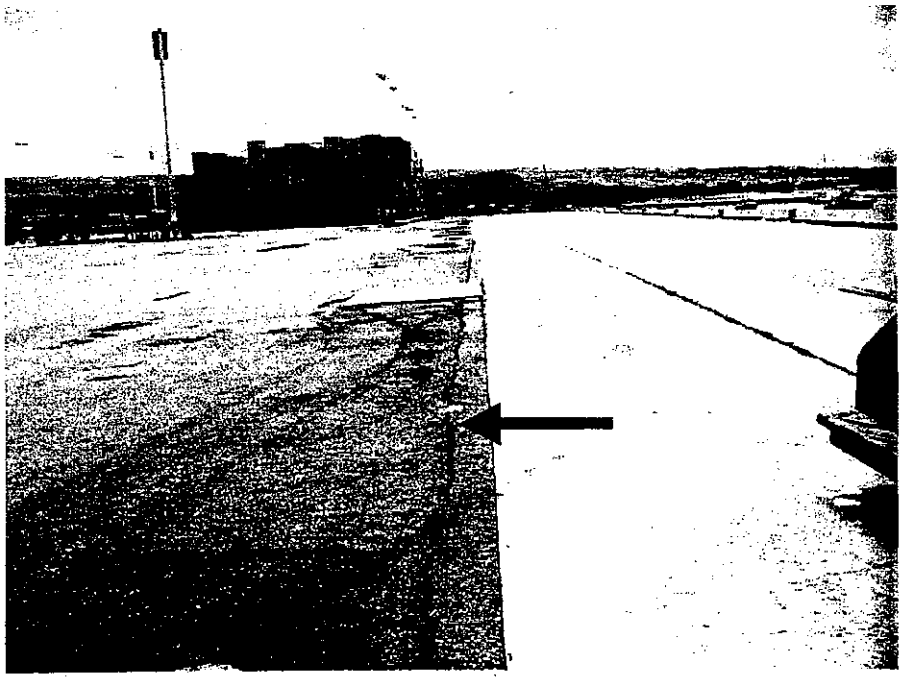


Photo 10- Pavement Settlement Crack along the bulkhead of Pier 2B

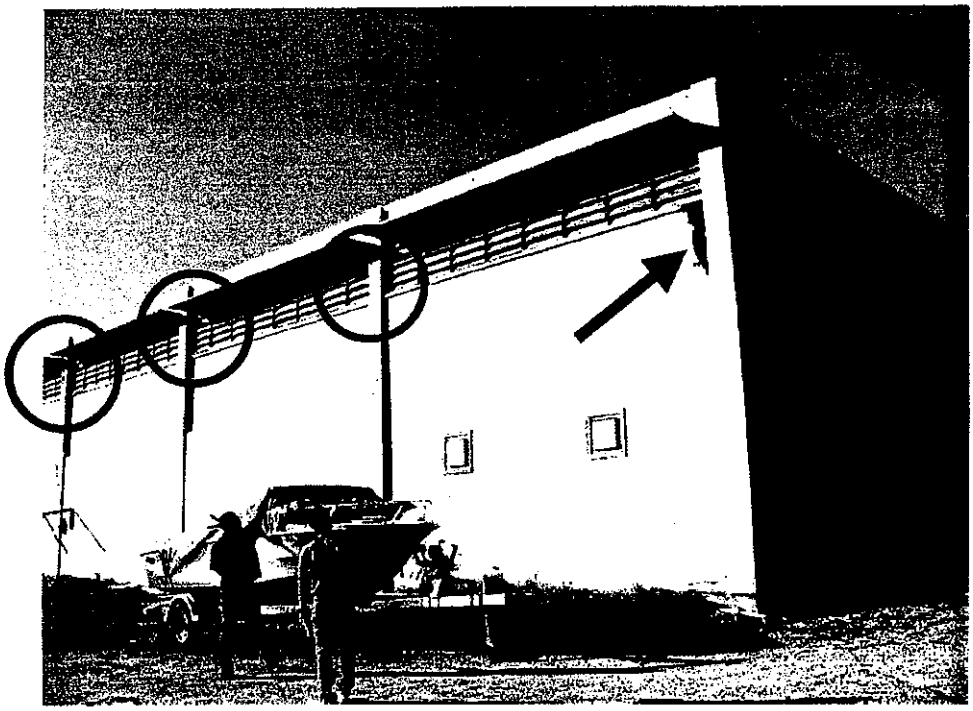


Photo 11- Damage at north-east wall at Brewer Building

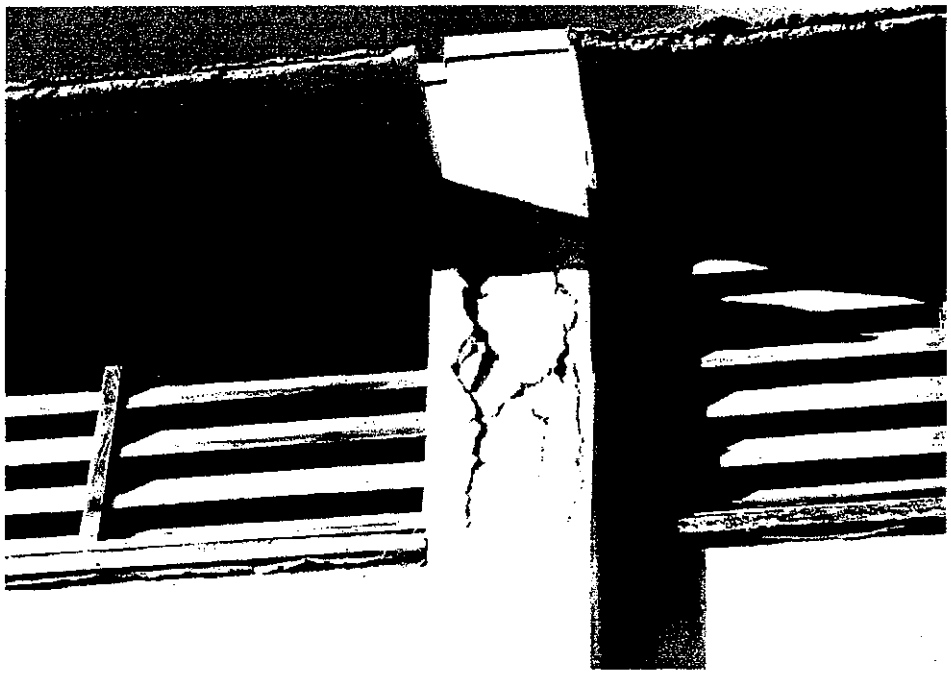
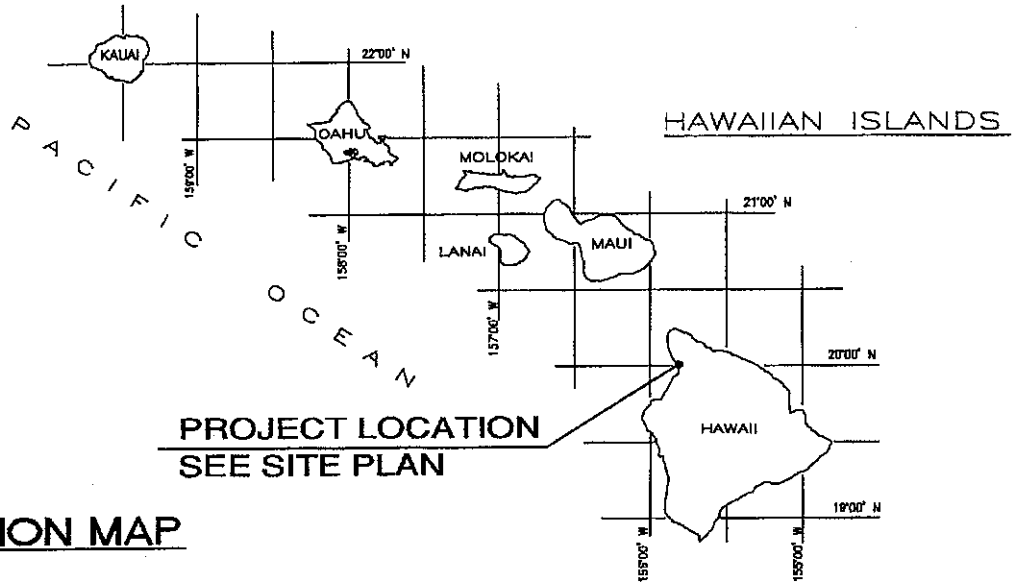


Photo 12- Shear failure of Concrete Columns at Brewer Building

FIGURES



LOCATION MAP

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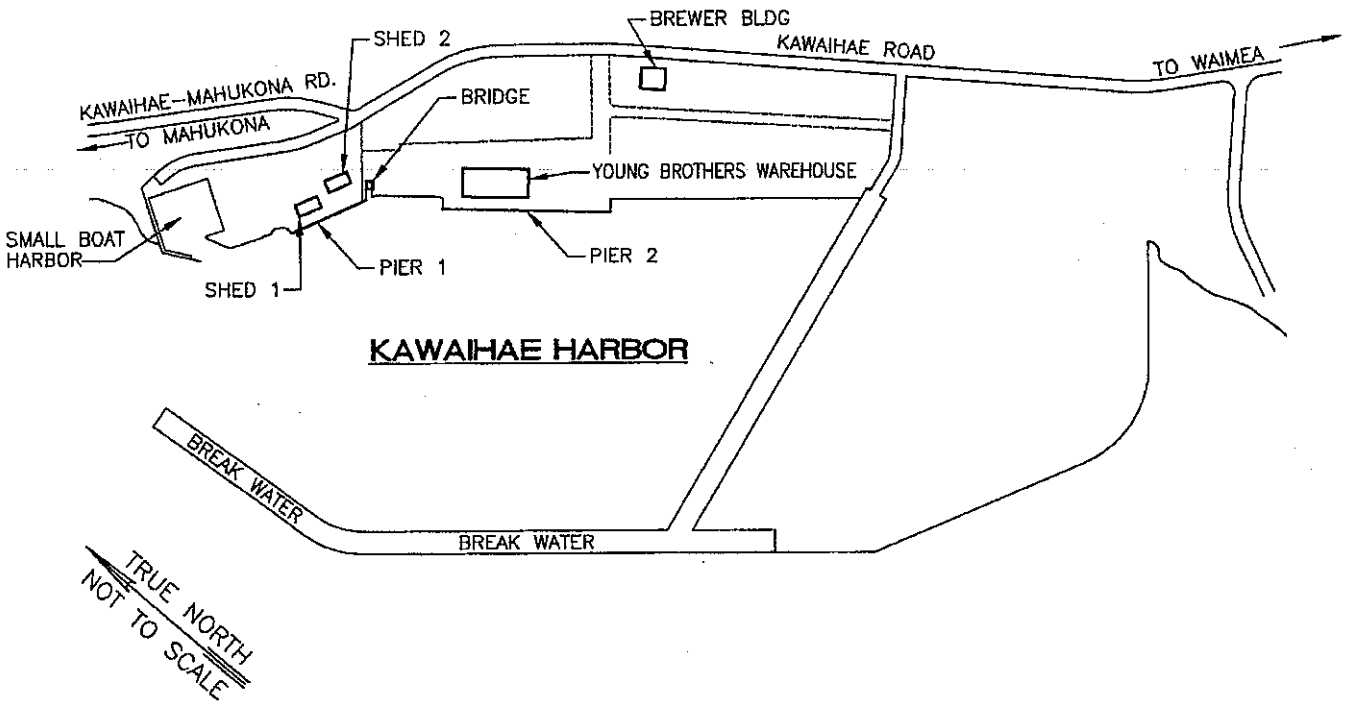

SITE PLAN

NOT TO SCALE

FIGURE 1	Date	EMERGENCY STRUCTURAL ASSESSMENT OF PIERS 1 & 2 AND VARIOUS STRUCTURES AT KAWAIHAE HARBOR, ISLAND OF HAWAII	Miyasato • Kuniyoshi ENGINEERS LLC Structural Engineering Services
	Scale		
	Drawn		
	Job		

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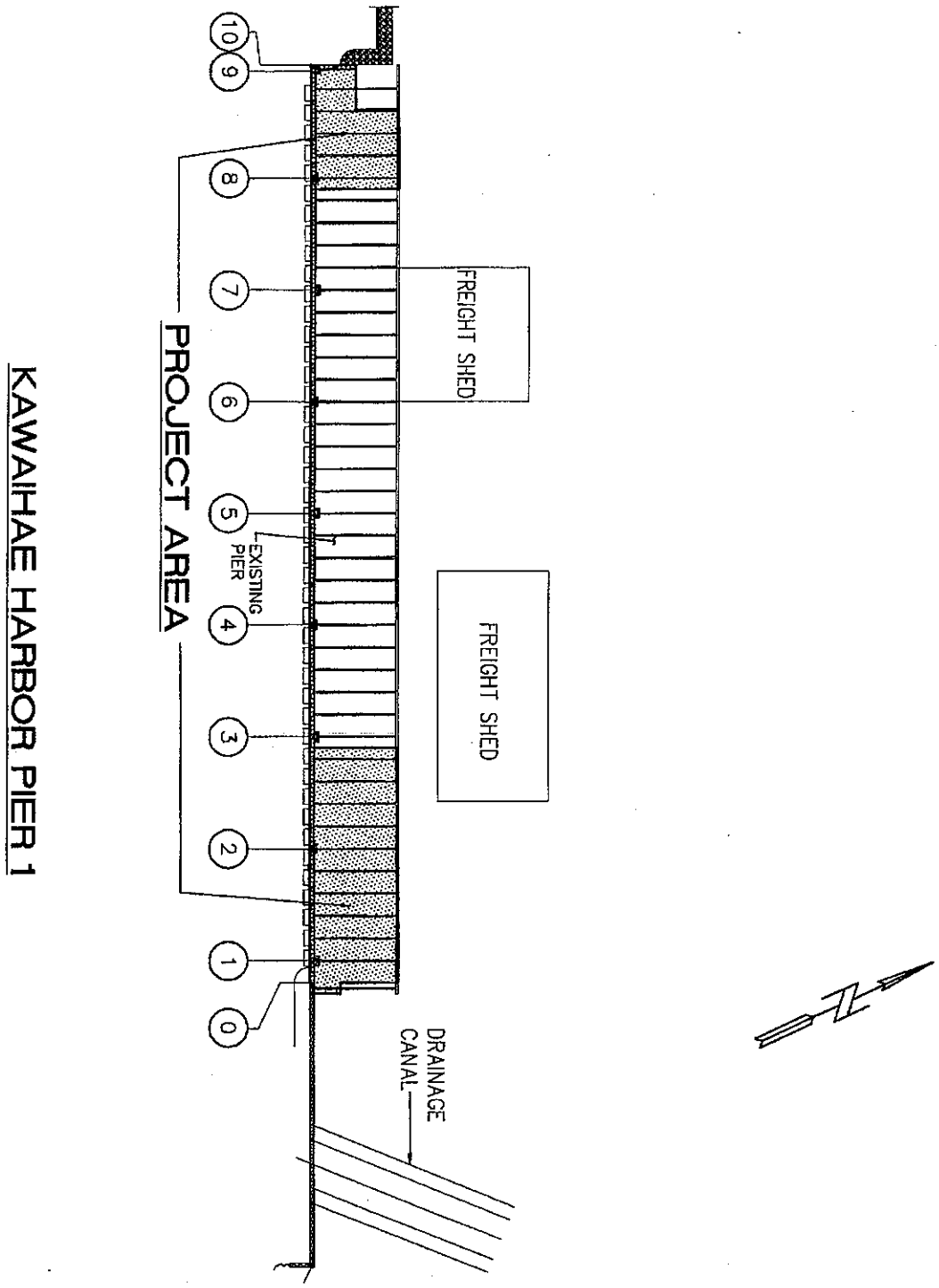
99-203 Moanalua Road • Aiea, Hawaii 96701
Phone: (808)486-7579 • Fax: (808)486-7519
E-mail: mkc@mkc.com



KAWAIHAE HARBOR PIER 1

FIGURE 2	Date	EMERGENCY STRUCTURAL ASSESSMENT OF PIERS 1 & 2 AND VARIOUS STRUCTURES AT KAWAIHAE HARBOR, ISLAND OF HAWAII	Miyasato Kuniyoshi ENGINEERS LLC Structural Engineering Services Aiea Commercial Center • Suite 205 98-205 Moanalua Road • Aiea, Hawaii 96701 Phone: (808)488-7579 • Fax: (808)488-7818 E-mail: miyak@mkeg.com
	Scale		
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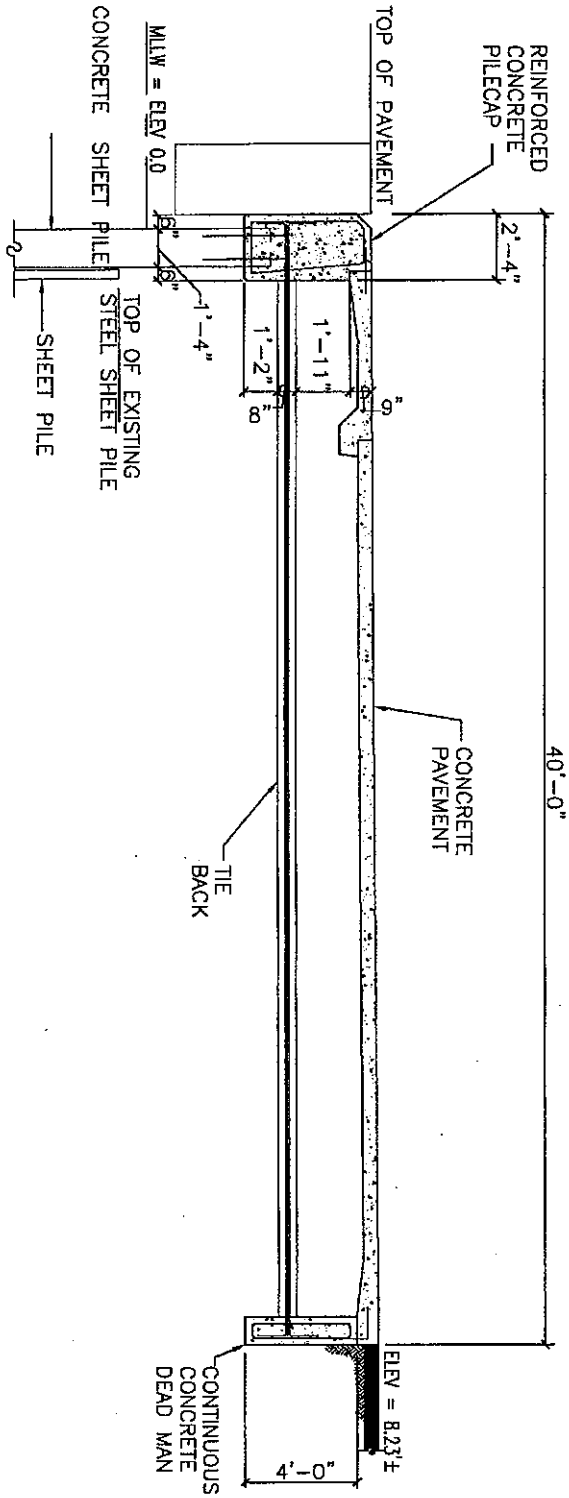
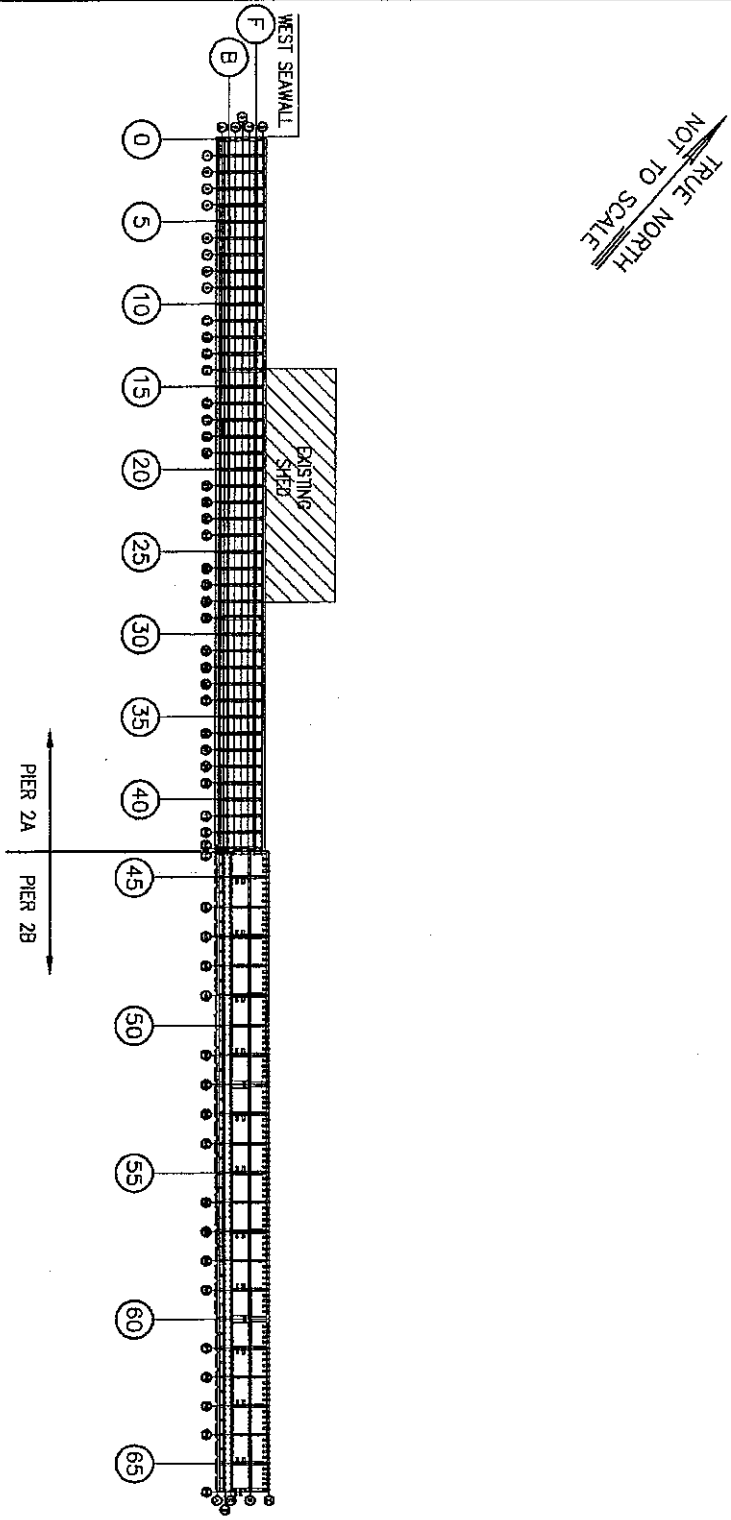
	Job		



TYPICAL SECTION AT PIER 1

SCALE: NONE

FIGURE 3	Date	EMERGENCY STRUCTURAL ASSESSMENT OF PIERS 1 & 2 AND VARIOUS STRUCTURES AT KAWAIHAE HARBOR, ISLAND OF HAWAII	Miyasato • Kuniyoshi ENGINEERS LLC Structural Engineering Services Aiea Commercial Center • Suite 205 99-203 Moanalua Road • Aiea, Hawaii 96701 Phone: (808)488-7579 • Fax: (808)488-7815 E-mail: mike@mkaiea.com
	Scale		
	Drawn		
	Job		



KAWAIHAE HARBOR PIER 2 PLAN

SCALE: NONE

FIGURE 4	Date	EMERGENCY STRUCTURAL ASSESSMENT OF PIERS 1 & 2 AND VARIOUS STRUCTURES AT KAWAIHAE HARBOR, ISLAND OF HAWAII	Miyasato • Kuniyoshi ENGINEERS LLC *Structural Engineering Services*
	Scale		
	Drawn		
	Job		

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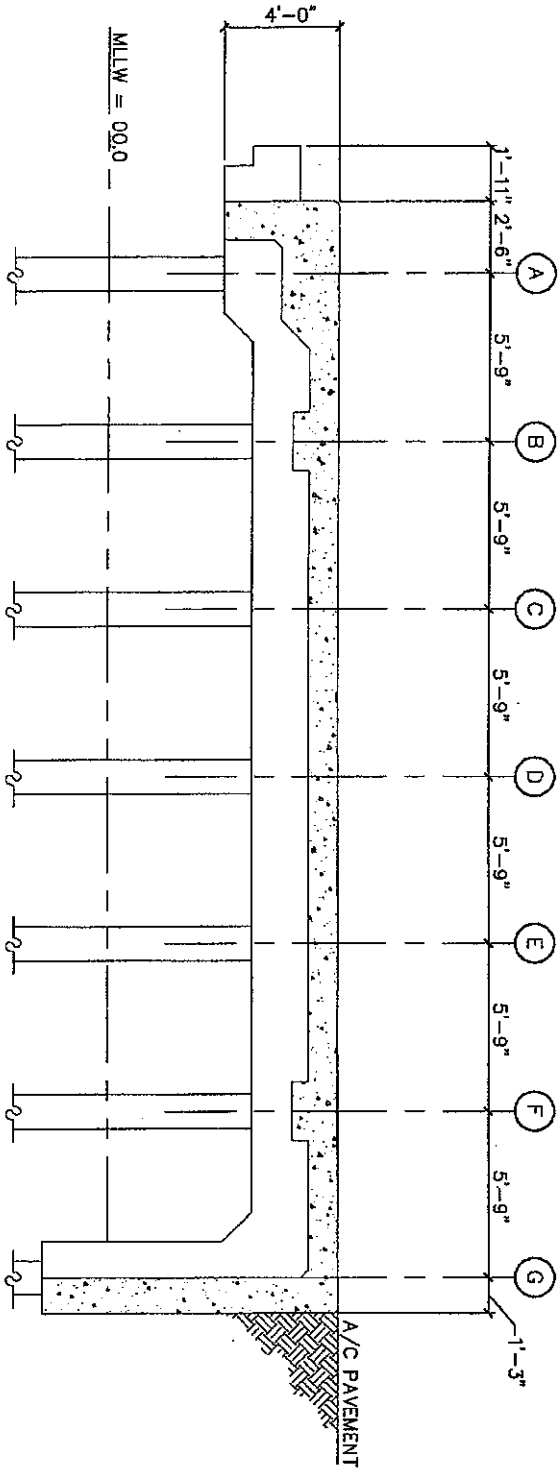

TYPICAL TRANSVERSE SECTION - PIER 2A

SCALE: NONE

FIGURE 5	Date	EMERGENCY STRUCTURAL ASSESSMENT OF PIERS 1 & 2 AND VARIOUS STRUCTURES AT KAWAIHAE HARBOR, ISLAND OF HAWAII	Miyasato - Kuniyoshi ENGINEERS LLC Structural Engineering Services
	Scale		
	Drawn		
	Job		

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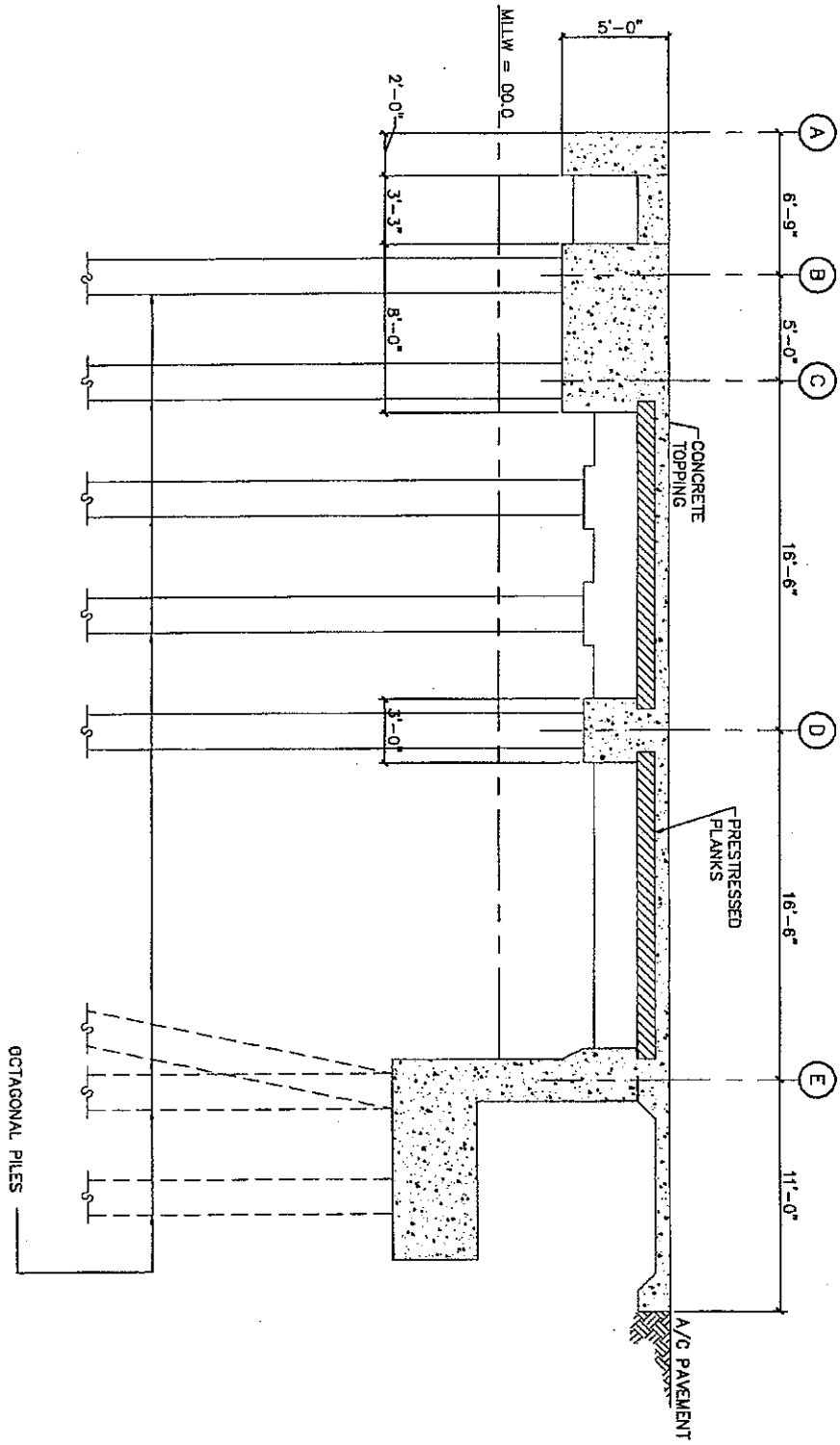

TYPICAL TRANSVERSE SECTION - PIER 2B

SCALE: NONE

FIGURE 6	Date	EMERGENCY STRUCTURAL ASSESSMENT OF PIERS 1 & 2 AND VARIOUS STRUCTURES AT KAWAIHAE HARBOR, ISLAND OF HAWAII	Miyasato • Kuniyoshi ENGINEERS LLC *Structural Engineering Services*
	Scale		
	Drawn		
	Job		

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Appendix C5
Geotechnical Report of Harbor Damage, 7 Borings, Liquefaction Evaluation
(Geolabs, Inc., January 12, 2007, 38 pages)

GEOLABS, INC.
Geotechnical Engineering and Drilling Services

January 12, 2007
W.O. 5762-00

Mr. Brian Kung
Miyasato Kuniyoshi Engineers LLC
Aiea Commercial Center, Suite 205
99-205 Moanalua Road
Aiea, HI 96701

Liquefaction Assessment
Piers 1 and 2 at Kawaihae Harbor
Kawaihae, South Kohala, Hawaii

Dear **Mr. Kung:**

As requested, we performed a geotechnical engineering exploration and conducted a liquefaction assessment of the subsurface materials encountered at the Piers 1 and 2 at Kawaihae Harbor sites in the District of South Kohala on the Island of Hawaii. The project site and vicinity are shown on the Project Location Map, Plate 1. This letter report summarizes our work and presents the findings and results of our liquefaction analyses of the subsurface materials encountered at the site. The liquefaction analyses were based on our field exploration and engineering analyses for the project.

PROJECT CONSIDERATIONS

Following the October 15, 2006 earthquakes, substantial damage to the piers, building structures, and surface pavements occurred at Kawaihae Harbor. Based on the substantial ground cracks and sandy soil deposits rising up from the subsurface to the surface along with groundwater, it was evident that widespread liquefaction and associated ground movements occurred during and following the two earthquakes that occurred on October 15, 2006. In an effort to review the subsurface conditions at the site and to evaluate the liquefaction potential of the subsurface materials, Geolabs, Inc. conducted a field exploration program to evaluate the subsurface materials and liquefaction potential assessment at the Kawaihae Harbor site.

The primary focus of the liquefaction potential assessment was at the Piers 1 and 2 locations of Kawaihae Harbor because these two areas suffered the most damage to the infrastructure. In order to conduct a liquefaction potential assessment of the subsurface materials at the Kawaihae Harbor sites, a limited field exploration program with an emphasis in obtaining data for the liquefaction potential assessment was performed at the project site. The borings were to be drilled with a 4-inch diameter drill bit utilizing rotary wash drilling methods. Bentonite mud slurry was used as the drilling fluid in the borings drilled to conduct liquefaction potential assessment.

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Hawaii • California

PURPOSE AND SCOPE

The purpose of our geotechnical engineering exploration was to obtain an overview of the surface and subsurface conditions at the Piers 1 and 2 locations at Kawaihae Harbor following the October 15, 2006 earthquake. The limited subsurface information obtained was utilized to perform a liquefaction assessment of the subsurface materials encountered at the project site. To accomplish this, we conducted an exploration program consisting of the following tasks and work efforts:

1. Reconnaissance of the project site by our principal engineer to observe the surface features and damages incurred shortly following the October 15, 2006 earthquake.
2. Coordination with State personnel on utility clearances at the proposed boring locations by our field engineer.
3. Mobilization and demobilization of a truck-mounted drill rig presently on the Island of Hawaii and two operators from Honolulu to and from the project site.
4. Drilling and sampling of seven borings extending to depths of approximately 46 to 86.5 feet below the existing ground surface.
5. Coordination of the field exploration and logging of the borings by our field engineer.
6. Laboratory testing of selected samples obtained during the field exploration as an aid in classifying the materials and evaluating their engineering properties.
7. Analyses of field and laboratory data to conduct liquefaction analyses and assessment.
8. Preparation of a letter report summarizing our work and presenting our findings and recommendations.
9. Quality assurance of our work and client/design team consultation by our principal engineer.
10. Miscellaneous work efforts, such as drafting, word processing, and clerical support.

Detailed descriptions of our field exploration methodology and the Logs of Borings are presented in Appendix A. Results of the laboratory tests performed on selected soil samples obtained from our field exploration are presented in Appendix B.

SUBSURFACE CONDITIONS

The subsurface conditions at the project site were explored by drilling and sampling seven borings, designated as Boring Nos. 1 through 7. Three borings (Boring Nos. 1 through 3) were drilled at the Pier 1 location, and four borings (Boring Nos. 4 through 7) were drilled at the Pier 2 location. Detailed descriptions of the materials encountered in the seven borings drilled (B-1 through B-7) are presented on the Logs of Borings in Appendix A.

The borings were extended to depths ranging from about 46 to 86.5 feet below the existing ground surface at the Pier 1 and Pier 2 locations at Kawaihae Harbor. A topographic survey was not available for our use in determining the existing ground elevations following the earthquake of October 15, 2006. We estimate the existing ground elevations at the boring locations are at approximately +8 to +12 feet Mean Sea Level (MSL). The approximate locations of the borings drilled for the project are shown on the Site Plan, Plate 2. The distribution of the borings drilled for the liquefaction assessment is described as follows.

LOCATION	APPLICABLE BORINGS
Pier 1 Area	Boring Nos. 1 through 3
Pier 2 Area	Boring Nos. 4 through 7

In general, the subsurface conditions at the Pier 1 area consist of dense granular surface fills (silty sands with gravel) placed over medium dense to dense coralline detritus deposits overlying basalt formation. The coralline detritus deposits encountered in the borings drilled generally consist of medium dense to very dense silty sands with coralline gravel. Due to the presence of some fines content and the normally erratic nature of the coralline detritus deposits, the relative density of the coralline detritus deposits varies considerably from medium dense to dense with some loose pockets. In some cases, the penetration resistance of the soils may range up to very dense conditions when advanced with a Standard Penetration Test (SPT) split-spoon sampler. The depths to the basalt formation encountered in the borings drilled at the Pier 1 area generally vary from about 31 to 45 feet below the ground level.

The subsurface conditions at the Pier 2 area generally are similar to the conditions at the Pier 1 area. The subsurface conditions at the Pier 2 area also consist of dense granular surface fills (silty sands with gravel) placed over loose to dense coralline detritus deposits overlying basalt formation. In some cases, alluvium consisting of cobbles and boulders or organic silts and clays were encountered embedded in the subsurface within the depths of the coralline detritus. The coralline detritus deposits encountered in the borings drilled generally consist of loose to dense silty sands with coralline gravel. Due to the normally erratic nature of the coralline detritus deposits, the relative density of the

coralline detritus deposits varies considerably from loose to dense with some pockets of alluvium. The depths to the basalt formation encountered in the borings drilled at the Pier 2 area generally vary from about 42 to 75 feet below the ground level.

EARTHQUAKES AND SEISMICITY

In general, earthquakes that occur throughout the world are caused by shifts in the tectonic plates. In contrast, earthquake activity in Hawaii is primarily linked to volcanic activity; therefore, earthquake activity in Hawaii generally occurs before or during volcanic eruptions. In addition, earthquakes may result from the underground movement of magma that comes close to the surface but does not erupt. The Island of Hawaii experiences thousands of earthquakes each year, but most are so small that only sensitive instruments can detect them. However, some of the earthquakes are strong enough to be felt, and a few cause minor to moderate damage.

In general, earthquakes associated with volcanic activity are most common on the Island of Hawaii. Earthquakes directly associated with the movement of magma are concentrated beneath the active Kilauea and Mauna Loa Volcanoes on the Island of Hawaii. Because majority of the earthquakes in Hawaii (over 90 percent) are related to volcanic activity, the risk of seismic activity and degree of ground shaking diminishes with increased distance from the active volcanoes located in the southern portion of the Island of Hawaii.

The Island of Hawaii has experienced numerous earthquakes greater than Magnitude 6 (M6+) including the recent October 15, 2006 earthquake. Based on information obtained from the United States Geological Survey (USGS) Bulletin 2006, the following is a list of some destructive earthquakes that occurred on the Island of Hawaii since 1868. It should be noted that the October 15, 2006 earthquake is not related to the movement of magma similar to the Honomu earthquake of 1973 (M6.2).

DATE	LOCATION	MAGNITUDE
March 28, 1868	South Hawaii	7.0
April 2, 1868	South Hawaii	7.9
October 5, 1929	Hualalai	6.5
August 21, 1951	Kona	6.9
April 26, 1973	North Hilo	6.2
November 29, 1975	Kalapana	7.2
November 16, 1983	Kaoiki	6.7
June 25, 1989	Kalapana	6.2
October 15, 2006	Kiholo Bay / Hawi	6.7 / 6.0

It should be noted that several of the significant earthquakes on the Island of Hawaii have occurred on the west side of the island in the past 100 years including two earthquakes in the Kailua-Kona area with magnitudes greater than Magnitude 6 in 1929 and 1951. In addition, the recent October 15, 2006 earthquakes are of Magnitudes 6.0 and 6.7. Therefore, it may be concluded that the west side of the island could experience moderate to severe earthquakes and associated ground shaking depending on the location of the earthquake origin.

Earthquake of October 15, 2006

On Sunday morning of October 15, 2006, two strong earthquakes with magnitudes of M6.7 and M6.0 occurred offshore from the northwestern coast of the Island of Hawaii in the vicinity of Kiholo Bay and Hawi. The western portion of the Island of Hawaii experienced strong to very strong ground shaking, which diminished to generally moderately strong shaking at the eastern half of the Island of Hawaii and the eastern portion of the Island of Maui. Moderate ground shaking was reported throughout the State of Hawaii including as far as the Island of Kauai.

Deaths were not attributed to the earthquake; however, some minor injuries were experienced and several hundred million dollars in damage to structures and property was reported, mainly on the Island of Hawaii and portions of the Island of Maui. It is believed that because the earthquakes occurred early Sunday morning in the closest proximity of rural areas, losses were minimized. The earthquakes were followed for weeks by hundreds of smaller aftershocks, with some of the larger aftershocks occurring in the range of magnitude 4 to 5.

The Kiholo Bay earthquake (M6.7) occurred at 7:07 AM with an epicenter location at 19.878 North Latitude, 155.935 West Longitude with a focal depth of about 24 miles. At 7:14 AM, the Hawi earthquake occurred with a Magnitude of 6.0 with an epicenter at 20.129 North Latitude, 155.983 West Longitude with a focal depth of about 12 miles (USGS website, 2006). The Hawi earthquake is not considered an aftershock because of the distinct source location although it likely was triggered by the larger preceding Kiholo Bay earthquake. The earthquakes are considered unusual events due to their location away from documented fault systems and the relatively deep focal depth. The cause of the earthquakes is believed related to tectonic flexing of the oceanic crust resulting from the accumulating weight of the island mass as the island continues to grow in size instead of the movement of magma typical of the Island of Hawaii.

Most of the existing infrastructure in the vicinity of the earthquake (Kona and Kohala Coasts) experienced little damage; however, some locally significant damage to buildings, water reservoirs, highways, rock slopes, irrigation infrastructure, and harbors on the west side of the Island of Hawaii were reported. Damages included spalled, cracked, and separated concrete structures, collapse of un-reinforced masonry walls, development of wide-open ground cracks, settlement due to liquefaction, rock slope, boulder, and embankment failures, and some shallow localized soil landslides. In addition, many

homes experienced cracks in walls and shifted post and pier foundations especially in the Kohala region. Reports of significant damage generally diminished toward the more populated areas of Kailua-Kona and Hilo.

For the Kiholo earthquake, recorded ground accelerations show a maximum horizontal ground acceleration of 1.03g and a maximum vertical ground acceleration of 0.72g at the Waimea Fire Station, which is 20 miles from the Kiholo Bay earthquake epicenter. Horizontal and vertical peak ground accelerations (PGA) of 0.64g and 0.35g, respectively, were recorded at the Honokaa Fire Station while horizontal and vertical PGAs of 0.51g and 0.28g, respectively, were recorded at the Kona Hospital in Kealakekua. The maximum horizontal and vertical PGAs recorded in the Hilo area were 0.23g and 0.10g, respectively, at the USDA laboratory station. The ground shaking recorded were significantly lower for the Hawi earthquake. Shaking reached Intensity VIII on the Modified Mercalli Intensity Scale (MMI) as reported by Island of Hawaii residents. The strong ground motions were felt for about 15 to 20 seconds during the two earthquakes.

Seismic Design Considerations

The project site will be subjected to seismic activity and should be evaluated for the potential for soil liquefaction. Typically two levels of design earthquakes, a functional (operating) and a contingency design level, are used for design of a structure, as recommended in the Design Criteria for Earthquake Hazard Mitigation of Navy Piers and Wharves (1997), Technical Report TR-2069-SHR, prepared by the Naval Facilities Engineering Service Center. The concept of dual-design earthquakes encompasses a lower level earthquake (designated as the Level I design earthquake), during which only minimal structural and architectural damage is expected to occur, and a higher level earthquake (Level II design earthquake), for which structures should be designed to include sufficient ductility to avoid catastrophic failure or collapse. The functional and contingency design earthquake levels may be taken as the Level I and Level II earthquakes, respectively. Based on the U.S. Navy Piers and Wharves Manual, the Level I earthquake should have a 50% probability of exceedance in 50 years, and the Level II earthquake should have a 10% probability of exceedance in 50 years.

Based on probabilistic seismic hazard analyses, the design peak ground acceleration associated with the Level 1 (50% probability of exceedance in 50 years) and Level 2 (10% probability of exceedance in 50 years) design earthquakes are 0.18g and 0.36g, respectively. We understand that the probable magnitudes of the earthquakes associated with the Level 1 and Level 2 design earthquakes are Magnitude 6.0 and Magnitude 6.5 seismic events. Based on the design peak ground accelerations and the design magnitude earthquakes, we conducted a liquefaction potential assessment of the saturated granular soils at the Piers 1 and 2 at Kawaihae Harbor sites.

LIQUEFACTION ASSESSMENT

Soil liquefaction is a condition where saturated cohesionless soils located near the ground surface undergo a substantial loss of strength due to the build-up of excess pore water pressures resulting from cyclic stress applications induced by earthquakes. In this process, when the loose saturated sand deposit is subjected to vibration (such as during an earthquake), the soil tends to densify and decrease in volume causing an increase in pore water pressure. If drainage is unable to occur rapidly enough to dissipate the build-up of pore water pressure, the effective stress (internal strength) of the soil is reduced. Under sustained vibrations, the pore water pressure build-up could equal the overburden pressure, essentially reducing the soil shear strength to zero and causing it to behave as a viscous fluid. During liquefaction, the soil acquires sufficient mobility to permit both horizontal and vertical movements, and if not confined, will result in significant deformations.

Soils most susceptible to liquefaction are loose, uniformly graded, fine-grained sands and loose silts with little cohesion. The major factors affecting the liquefaction characteristics of a soil deposit are as follows:

1. Grain Size Distribution - Fine and uniform sands and silts are more susceptible to liquefaction than coarse or well-graded sands.

2. Initial Relative Density - Loose sands and silts are most susceptible to liquefaction. Liquefaction potential is inversely proportional to relative density.

3. Magnitude and Duration of Vibration - Liquefaction potential is directly proportional to the magnitude and duration of the earthquake.

We evaluated the liquefaction potential of the saturated granular soils at the Piers 1 and 2 at Kawaihae Harbor sites based on the procedures outlined by Seed and Idriss (1982) and Seed, Idriss and Arango (1983) and updated by the National Center for Earthquake Engineering Research (NCEER) in 1997. In general, the field SPT N-values obtained from the seven borings (Boring Nos. 1 through 7) were normalized for the standard corrections, such as overburden pressure, fines content, and design earthquake magnitude, in the liquefaction potential assessment. The seven borings were drilled utilizing approximately 4-inch diameter rotary wash methods with bentonite mud.

In general, the subsurface information obtained from the borings indicates that coralline detritus deposits and basalt formation underlie the project site. It should be noted that the relative density of the coralline detritus deposits encountered at the Kawaihae Harbor sites is highly variable ranging from loose to dense sands with gravel and traces of silt. In some cases, very dense conditions (based on SPT sampler penetration resistance) were encountered indicating highly variable and erratic conditions at the project site.

Some of the coralline detritus deposits consist of loose silty sands with some cohesion, which may be considered as potentially liquefiable during seismic events. Based on the subsurface conditions encountered and a design peak ground acceleration of 0.18g (Level 1 EQ), only a small portion of the subsurface profile may be susceptible to liquefaction. For the Level 2 EQ with a design peak ground acceleration of 0.36g, a larger portion of the subsurface profile may be susceptible to liquefaction. Therefore, the effects of potential liquefaction should be taken into account in the design of future improvements at the sites.

To evaluate the conditions that occurred as a result of the October 15, 2006 earthquake, we adopted an estimated PGA of 0.60g in our liquefaction analyses considering that PGAs of 0.60g were encountered at locations further away from the earthquake epicenter than Kawaihae Harbor. Using an estimated PGA of 0.60g and Magnitude 6.7, a substantial portion of the soil profile appears to have liquefied.

The results of our liquefaction potential assessment of the borings drilled and the seismically induced settlement estimates of the soil profile encountered in the borings are summarized in the following table. The seismically induced settlement estimates are based generally on the simplified procedures outlined by Tokimatsu and Seed (1987).

Boring Location	**Approximate Depth of Liquefiable Zone** (feet)			**Seismically Induced Settlement Estimates** (inches)		
	Level 1 EQ	**Level 2 EQ**	**Oct. 15 EQ**	**Level 1 EQ**	**Level 2 EQ**	**Oct. 15 EQ**
B-1	None	14.5 - 24.5	14.5 - 44.5	Negligible	2.5	4
B-2	None	None	10 - 14.5, & 24.5-38.5	Negligible	Negligible	2
B-3	None	10 - 14	10 - 14, & 24 - 31.5	Negligible	1.5	1.5
B-4	16.5-23.5	16.5 - 23.5, 38.5 - 58.5	5 - 10, & 16.5 - 58.5	2	8	10
B-5	None	39 - 44	29 - 44	Negligible	1	1.5
B-6	None	21.5 - 45, 46.5 - 56.5	22 - 45, 46.5 - 56.5, & 61.5 - 65.5	Negligible	9	9
B-7	None	None	34.5 - 39.5	Negligible	Negligible	1

Based on field observations following the October 15, 2006 earthquake, it was evident that widespread liquefaction occurred and significant differential settlements were observed at the Kawaihae Harbor site. From the results of the liquefaction assessment presented above, it appears that the seismically induced settlement estimates provided in the table above are somewhat consistent with the reported settlements observed at the Piers 1 and 2 at Kawaihae Harbor sites following the October 15, 2006 earthquakes.

Based on the results of the above preliminary liquefaction analyses, it appears that the seismically induced settlement estimates for the two design earthquakes are approximately 0 to 2 inches for the Level 1 earthquake and 2 to 9 inches for the Level 2 earthquake. From our field exploration, it is evident that the subsurface profile at the two pier locations are highly variable; therefore, it is reasonable to expect that the majority of the seismically induced settlements from liquefaction likely will result in significant differential settlements across the project site.

It also should be noted that these seismically induced settlement estimates exceed the threshold established by the U.S. Navy for the Level 1 and Level 2 earthquakes for naval facilities. The U.S. Navy Piers and Wharves Manual considers the settlement performance goal to be 1 inch and 4 inches for the Level 1 and Level 2 earthquakes, respectively. Therefore, the seismically induced settlement estimates at the Piers 1 and 2 at Kawaihae Harbor sites do not meet these performance goals.

Based on the preliminary liquefaction analyses and the results of the liquefaction assessment, we recommend that additional investigation and/or liquefaction mitigation alternatives be developed for detailed review and implementation because our preliminary liquefaction assessment indicates that the seismically induced settlements at the Piers 1 and 2 at Kawaihae Harbor do not meet the performance goals for typical pier and wharf facilities.

LIMITATIONS

The analyses and findings submitted in this report are based in part upon information obtained from the test borings and laboratory tests. Variations of conditions between and beyond the test borings may occur, and the nature and extent of these variations may not become evident until additional investigation or construction is underway. If variations then appear evident, it will be necessary to re-evaluate the findings presented in this report.

The stratification breaks shown on graphic representations of the borings depict the approximate boundaries between soil and/or rock types and, as such, may denote a gradual transition. Water level data from the borings drilled were measured at the times shown on the graphic representations and/or presented in the text of this report. These data have been reviewed and interpretations made in the formulation of this report. However, it must be noted that fluctuation may occur due to variation in seasonal rainfall, and other factors.

This report has been prepared for the exclusive use of Miyasato Kuniyoshi Engineers LLC and their client, State of Hawaii – Department of Transportation, Harbors Division for specific application to the Liquefaction Assessment for Piers 1 and 2 at Kawaihae Harbor in accordance with generally accepted geotechnical engineering principles and practices. No warranty is expressed or implied.

This report has been prepared solely for the purpose of assisting the engineers in the planning and preliminary design for the project. Therefore, this report may not contain sufficient data, or the proper information, for use to form the basis for preparation of detailed design and/or construction cost estimates. A contractor wishing to bid on this project should retain a competent geotechnical engineer to assist in the interpretation of this report and/or performance of site-specific exploration for bid estimating purposes.

The owner/client should be aware that unanticipated soil conditions are encountered commonly. Unforeseen subsurface conditions, such as perched groundwater, soft deposits, hard layers, or cavities, may occur in localized areas and may require additional probing or corrections in the field (which may result in construction delays) to attain a properly constructed project. Therefore, a sufficient contingency fund is recommended to accommodate these possible extra costs.

CLOSURE

We appreciate the opportunity to provide geotechnical engineering services to you on this project. If you have questions or need additional information, please contact our office.

Respectfully submitted,

GEOLABS, INC.

By _____
Robin M. Lim, P.E.
Vice President

RML:mj

ROBIN M. LIM
LICENSED PROFESSIONAL ENGINEER
No. 8436-C
HAWAII, U.S.A.

THIS WORK WAS PREPARED BY ME OR UNDER MY SUPERVISION.

SIGNATURE

4-30-08
EXPIRATION DATE OF THE LICENSE

Attachments: Project Location Map - Plate 1
Site Plan - Plate 2
Field Exploration – Appendix A (17 sheets)
Laboratory Testing – Appendix B (6 sheets)

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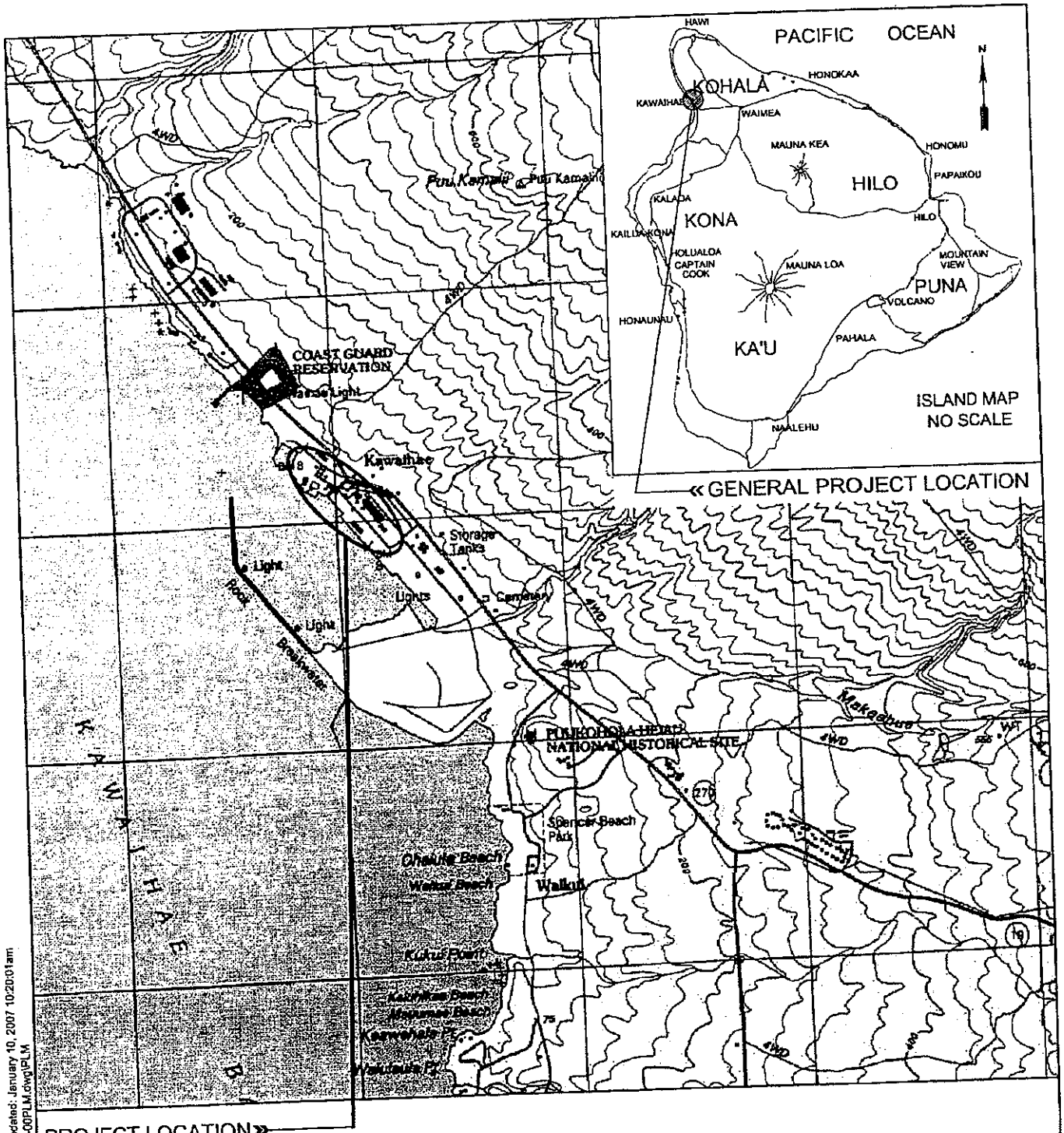
PLATES



PROJECT LOCATION MAP

LIQUEFACTION ASSESSMENT
PIERS 1 AND 2 AT KAWAIHAE HARBOR
KAWAIHAE HARBOR, SOUTH KOHALA, HAWAII

2000 1000 0 2000 FT.
GRAPHIC SCALE

N

REFERENCE: MAP CREATED WITH TOPO!® ©2001 NATIONAL GEOGRAPHIC (WWW.NATIONALGEOGRAPHIC.COM/TOPO).

GEOLABS, INC. Geotechnical Engineering		
DATE DECEMBER 2006	DRAWN BY HYC	PLATE
SCALE 1" = 2,000'	W.O. 5762-00	1

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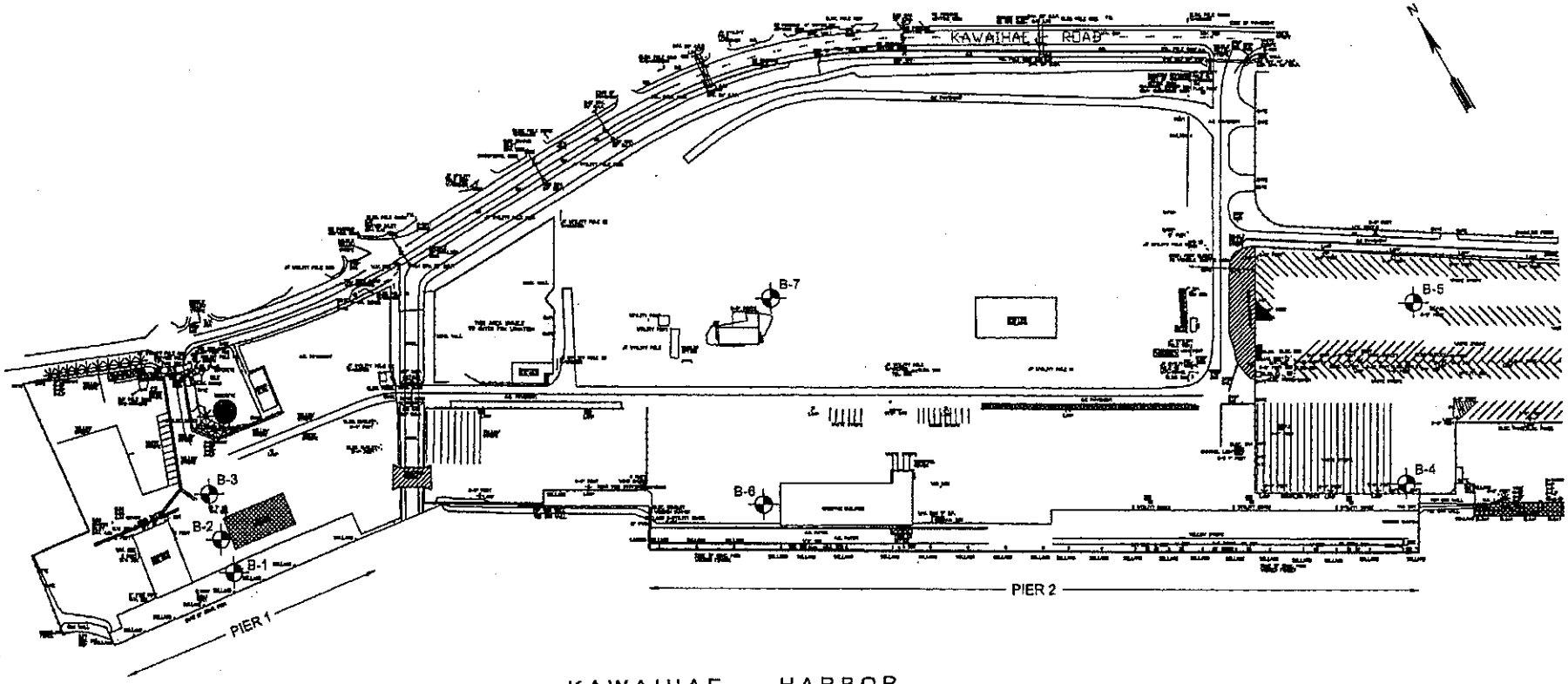

KAWAIHAE HARBOR

LEGEND:

APPROXIMATE BORING LOCATION

REFERENCE: SITE SURVEY TRANSMITTED BY HIDOT ON JANUARY 8, 2007.

SITE PLAN

LIQUEFACTION ASSESSMENT
PIERS 1 AND 2 AT KAWAIHAE HARBOR
KAWAIHAE HARBOR, SOUTH KOHALA, HAWAII

GEOLABS, INC.
Geotechnical Engineering

DATE	DRAWN BY	PLATE
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User: HENRY File Created: January 09, 2007 File Last Updated: January 18, 2007 10:20:11 am
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APPENDIX A

Field Exploration

APPENDIX A

Field Exploration

The subsurface conditions at the project site were explored by drilling and sampling seven borings, designated as Boring Nos. 1 through 7, extending to depths ranging from about 46 to 86.5 feet below the existing ground surface. The borings were drilled with a truck-mounted drill rig equipped with rotary wash drilling method and coring tools. Bentonite mud slurry was used as the drilling fluid in the borings drilled to conduct liquefaction potential assessment. The seven borings were drilled at the approximate locations shown on the Site Plan, Plate 2.

The materials encountered in the borings were classified by visual and textural examination in the field by our field engineer, who monitored the drilling operations on a near-continuous basis. These field classifications were further reviewed visually and by testing in the laboratory. Soils were classified in general conformance with the Unified Soil Classification System, as shown on Plate A. Graphic representations of the materials encountered are provided on the Logs of Borings, Plates A-1 through A-7.

The soil samples were obtained from the borings drilled in general accordance with ASTM D 1586, Penetration Test and Split-Barrel Sampling of Soils, by driving a 2-inch OD standard penetration sampler using a 140-pound hammer falling 30 inches. The blow counts needed to drive the sampler the second and third 6 inches of an 18-inch drive are shown as the "Penetration Resistance" on the Logs of Borings at the appropriate sample depths.

Core samples of the basalt formations encountered at the site were obtained using diamond core drilling techniques in general accordance with ASTM D 2113, Diamond Core Drilling for Site Investigation. Core drilling is a rotary drilling method that uses a hollow bit to cut into the basalt formation. The material left in the hollow core of the bit is mechanically recovered for examination and description.

Recovery (REC) is used as a subjective guide to the interpretation of the relative quality of rock masses. Recovery is defined as the actual length of material recovered from a coring attempt versus the length of the core attempt. For example, if 3.7 feet of material is recovered from a 5.0-foot core run, the recovery would be 74 percent and would be shown on the Logs of Borings as REC = 74%.

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GEOLABS, INC.

Geotechnical Engineering

Log Legend

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

MAJOR DIVISIONS			USCS	TYPICAL DESCRIPTIONS
COARSE-GRAINED SOILS MORE THAN 50% OF MATERIAL RETAINED ON NO. 200 SIEVE	GRAVELS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS LESS THAN 5% FINES	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
			GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES MORE THAN 12% FINES	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS 50% OR MORE OF COARSE FRACTION PASSING THROUGH NO. 4 SIEVE	CLEAN SANDS LESS THAN 5% FINES	SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES MORE THAN 12% FINES	SM	SILTY SANDS, SAND-SILT MIXTURES
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE-GRAINED SOILS 50% OR MORE OF MATERIAL PASSING THROUGH NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50	ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT 50 OR MORE	MH	INORGANIC SILT, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

LEGEND

- (2-INCH) O.D. STANDARD PENETRATION TEST
- (3-INCH) O.D. MODIFIED CALIFORNIA SAMPLE
- SHELBY TUBE SAMPLE
- GRAB SAMPLE
- CORE SAMPLE

LL	LIQUID LIMIT
PI	PLASTICITY INDEX
TV	TORVANE SHEAR (tsf)
PEN	POCKET PENETROMETER (tsf)
UC	UNCONFINED COMPRESSION (psi)
▽	WATER LEVEL OBSERVED IN BORING

Plate A

GEOLABS, INC.

Geotechnical Engineering

LIQUEFACTION ASSESSMENT
PIERS 1 AND 2 AT KAWAIHAE HARBOR
KAWAIHAE HARBOR, SOUTH KOHALA, HAWAII

Log of Boring **1**

Approximate Ground Surface Elevation (feet MSL): 8 *

Laboratory: Other Tests	Laboratory: Moisture Content (%)	Laboratory: Dry Density (pcf)	Field: Core Recovery (%)	Field: RQD (%)	Field: Penetration Resistance (blows/foot)	Field: Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
											12-inch **CONCRETE APRON**
	14				76					SP-SM	Tannish brown fine to medium grained **SILTY SAND** with some gravel (coralline), very dense, damp (fill)
											grades with some gravel (basaltic) at 2 feet
	24				49		5				grades to very moist
	37				25		10			SP-SM	Grayish brown **GRAVELLY SAND (CORALLINE)** with traces of silt and shells, medium dense
-#200 =5%	32				11		15				grades to loose to medium dense
	44				16		20				grades with traces of clay seams, loose
-#200 =4%	34				25		25			SP	Gray fine grained **SAND (CORALLINE)** with shells, loose to medium dense
	28				54		30				grades to very dense
	24				44		35			SP-SM	

Date Started:	October 30, 2006	Water Level: ∇ 5.5 ft. 10/30/06 1140 HRS	
Date Completed:	October 31, 2006		
Logged By:	E. Shinsato	Drill Rig:	MOBILE B-53
Total Depth:	57 feet	Drilling Method:	Rotary Wash w/Bentonite & HQ Coring
Work Order:	5762-00	Driving Energy:	140 lb. wt., 30 in. drop

Plate

A - 1.1

BORING LOG 5762-00.GPJ GEOLABS.GDT 1/12/07

GEOLABS, INC. Geotechnical Engineering	LIQUEFACTION ASSESSMENT PIERS 1 AND 2 AT KAWAIHAE HARBOR KAWAIHAE HARBOR, SOUTH KOHALA, HAWAII	Log of Boring 1

Laboratory			Field								
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
											(Continued from previous plate)
										SP-SM	Gray to dark gray well-graded **GRAVELLY SAND (CORALLINE)** with traces of silt and shells, very dense
-#200 =6%	28				24		40				grades to tan, medium dense
	20				57		45				Dark grayish brown with red mottling **BASALT**, moderately weathered, medium hard grades to hard
			100	100			50				Gray vesicular **BASALT**, slightly fractured, slightly weathered, medium hard (basalt formation)
			100	80			55				grades to severely fractured, moderately to highly weathered, soft rock grades to slightly fractured, slightly weathered, medium hard
											Boring terminated at 57 feet
							60				
							65				
							70				

Date Started:	October 30, 2006	Water Level: ▽	5.5 ft. 10/30/06 1140 HRS	Plate
Date Completed:	October 31, 2006			
Logged By:	E. Shinsato	Drill Rig:	MOBILE B-53	A - 1.2
Total Depth:	57 feet	Drilling Method:	Rotary Wash w/Bentonite & HQ Coring	
Work Order:	5762-00	Driving Energy:	140 lb. wt., 30 in. drop	

GEOLABS, INC. Geotechnical Engineering	LIQUEFACTION ASSESSMENT PIERS 1 AND 2 AT KAWAIHAE HARBOR KAWAIHAE HARBOR, SOUTH KOHALA, HAWAII	Log of Boring 2

Approximate Ground Surface Elevation (feet MSL): 8 *

Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
											5-inch **ASPHALTIC CONCRETE**
	12				55/.5' Ref.					SM	Tan **SILTY SAND (CORALLINE)** with some gravel (coralline), very dense, moist (fill)
	23				35		5				grades to dense, very moist
-#200 =8%	32				13		10			SP-SM	Gray **GRAVELLY SAND (CORALLINE)** with traces of silt and shells, medium dense
	53				28		15				
	38				20		20				
-#200 =15%	34				27		25				grades to silty sand with gravel
	28				46		30				grades to dense
	22				48		35			SP-SM	Gray to dark gray well-graded **GRAVELLY SAND (CORALLLINE)** with traces of silt, dense

Date Started:	October 31, 2006	Water Level: ∇ 5.8 ft. 10/31/06 1328 HRS	Plate
Date Completed:	November 1, 2006		
Logged By:	E. Shinsato	Drill Rig: MOBILE B-53	A - 2.1
Total Depth:	56.5 feet	Drilling Method: Rotary Wash w/Bentonite & HQ Coring	
Work Order:	5762-00	Driving Energy: 140 lb. wt., 30 in. drop	

GEOLABS, INC. Geotechnical Engineering	LIQUEFACTION ASSESSMENT PIERS 1 AND 2 AT KAWAIHAE HARBOR KAWAIHAE HARBOR, SOUTH KOHALA, HAWAII	Log of Boring 2

Laboratory			Field								
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
										SP-SM	(Continued from previous plate)
			50	37							Gray **COBBLES (BASALTIC)** with some brown silt and rounded gravel (alluvium)
							40				Gray vesicular **BASALT**, severely fractured, moderately weathered, medium hard
			30	39							
											BASALT FRAGMENTS (clinker)
							45				
			37	46	10/.0' Ref.						Gray vesicular **BASALT**, moderately fractured, slightly weathered, medium hard to hard (basalt formation)
											BASALT FRAGMENTS (clinker)
							50				
			100	90							Gray vugular **BASALT**, slightly fractured, slightly weathered, medium hard to hard (basalt formation)
							55				
											Boring terminated at 56.5 feet
							60				
							65				
							70				

Date Started:	October 31, 2006	Water Level: ▽	5.8 ft. 10/31/06 1328 HRS	Plate
Date Completed:	November 1, 2006			
Logged By:	E. Shinsato	Drill Rig:	MOBILE B-53	A - 2.2
Total Depth:	56.5 feet	Drilling Method:	Rotary Wash w/Bentonite & HQ Coring	
Work Order:	5762-00	Driving Energy:	140 lb. wt., 30 in. drop	

GEOLABS, INC.

Geotechnical Engineering

LIQUEFACTION ASSESSMENT
PIERS 1 AND 2 AT KAWAIHAE HARBOR
KAWAIHAE HARBOR, SOUTH KOHALA, HAWAII

Log of Boring 3

Approximate Ground Surface Elevation (feet MSL): 8 *

Laboratory			Field								Description
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	
											5-inch **ASPHALTIC CONCRETE**
	20				50					SP SM	Tan medium grained **SILTY SAND (CORALLINE)** with some gravel (coralline), dense, damp
	14				42		5			GP	Tan **SANDY GRAVEL (CORALLINE)** with traces of silt, dense
	30				10		10			SM	Gray medium grained **SILTY SAND (CORALLINE)** with some gravel (coralline) and shells, loose
	23				36		15			SP-SM	Dark grayish brown **GRAVELLY SAND (CORALLINE)** with traces of silt and shells, medium dense to dense
-#200 =6%	35				25		20				grades to gray, medium dense
	30				37		25			SP-SM	Gray fine grained **SAND (CORALLINE)** with traces of silt and shells, dense
-#200 =6%	31				48		30				grades with traces of gravel (coralline), very dense
			0	0			35				Dark brown **BASALT FRAGMENTS** (clinker)

Date Started:	November 6, 2006	Water Level: ∇ 5.6 ft. 11/6/06 1040 HRS	Plate
Date Completed:	November 6, 2006		
Logged By:	E. Shinsato	Drill Rig: MOBILE B-53	A - 3.1
Total Depth:	46 feet	Drilling Method: Rotary Wash w/Bentonite & HQ Coring	
Work Order:	5762-00	Driving Energy: 140 lb. wt., 30 in. drop	

BORING LOG 5762-00.GPJ GEOLABS.GDT 1/12/07

GEOLABS, INC.

Geotechnical Engineering

LIQUEFACTION ASSESSMENT
PIERS 1 AND 2 AT KAWAIHAE HARBOR
KAWAIHAE HARBOR, SOUTH KOHALA, HAWAII

Log of Boring

3

Laboratory			Field							
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
											(Continued from previous plate)
			52	36							
							40				Gray vesicular **BASALT**, severely fractured, moderately weathered, medium hard (basalt formation)
			100	100	10/.0' Ref.						Gray vesicular **BASALT**, slightly fractured, slightly weathered, medium hard to hard (basalt formation)
							45				
											Boring terminated at 46 feet
							50				
							55				
							60				
							65				
							70				

Date Started:	November 6, 2006	Water Level: ▽ 5.6 ft. 11/6/06 1040 HRS	Plate
Date Completed:	November 6, 2006		
Logged By:	E. Shinsato	Drill Rig: MOBILE B-53	A - 3.2
Total Depth:	46 feet	Drilling Method: Rotary Wash w/Bentonite & HQ Coring	
Work Order:	5762-00	Driving Energy: 140 lb. wt., 30 in. drop	

BORING LOG 5762-00.GPJ GEOLABS.GDT 1/12/07

GEOLABS, INC.

Geotechnical Engineering

LIQUEFACTION ASSESSMENT
PIERS 1 AND 2 AT KAWAIHAE HARBOR
KAWAIHAE HARBOR, SOUTH KOHALA, HAWAII

Log of Boring **4**

Approximate Ground Surface Elevation (feet MSL): 8 *

Laboratory: Other Tests	Laboratory: Moisture Content (%)	Laboratory: Dry Density (pcf)	Field: Core Recovery (%)	Field: RQD (%)	Field: Penetration Resistance (blows/foot)	Field: Pocket Pen. (tsf)	Depth (feet)	USCS	Description
									5-inch **ASPHALTIC CONCRETE**
	11				57			SM	Tannish brown **SILTY SAND (CORALLINE)** with some gravel (coralline), very dense, moist
-#200 =14%	28				13		5	SM	Tan **SILTY SAND (CORALLINE)** with gravel (coralline), medium dense, very moist
	92				11		10	CL	Gray **SILTY CLAY** with traces of fine sand (coralline), soft
									Gray **COBBLES (BASALTIC)** with some dark brown silt, dense
			100	71	50/.3' Ref.		15		Gray **BOULDERS (BASALTIC)** with some dark brown silt, very dense
					4			ML	Dark grayish brown **SANDY SILT**, very soft
-#200 =48%	23				52		25	SM	Light gray **SILTY SAND (CORALLINE)**, dense
-#200 =6%	31				48		30	SP-SM	Gray **SAND (CORALLINE)** with traces of silt, dense
	40				58		35		grades with some gravel (coralline)

Date Started:	November 1, 2006	Water Level: ∇ 5 ft. 11/1/06 1332 HRS	Plate
Date Completed:	November 2, 2006		
Logged By:	E. Shinsato	Drill Rig: MOBILE B-53	A - 4.1
Total Depth:	70 feet	Drilling Method: Rotary Wash w/Bentonite & HQ Coring	
Work Order:	5762-00	Driving Energy: 140 lb. wt., 30 in. drop	

GEOLABS, INC.

Geotechnical Engineering

LIQUEFACTION ASSESSMENT
PIERS 1 AND 2 AT KAWAIHAE HARBOR
KAWAIHAE HARBOR, SOUTH KOHALA, HAWAII

Log of Boring 4

Laboratory			Field								
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
										SP-SM	(Continued from previous plate)
	62				17		40			SM	Dark gray **SILTY SAND (CORALLINE)** with some gravel and shell fragments, loose to medium dense
-#200 =12%	52				12		45				
	40				9		50			GP-GM	Gray **SANDY GRAVEL (CORALLINE)** with some silt, loose
-#200 =11%	48				21		55				grades to medium dense
	14				39		60			GW	Dark brown **SANDY GRAVEL (BASALTIC)** with some silt, dense (alluvium)
			78	0			65				Gray vesicular **BASALT**, severely fractured, moderately weathered, medium hard (basalt formation)
			100	93							grades to slightly fractured, slightly weathered, medium hard
							70				Boring terminated at 70 feet

Date Started:	November 1, 2006	Water Level: ∇ 5 ft. 11/1/06 1332 HRS	Plate
Date Completed:	November 2, 2006		
Logged By:	E. Shinsato	Drill Rig: MOBILE B-53	A - 4.2
Total Depth:	70 feet	Drilling Method: Rotary Wash w/Bentonite & HQ Coring	
Work Order:	5762-00	Driving Energy: 140 lb. wt., 30 in. drop	

BORING LOG 5762-00.GPJ GEOLABS.GDT 1/12/07

GEOLABS, INC.

Geotechnical Engineering

LIQUEFACTION ASSESSMENT
PIERS 1 AND 2 AT KAWAIHAE HARBOR
KAWAIHAE HARBOR, SOUTH KOHALA, HAWAII

Log of Boring

5

Approximate Ground Surface Elevation (feet MSL): 8 *

Laboratory: Other Tests	Laboratory: Moisture Content (%)	Laboratory: Dry Density (pcf)	Field: Core Recovery (%)	Field: RQD (%)	Field: Penetration Resistance (blows/foot)	Field: Pocket Pen. (tsf)	Depth (feet)	USCS	Description
									10-inch **ASPHALTIC CONCRETE**
								GM SP	Grayish brown **SILTY GRAVEL (BASALTIC)** with sand, dense, dry (base course)
	10				29				Tan fine grained **SAND (CORALLINE)** with some gravel (coralline), dense, damp
	15				23		5		grades to medium dense at 3 feet
	74				7	0	10	OL	Dark gray to black **SANDY SILT (ORGANIC)** with traces of sand (coralline) and roots, very soft
	60				19		15	SP-SM	Dark gray **SAND (CORALLINE)** with traces of silt and shells, medium dense
-#200 =8%	29				29		20		
	41				6	0.3	25	OH	Dark grayish brown **SILTY CLAY (ORGANIC)** with traces of sand (coralline) and roots, soft
								SM	Tan fine grained **SILTY SAND (CORALLINE)**, loose
	27				43		30	SP-SM	Gray fine to medium grained **SILTY SAND (CORALLINE)** with traces of shell fragments, dense
-#200 =2%	35				31		35	SP	Tan and dark gray medium grained **GRAVELLY SAND (CORALLINE)**, medium dense to dense

Date Started:	November 7, 2006	Water Level: ∇ 6.8 ft. 11/7/06 0914 HRS	
Date Completed:	November 7, 2006		
Logged By:	E. Shinsato	Drill Rig:	MOBILE B-53
Total Depth:	55.5 feet	Drilling Method:	Rotary Wash w/Bentonite & HQ Coring
Work Order:	5762-00	Driving Energy:	140 lb. wt., 30 in. drop

Plate

A - 5.1

BORING LOG 5762-00.GPJ GEOLABS.GDT 1/12/07

GEOLABS, INC. Geotechnical Engineering	LIQUEFACTION ASSESSMENT PIERS 1 AND 2 AT KAWAIHAE HARBOR KAWAIHAE HARBOR, SOUTH KOHALA, HAWAII	Log of Boring 5

Laboratory			Field								
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
										SP	(Continued from previous plate)
LL=51 PI=11	42				18		40			MH	Brown **CLAYEY SILT** with some highly weathered sub-rounded gravel and sand (basaltic), stiff (alluvium)
	36		39	0	53/.5' Ref.		45				Orange and grayish brown **BASALT**, severely fractured, moderately weathered, soft to medium hard (basalt formation)
			92	83			50				Gray vugular **BASALT**, slightly fractured, slightly weathered, hard (basalt formation)
			40	0			55				Grayish brown vesicular **BASALT**, severely fractured, moderately weathered, medium hard (basalt formation)
											Boring terminated at 55.5 feet
							60				
							65				
							70				

Date Started:	November 7, 2006	Water Level: ▽	6.8 ft. 11/7/06 0914 HRS	Plate
Date Completed:	November 7, 2006			
Logged By:	E. Shinsato	Drill Rig:	MOBILE B-53	A - 5.2
Total Depth:	55.5 feet	Drilling Method:	Rotary Wash w/Bentonite & HQ Coring	
Work Order:	5762-00	Driving Energy:	140 lb. wt., 30 in. drop	

GEOLABS, INC.

Geotechnical Engineering

LIQUEFACTION ASSESSMENT
PIERS 1 AND 2 AT KAWAIHAE HARBOR
KAWAIHAE HARBOR, SOUTH KOHALA, HAWAII

Log of Boring 6

Approximate Ground Surface Elevation (feet MSL): 8 *

Laboratory: Other Tests	Laboratory: Moisture Content (%)	Laboratory: Dry Density (pcf)	Field: Core Recovery (%)	Field: RQD (%)	Field: Penetration Resistance (blows/foot)	Field: Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
										GM	4-inch **ASPHALTIC CONCRETE**
	9				33					SP	Grayish brown **SILTY GRAVEL (BASALTIC)** with some cobbles and sand, medium dense
											Tan medium grained **SAND (CORALLINE)** with some gravel (coralline), dense
-#200 =15%	26				22		5			SM	Dark brown **SILTY SAND (CORALLINE)** with some gravel (coralline), medium dense
			12	12	20/.0' Ref.		10				Reddish gray **BOULDERS (BASALTIC)** with some cobbles and gravel, very dense
-#200 =8%	34				18		15			SP-SM	Grayish brown **GRAVELLY SAND (CORALLINE)** with traces of silt and shell fragments, medium dense
	39				7		20			SP-SM	Gray **GRAVELLY SAND (CORALLINE)** with some traces of silt and some shell fragments, loose
-#200 =9%	31				17		25				grades to medium dense
	32				12		30				
							35				

Date Started:	November 8, 2006	Water Level: ∇ 5.8 ft. 11/8/06 1245 HRS	
Date Completed:	November 8, 2006		
Logged By:	E. Shinsato	Drill Rig:	MOBILE B-53
Total Depth:	86.5 feet	Drilling Method:	Rotary Wash w/Bentonite & HQ Coring
Work Order:	5762-00	Driving Energy:	140 lb. wt., 30 in. drop

Plate A - 6.1

BORING LOG 5762-00.GPJ GEOLABS.GDT 1/12/07

GEOLABS, INC. Geotechnical Engineering	LIQUEFACTION ASSESSMENT PIERS 1 AND 2 AT KAWAIHAE HARBOR KAWAIHAE HARBOR, SOUTH KOHALA, HAWAII	Log of Boring 6

Laboratory			Field								
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
											(Continued from previous plate)
#200 =16%	34				14					SM	Gray fine to medium grained **SILTY SAND (CORALLINE)** with some gravel (coralline) and shells, loose to medium dense
							40				
	39				8						grades to loose
			72				45				Tannish white **CORAL**
	32				17					GM	Gray **SANDY GRAVEL (CORALLINE)** with some silt, medium dense (coralline detritus)
			0				50				
	36				21						Tan **CORAL**, severely fractured
			0							GM	Tan **SILTY GRAVEL (CORALLINE)** with sand, medium dense (coralline detritus)
							55				
	55				10					CL	Dark gray **SILTY CLAY** with some gravel (coralline), sand, and shells, soft
			0				60				
	42				23					GM	Dark gray **SILTY GRAVEL (CORALLINE)** with sand and some clay seams, medium dense
			50	100							
							65				Gray **BOULDERS (BASALTIC)** with some cobbles (clinker) grades with some sand (coralline) at 67 feet
			18	18							
							70				

Date Started:	November 8, 2006	Water Level: ▽ 5.8 ft. 11/8/06 1245 HRS	Plate
Date Completed:	November 8, 2006		
Logged By:	E. Shinsato	Drill Rig: MOBILE B-53	A - 6.2
Total Depth:	86.5 feet	Drilling Method: Rotary Wash w/Bentonite & HQ Coring	
Work Order:	5762-00	Driving Energy: 140 lb. wt., 30 in. drop	

GEOLABS, INC. Geotechnical Engineering	LIQUEFACTION ASSESSMENT PIERS 1 AND 2 AT KAWAIHAE HARBOR KAWAIHAE HARBOR, SOUTH KOHALA, HAWAII	Log of Boring 6

Laboratory			Field								
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
											(Continued from previous plate)
			53	32	25/.0' Ref.						
							75				Reddish gray **BASALT FRAGMENTS** (clinker)
			80	55							Gray vesicular **BASALT**, moderately fractured, moderately to slightly weathered, medium hard (basalt formation)
							80				
			40	0							Brown vesicular **BASALT**, severely fractured, highly weathered, soft rock
							85				
											Boring terminated at 86.5 feet
							90				
							95				
							100				
							105				

Date Started:	November 8, 2006	Water Level: ∇	5.8 ft. 11/8/06 1245 HRS	Plate
Date Completed:	November 8, 2006			
Logged By:	E. Shinsato	Drill Rig:	MOBILE B-53	A - 6.3
Total Depth:	86.5 feet	Drilling Method:	Rotary Wash w/Bentonite & HQ Coring	
Work Order:	5762-00	Driving Energy:	140 lb. wt., 30 in. drop	

GEOLABS, INC.

Geotechnical Engineering

LIQUEFACTION ASSESSMENT
PIERS 1 AND 2 AT KAWAIHAE HARBOR
KAWAIHAE HARBOR, SOUTH KOHALA, HAWAII

Log of Boring **7**

Approximate Ground Surface Elevation (feet MSL): 8 *

Laboratory: Other Tests	Laboratory: Moisture Content (%)	Laboratory: Dry Density (pcf)	Field: Core Recovery (%)	Field: RQD (%)	Field: Penetration Resistance (blows/foot)	Field: Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
	6				37					SP	Tan fine to medium grained **SAND (CORALLINE)** with some gravel (coralline), medium dense to dense
	10				26		5				grades to medium dense
	14				36		10			SM	Tannish brown **SILTY SAND (CORALLINE)** with some gravel (coralline), dense
	24				36		15			GM	Light gray **SILTY GRAVEL (CORALLINE)** with some sand (coralline) and shells, dense
										SM	Dark grayish brown **SANDY SILT** with some gravel (basaltic), medium dense
	36				23		20			ML	Dark grayish brown **SANDY SILT** with some gravel (basaltic), medium dense
-#200 =6%	33				41		25			SP-SM	Tannish gray fine grained **SAND (CORALLINE)** with traces of silt, dense
	29				36		30			SW	Tannish gray well-graded **SILTY SAND (CORALLINE)** with some gravel (coralline) and shells, dense
	51				21		35			GP-GM	Gray **SANDY GRAVEL (CORALLINE)** with some silt and shells, medium dense

Date Started:	November 2, 2006	Water Level: ∇ 12.8 ft. 11/2/06 1341 HRS	
Date Completed:	November 3, 2006		
Logged By:	E. Shinsato	Drill Rig:	MOBILE B-53
Total Depth:	65 feet	Drilling Method:	Rotary Wash w/Bentonite & HQ Coring
Work Order:	5762-00	Driving Energy:	140 lb. wt., 30 in. drop

Plate A - 7.1

BORING LOG 5762-00.GPJ GEOLABS.GDT 1/12/07

GEOLABS, INC.

Geotechnical Engineering

LIQUEFACTION ASSESSMENT
PIERS 1 AND 2 AT KAWAIHAE HARBOR
KAWAIHAE HARBOR, SOUTH KOHALA, HAWAII

Log of Boring 7

Laboratory			Field								
Other Tests	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	RQD (%)	Penetration Resistance (blows/foot)	Pocket Pen. (tsf)	Depth (feet)	Sample	Graphic	USCS	Description
											(Continued from previous plate)
										GP-GM	
-#200 =10%	40				28		40				
										GW	Dark grayish brown well-graded **SANDY GRAVEL (BASALTIC)** with some silt, medium dense to dense
	30				32		45				
										CH	Brown **SILTY CLAY** with some moderately weathered gravel (basaltic), stiff
											Grayish brown **BASALT FRAGMENTS**, medium dense to dense (clinker)
	35				38		50				Orange brown and gray **BASALT**, severely fractured, highly weathered, soft rock
			75	0							grades to moderately weathered, medium hard
							55				
			37	0							Reddish gray **BASALT FRAGMENTS**, medium dense (clinker)
							60				Gray vesicular **BASALT**, severely fractured, moderately weathered, medium hard (basalt formation)
			100	71							grades to moderately fractured
							65				Boring terminated at 65 feet
							70				

Date Started:	November 2, 2006	Water Level: ▽ 12.8 ft. 11/2/06 1341 HRS	Plate
Date Completed:	November 3, 2006		
Logged By:	E. Shinsato	Drill Rig: MOBILE B-53	A - 7.2
Total Depth:	65 feet	Drilling Method: Rotary Wash w/Bentonite & HQ Coring	
Work Order:	5762-00	Driving Energy: 140 lb. wt., 30 in. drop	

BORING LOG 5762-00.GPJ GEOLABS.GDT 1/12/07

APPENDIX B

Laboratory Testing

APPENDIX B

Laboratory Testing

Moisture Content (ASTM D 2216) and Unit Weight (ASTM D 2937) determinations were performed on selected soil samples as an aid in the classification and evaluation of soil properties. The test results are presented on the Logs of Borings at the appropriate sample depths.

One Atterberg Limits test (ASTM D 4318) was performed on a selected soil sample to evaluate the liquid and plastic limits and to aid in soil classification. The test results are summarized on the Logs of Borings at the appropriate sample depth, and graphic presentation of the test results is provided on Plate B-1.

Twenty Sieve Analysis tests (ASTM C 117 & C 136) were performed on selected soil samples to evaluate the gradation characteristics of the soils and to aid in soil classification. The fines content of the sample tested (percent particles passing the No. 200 sieve) is indicated on the Logs of Borings at the appropriate sample depths. One of the samples tested was subjected to a hydrometer analysis (ASTM D 422). Graphic presentations of the grain size distribution (including the hydrometer analyses) are provided on Plates B-2 through B-5.

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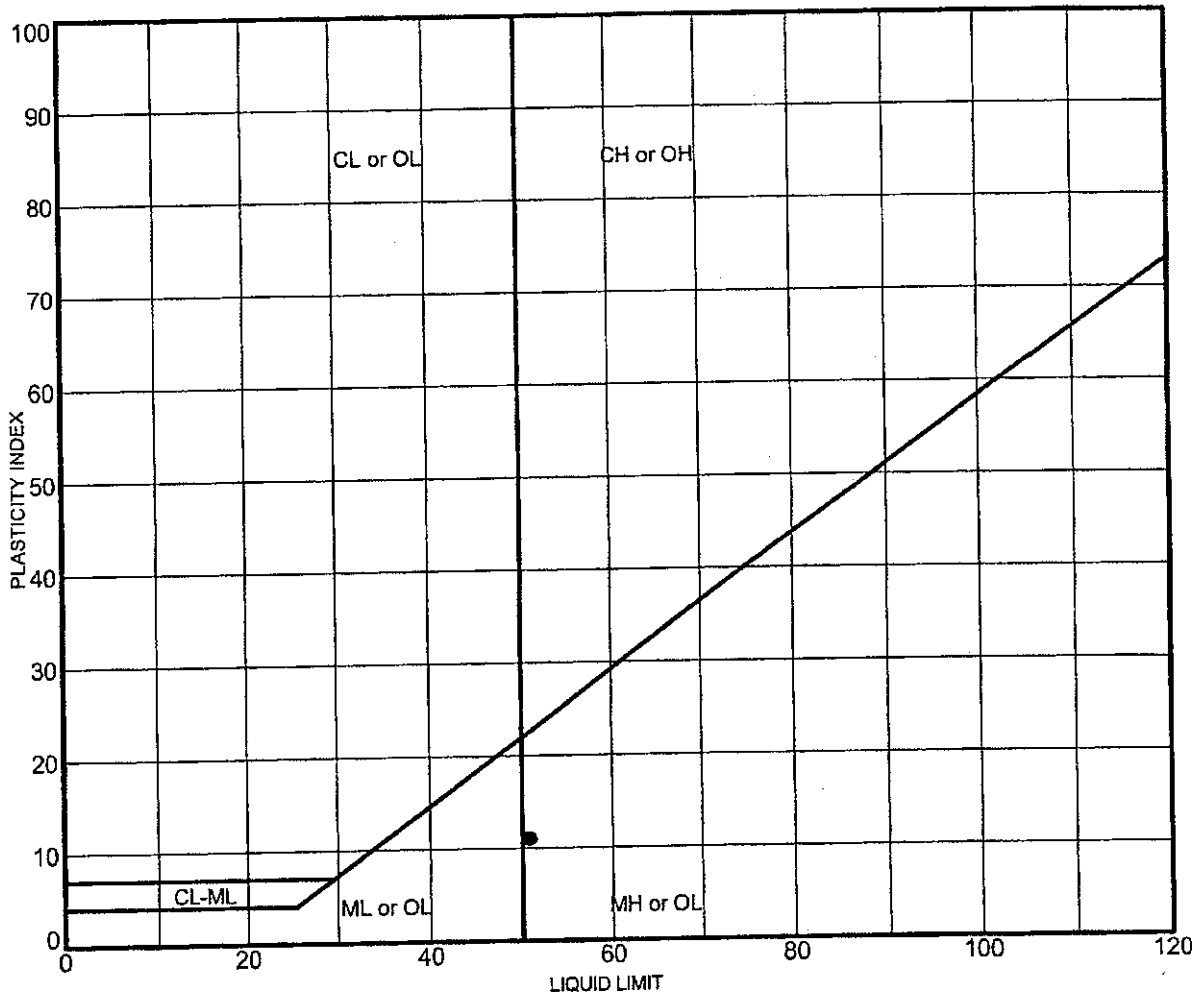

	Sample	Depth (ft)	LL	PL	PI	Description
●	B-5	39.0-40.5	51	40	11	Brown clayey silt (MH)

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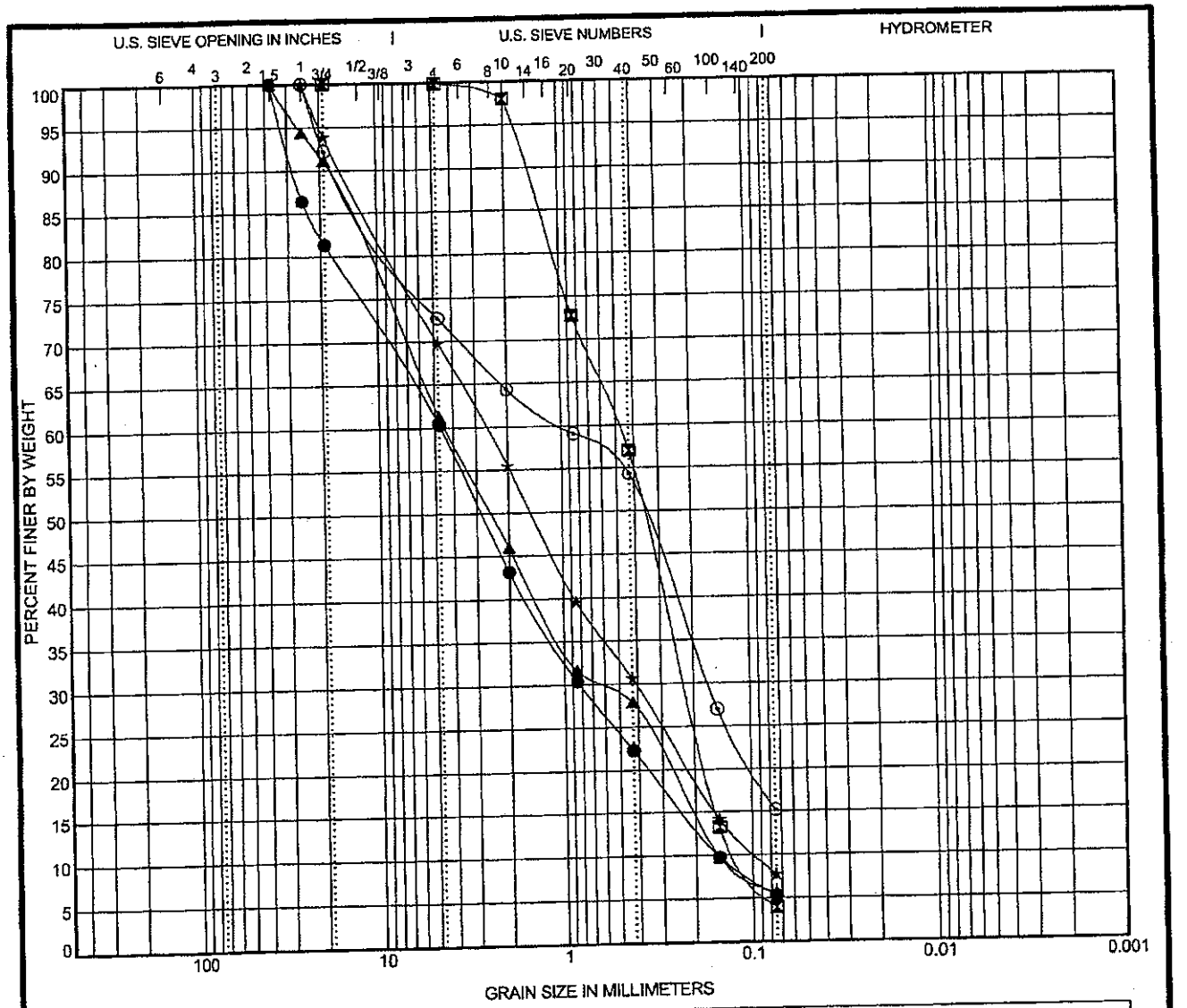
ATTERBERG LIMITS TEST RESULTS - ASTM D 4318

LIQUEFACTION ASSESSMENT
PIERS 1 AND 2 AT KAWAIHAE HARBOR
KAWAIHAE HARBOR, SOUTH KOHALA, HAWAII

Plate
B - 1

G ATTERBERG 5762-00.GPJ GEOLABS.GDT 1/12/07



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	B-1	14.5-16.0	Grayish brown gravelly sand (SP-SM) with traces of silt				0.9	30.4
⊠	B-1	24.5-26.0	Gray sand (SP)				0.9	4.1
▲	B-1	39.5-41.0	Gray gravelly sand (SP-SM) with traces of silt				0.6	28.9
★	B-2	10.0-11.5	Gray gravelly sand (SP-SM) with traces of silt				0.7	27.5
⊙	B-2	24.5-26.0	Gray silty sand (SM) with gravel					

	Sample	Depth (ft)	D100 (mm)	D60 (mm)	D30 (mm)	D10 (mm)	%Gravel	%Sand	%Fine
●	B-1	14.5-16.0	37.5	4.66	0.821	0.153	39.6	55.1	5.3
⊠	B-1	24.5-26.0	19	0.485	0.223	0.117	0.1	95.9	4.0
▲	B-1	39.5-41.0	37.5	4.404	0.626	0.152	38.7	55.6	5.7
★	B-2	10.0-11.5	25	2.625	0.408	0.095	30.2	62.1	7.8
⊙	B-2	24.5-26.0	25	0.985	0.168		27.3	57.5	15.2

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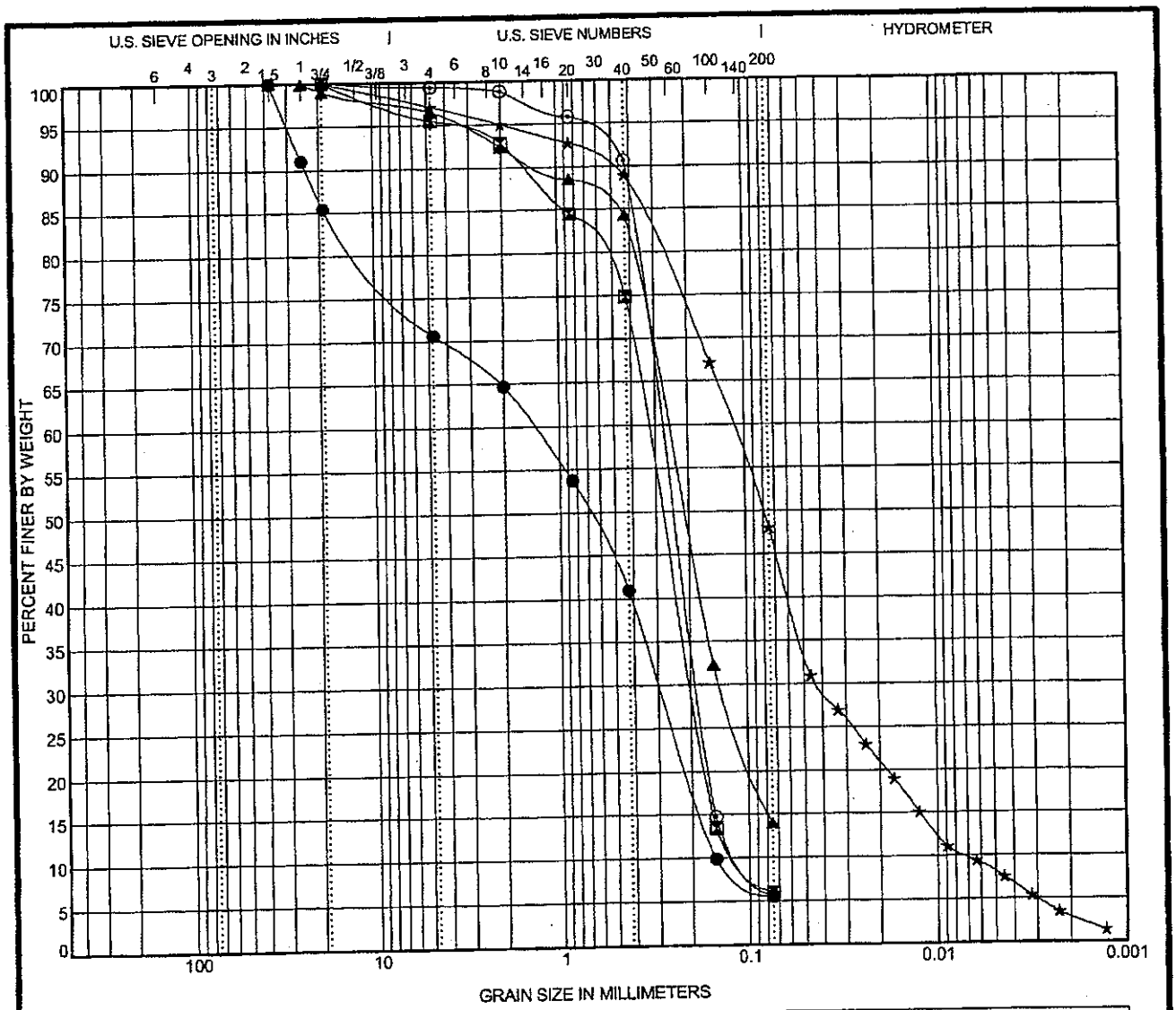
GRAIN SIZE DISTRIBUTION - ASTM C 117 & C 136

LIQUEFACTION ASSESSMENT
PIERS 1 AND 2 AT KAWAIHAE HARBOR
KAWAIHAE HARBOR, SOUTH KOHALA, HAWAII

Plate
B - 2

G GRAIN SIZE 5762-00.GPJ GEOLABS.GDT 1/12/07



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	B-3	19.0-20.5	Grayish brown gravelly sand (SP-SM) w/ traces of silt				0.4	9.2
⊠	B-3	29.0-30.5	Gray sand (SP-SM) with traces of silt				1.1	3.0
▲	B-4	5.0-6.5	Tan silty sand (SM)					
★	B-4	23.5-25.0	Dark brownish gray silty sand (SM)				2.2	16.3
⊙	B-4	28.5-30.0	Gray sand (SP-SM) with traces of silt				1.2	2.7

	Sample	Depth (ft)	D100 (mm)	D60 (mm)	D30 (mm)	D10 (mm)	%Gravel	%Sand	%Fine
●	B-3	19.0-20.5	37.5	1.387	0.294	0.151	29.3	65.2	5.5
⊠	B-3	29.0-30.5	19	0.33	0.199	0.109	4.5	89.5	6.0
▲	B-4	5.0-6.5	37.5	0.261	0.138		3.5	82.6	14.0
★	B-4	23.5-25.0	19	0.116	0.043	0.007	2.8	49.1	48.1
⊙	B-4	28.5-30.0	19	0.279	0.185	0.105	0.6	93.9	5.5

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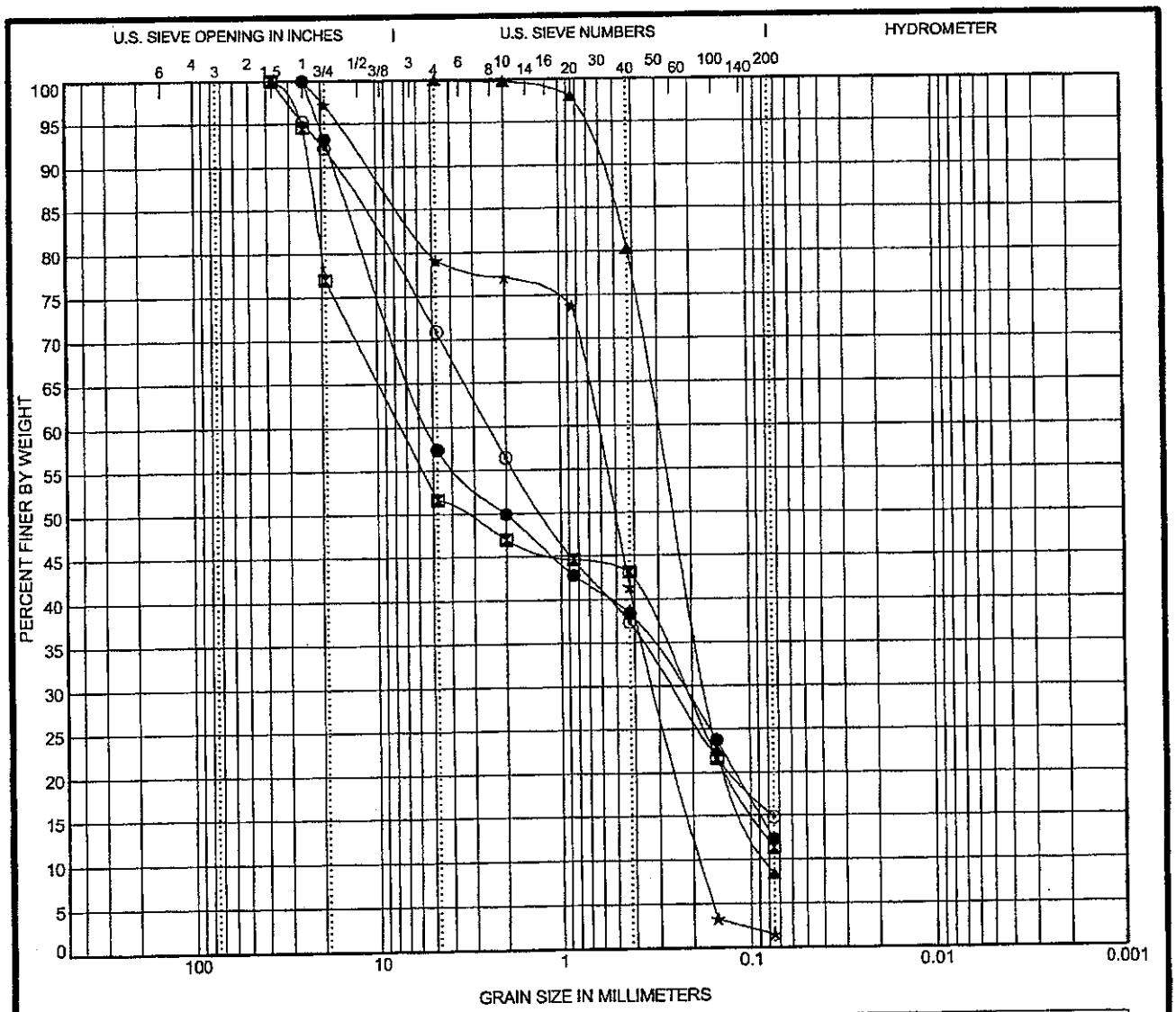
GRAIN SIZE DISTRIBUTION - ASTM C 117 & C 136

LIQUEFACTION ASSESSMENT
PIERS 1 AND 2 AT KAWAIHAE HARBOR
KAWAIHAE HARBOR, SOUTH KOHALA, HAWAII

Plate
B - 3

G GRAIN SIZE 5762-00.GPJ GEOLABS.GDT 1/12/07



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	B-4	43.5-45.0	Dark gray silty sand (SM) with gravel				0.2	80.6
⊠	B-4	53.5-55.0	Gray sandy gravel (GP-GM) with some silt				0.1	109.7
▲	B-5	19.0-20.5	Dark gray sand (SP-SM) with traces of silt				1.2	3.6
★	B-5	34.0-35.5	Tan and dark gray mottled gravelly sand (SP)				0.8	3.5
⊙	B-6	5.0-6.5	Dark brown silty sand (SM) with some gravel					

	Sample	Depth (ft)	D100 (mm)	D60 (mm)	D30 (mm)	D10 (mm)	%Gravel	%Sand	%Fine
●	B-4	43.5-45.0	25	5.255	0.234		42.6	45.1	12.3
⊠	B-4	53.5-55.0	37.5	7.512	0.225		48.4	40.3	11.3
▲	B-5	19.0-20.5	4.75	0.295	0.172	0.081	0.0	91.6	8.4
★	B-5	34.0-35.5	25	0.633	0.311	0.18	20.9	77.6	1.5
⊙	B-6	5.0-6.5	37.5	2.476	0.26		29.1	56.1	14.8

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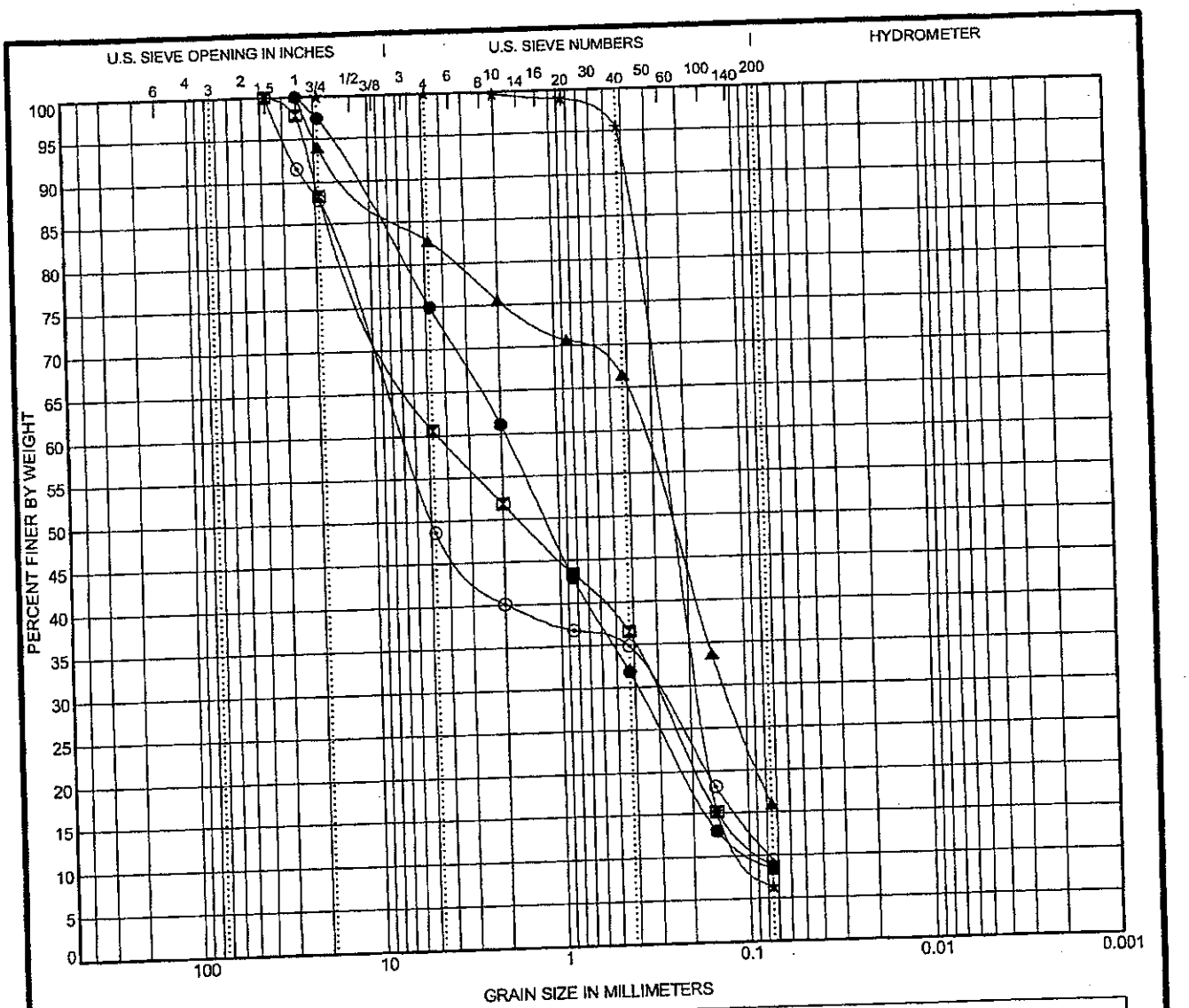
W.O. 5762-00

GRAIN SIZE DISTRIBUTION - ASTM C 117 & C 136

LIQUEFACTION ASSESSMENT
PIERS 1 AND 2 AT KAWAIHAE HARBOR
KAWAIHAE HARBOR, SOUTH KOHALA, HAWAII

Plate
B - 4

G GRAIN SIZE 5762-00.GPJ GEOLABS.GDT 1/12/07



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	B-6	16.5-18.0	Grayish brown gravelly sand (SP-SM) with traces of silt				0.8	19.8
⊠	B-6	26.5-28.0	Gray gravelly sand (SP-SM) with traces of silt				0.2	51.6
▲	B-6	36.5-38.0	Gray silty sand (SM) with gravel					
★	B-7	24.5-26.0	Tannish gray sand (SP-SM) with traces of silt				1.2	2.7
⊙	B-7	39.5-41.0	Gray sandy gravel (GP-GM) with some silt				0.2	90.6

	Sample	Depth (ft)	D100 (mm)	D60 (mm)	D30 (mm)	D10 (mm)	%Gravel	%Sand	%Fine
●	B-6	16.5-18.0	25	1.898	0.385	0.096	25.0	66.6	8.4
⊠	B-6	26.5-28.0	37.5	4.48	0.309	0.087	39.4	52.0	8.6
▲	B-6	36.5-38.0	25	0.346	0.13		17.2	66.8	16.0
★	B-7	24.5-26.0	19	0.268	0.182	0.1	0.0	93.6	6.4
⊙	B-7	39.5-41.0	37.5	7.095	0.314	0.078	51.3	39.2	9.5

GRAIN SIZE DISTRIBUTION - ASTM C 117 & C 136

GEOLABS, INC.
GEOTECHNICAL ENGINEERING

W.O. 5762-00

LIQUEFACTION ASSESSMENT
PIERS 1 AND 2 AT KAWAIHAE HARBOR
KAWAIHAE HARBOR, SOUTH KOHALA, HAWAII

Plate
B - 5

G GRAIN SIZE 5762-00.GPJ GEOLABS.GDT 1/12/07

Appendix C6
Diving and Side Scan Sonar Survey Report
(Sea Engineering, Inc., September 2006)

Kawaihae Harbor Pier 1
Diving and Side Scan Sonar Survey Report

September, 2006

Prepared for:

ControlPoint Surveying, Inc.
1150 South King Street, Suite 102
Honolulu, Hawaii 96814

Submitted by:

Sea Engineering, Inc.
Makai Research Pier
Waimanalo, Hawaii 96795

#06-19

KAWAIHAE HARBOR PIER 1
DIVING AND SIDE SCAN SONAR SURVEY REPORT

INTRODUCTION

On August 3 and 4, 2006, Sea Engineering, Inc. conducted a multi-beam hydrographic survey, a side scan sonar survey and a reconnaissance dive of the area fronting the location of Pier 1, in Kawaihae Harbor. The objective of the survey was to investigate existing site conditions that may affect new ferry terminal operations. Charts showing soundings from the multi-beam survey are submitted under different cover. This report explains the findings of the side scan sonar and reconnaissance diving survey.

METHODS

A C-Max 800 side scan system with an operating frequency of 325 kHz and a slant range of 50 meters was used to image the project area. The sonar was integrated with differential GPS with sub-meter accuracy using Hypack navigation software. Multiple overlapping lines were run to ensure adequate bottom imagery.

Side scan sonar is useful in low visibility environments for finding large debris items, rock outcrops, and for determining general bottom conditions. The sonar is usually mounted on a towfish, and towed behind the survey boat at some depth in the water column. The instrument measures the backscatter intensity of acoustic pulses emitted from the towfish. The amount of sound reflected depends on the material properties of the bottom or the objects on the bottom. For example, hard metallic objects will reflect a greater percentage of acoustic energy, and will have a darker return than harbor muds. Objects that are elevated above the bottom will throw an acoustic shadow that will appear as a white streak on the sonar records.

Side scan images were processed using SonarWeb processing software and plotted with project bathymetry using AutoCad.

After field data acquisition, side scan data were reviewed for targets that should be further investigated for diving. Most targets were identified as boulders or small debris. One large target was found in 40 ft of water depth that was further investigated during the dive surveys.

A dive survey was also conducted by two SEI divers on SCUBA. The dive survey route began at the target found at 40 ft, and continued to the concrete sheet pile wharf edge. The divers turned east at the wharf edge to investigate shallow water conditions on the east side

of the wharf, and then continued to across the entire pier length to the small boat harbor breakwater.

RESULTS

Digitized and geo-rectified sonar images were overlaid to form an image mosaic, shown in Figure 1. The route of the dive survey is shown as a heavy dashed line, and approximate photograph locations are keyed as P1, P2, etc… The bottom at the site is mostly rocky, and is littered with numerous boulders and various types of debris, including numerous tires, concrete slabs (relict sheet piles), bundles of steel rebar and other steel debris. The largest debris item found is a 6 ft x 6 ft steel bucket or chute (positive identification was not made) lying in 40 ft of water depth (Figures 2 and 3 – Photographs P1 and P2).

Of particular note for the project are two shallow areas at either end of Pier 1. Both areas are formed from piles of basalt boulders. Figure 4 (Photograph P3) shows the base of the eastern boulder field. Figure 5 (Photograph P4) shows basalt boulders near the top of the pile, adjacent to a steel sheet pile wall (in the background of the photograph). The steel sheet pile is filled with concrete and acts as a toe for the shoreline CRM (cement rubble masonry) wall. The boulder pile is a massive structure outlined in Figure 1.

Isolated boulders also occur along the bottom between the ends of the pier (Figure 6, Photograph P5). At the western end of the pier, another pile of basalt boulders rest at a steep angle. Figure 7 (Photograph P6) shows the boulders up against the concrete sheet pile at the base of the pier. Figure 8, (Photograph P8) is at roughly the same location, but is a view looking west at the steeply placed boulders. The boulders are typically 2 to 4 feet in nominal diameter, however some are quite massive, on the order of 6 feet in diameter.

Between the edge of the pier and the beginning of the small boat harbor breakwater, there are steep drop-offs and ledges, as well as isolated boulders.

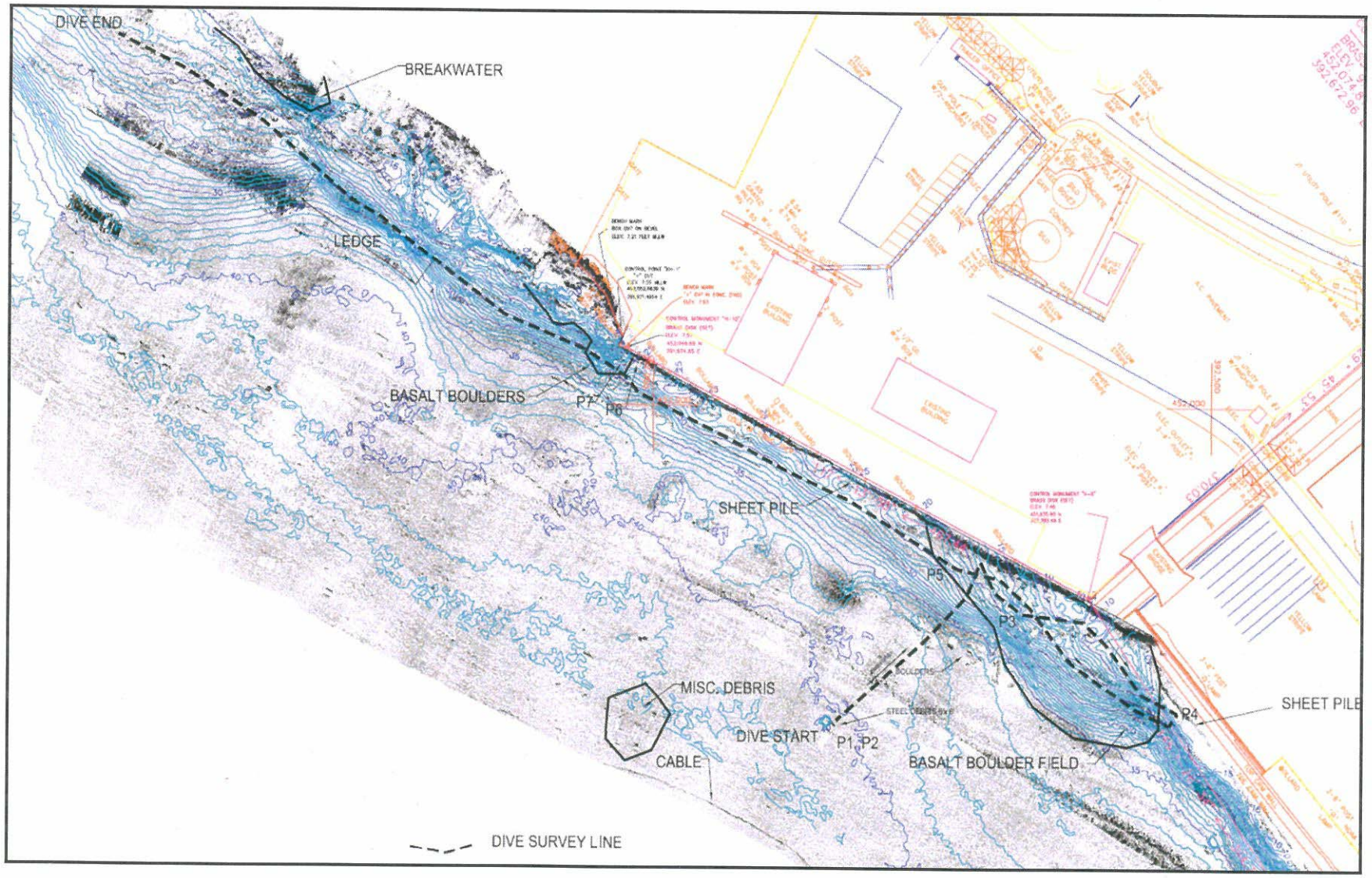


Figure 1. Results of mult-beam bathymetry, side scan sonar, and diving surveys.

Figure 2. Photograph P1: 6 ft x 6 ft steel debris

Figure 3. Photograph P2: 6 ft x 6 ft steel debris

Figure 4. Photograph P3: base of boulder pile near pier

Figure 5. Photograph P4: top of boulder pile near steel sheet pile

Figure 6. Photograph P5: isolated basalt boulder

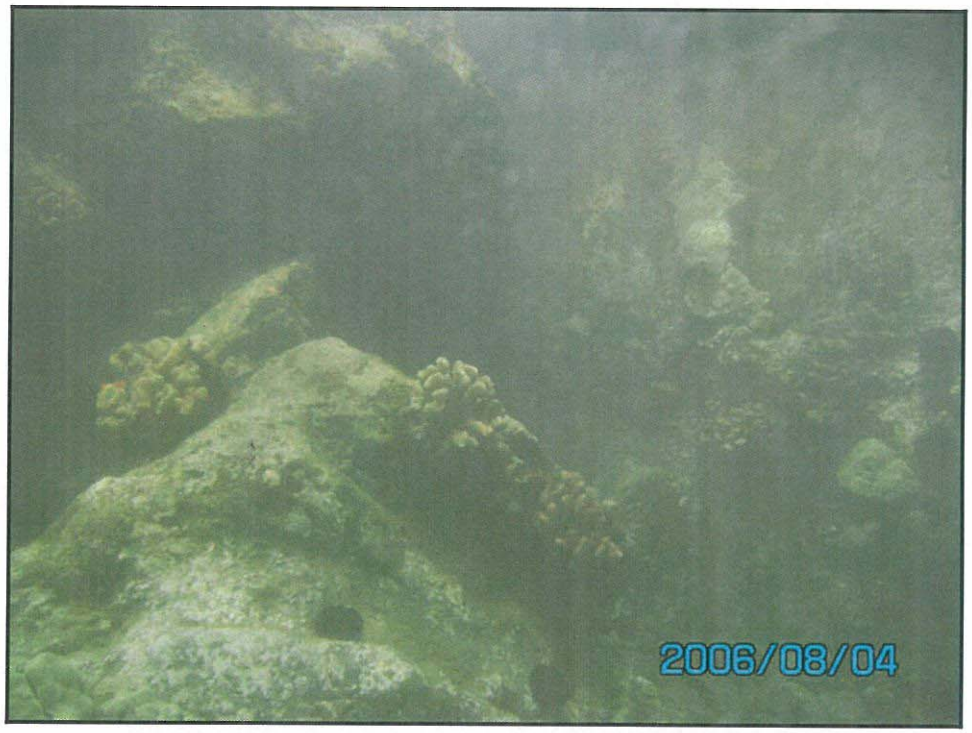
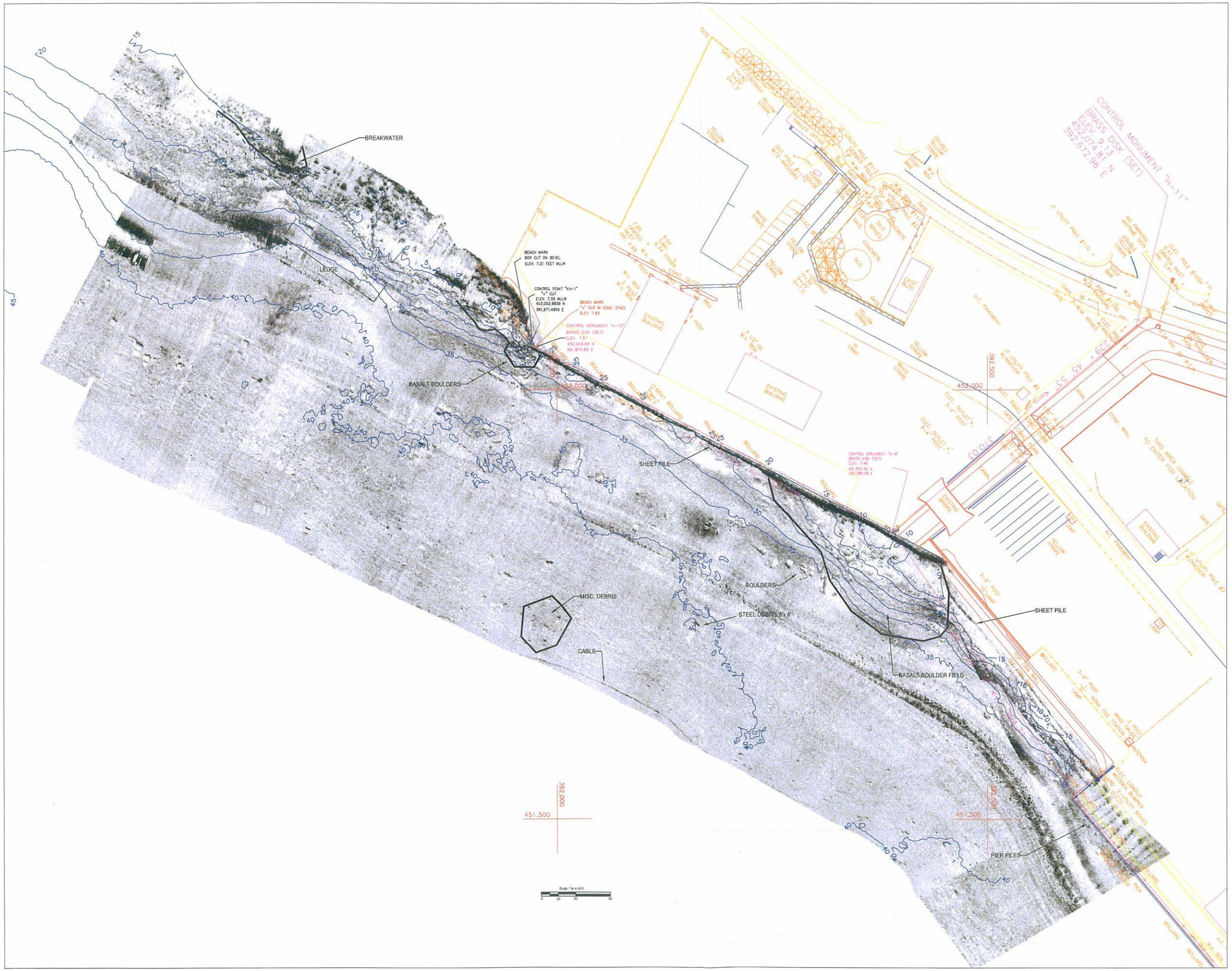


Figure 7. Photograph P6: basalt boulders against concrete sheet pile, west end of pier

Figure 8. Photograph P7: pile of boulders and debris, west end of pier



Sea Engineering, Inc.

MAKAI RESEARCH PIER
WAIMANALO, HI 96795
(808) 259-7966/FAX (808) 259-8143

KEYNOTES

GENERAL NOTES:

1. Vertical Datum: MLLW

2. Horizontal Control in Feet
"+" Cut "KH-1"
452052.8829 N
391971.4954 E

3. Vertical Control
Box Cut on Pier Bevel
Elevation: 7.21 ft MLLW

4. Contour Interval: 5 ft

MULTI-BEAM SURVEY NOTES:

1. Multi-Beam Survey conducted on 8-3-06 by Sea Engineering, Inc., using an Odom Echoscan Multi-Beam and Leica C. G. Beacon DGPS

2. Multi-Beam Survey conducted in accord with USACE Manual EM 1110-2-1003 Hydrographic Surveying

3. Data processing includes:
   a. Statistical removal of data outliers
   b. Data sort using average data value in 3 ft x 3 ft cell
   c. Final data display is min. depth over 10 ft radius

SIDE SCAN SONAR SURVEY NOTES:

1. Multi-Beam Survey conducted on 8-4-06 by Sea Engineering, Inc., using a C-Max CM800 side scan sonar at 325 kHz

Kawaihae Pier 1 Hydrographic Survey

Side Scan Sonar Survey

Kawaihae Harbor, Hawaii

August 2006

Appendix D
Seismicity and Liquefaction Supplement

Appendix D1 Supplemental Site Seismicity Figures.....
Figure D1-1 Comparison of Youngs et al. (1997) Attenuation Relationship for Soil and M 6.7 Kiholo Bay Earthquake Strong Motion Data and NEHRP Site Class..... D1-1
Figure D1-2 Comparison of Munson and Thurber (1997) Attenuation Relationship for Ash and M 6.7 Kiholo Bay Earthquake Strong Motion Data..... D1-2
Figure D1-3 Comparison of Abrahamson and Silva (1997) Attenuation Relationship for Soil and M 6.7 Kiholo Bay Earthquake Strong Motion Data..... D1-3
Figure D1-4 Comparison of Youngs et al. (1997) Attenuation Relationship for Soil and M 6.0 Mahukona Earthquake Strong Motion Data..... D1-4
Figure D1-5 Comparison of Abrahamson and Silva (2007) Attenuation Relationship for Soil and M 6.0 Mahukona Earthquake Strong Motion Data D1-5
Figure D1-6 Comparison of Boore and Atkinson (2006) Attenuation Relationship for Soil and M 6.0 Mahukona Earthquake Strong Motion Data..... D1-6
Figure D1-7 Comparison of Munson and Thurber (1997) Attenuation Relationship for Ash and M 6.0 Mahukona Earthquake Strong Motion Data D1-7
Figure D1-8 Comparison of Youngs et al. (1997) Attenuation Relationship for Soil and M 5.0 Puako Earthquake Strong Motion Data..... D1-8
Figure D1-9 Comparison of Munson and Thurber (1997) Attenuation Relationship for Ash and M 5.0 Puako Earthquake Strong Motion Data..... D1-9

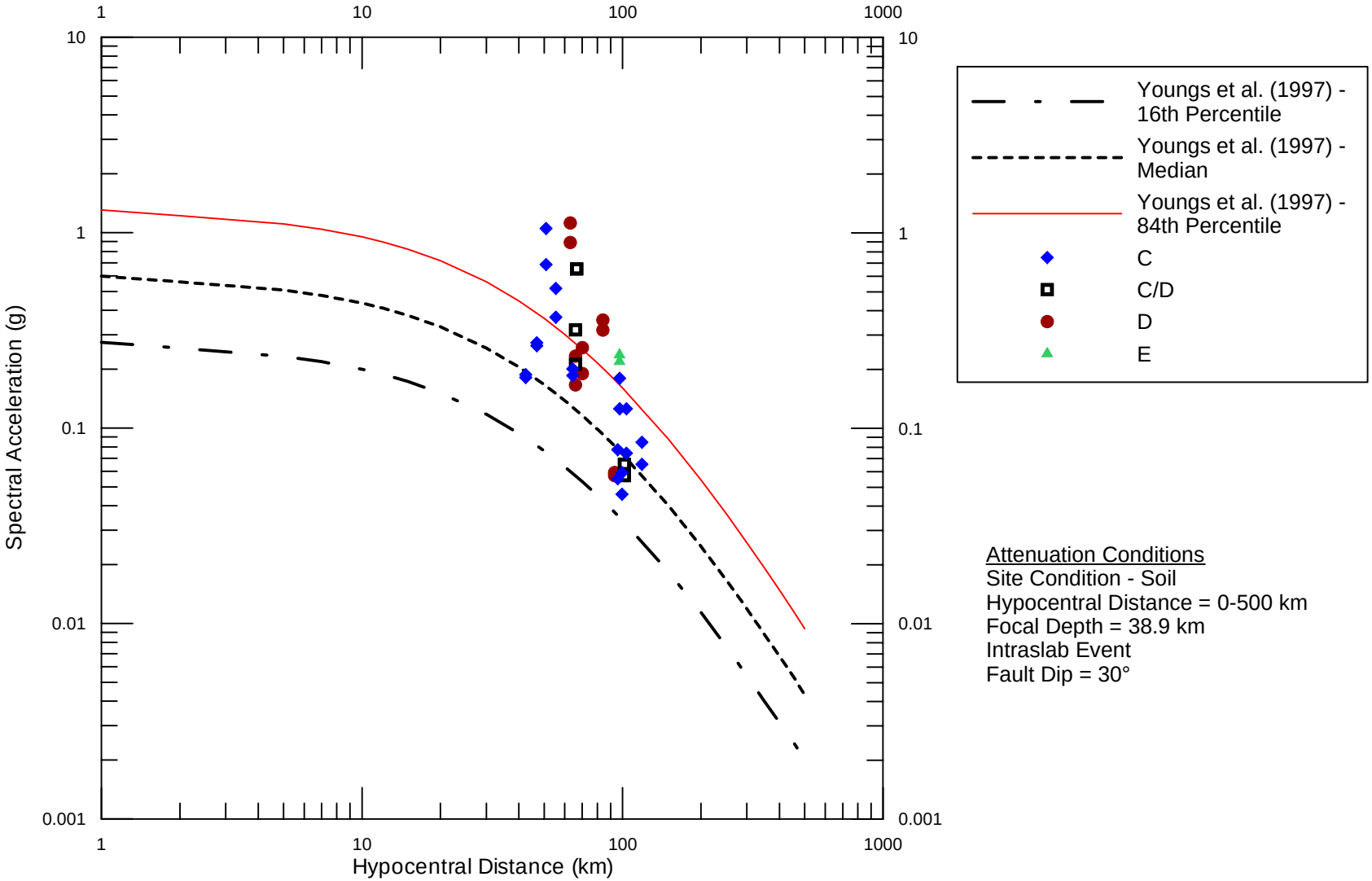
Appendix D2 Shear-Wave Velocity Profiling of Kawaihae Harbor and the USGS Strong Motion Stations .....

Appendix D3 Primer on Coralline Liquefaction.....

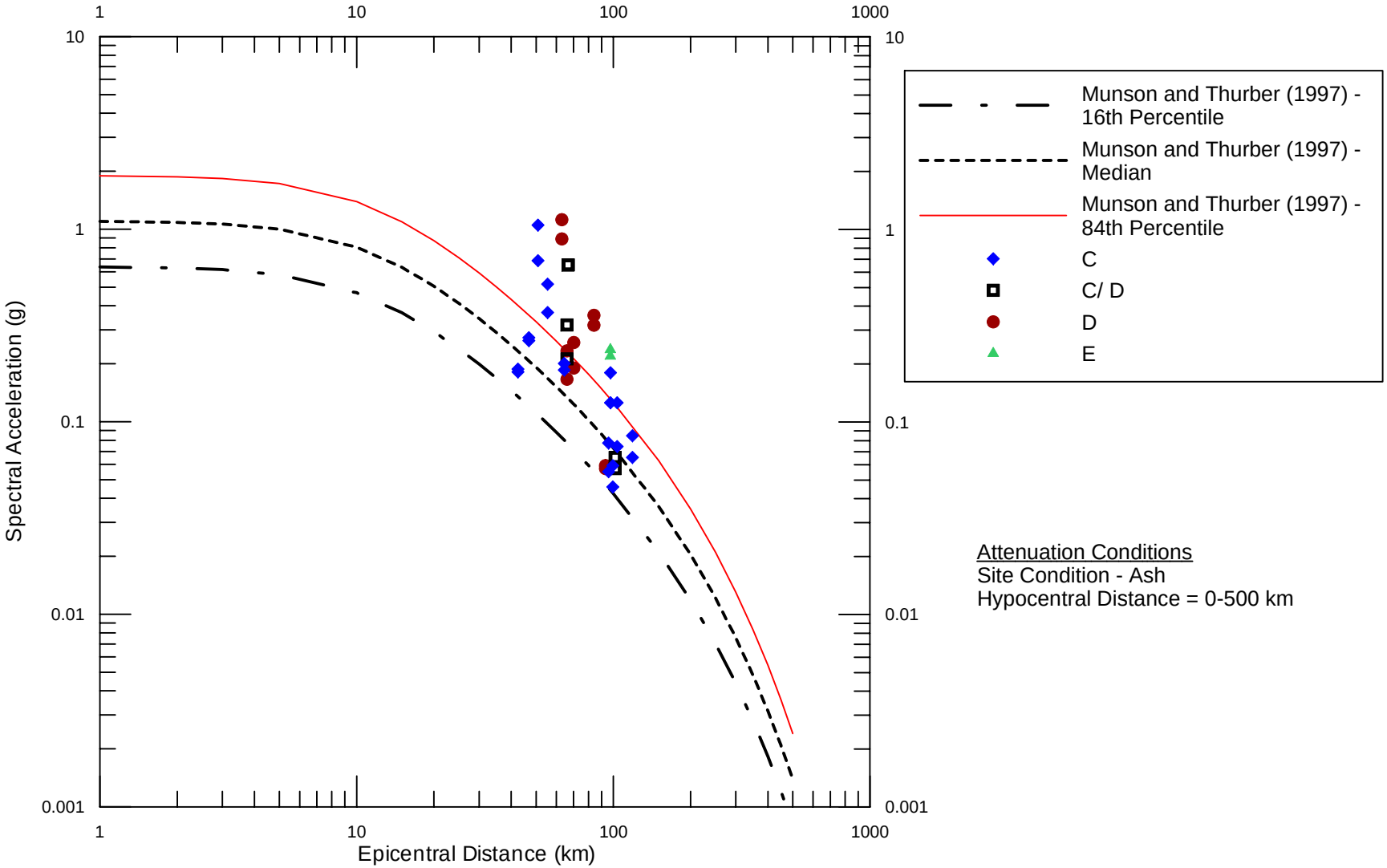
Appendix D4 Coralline Liquefaction Tables and Figures .....
Table D4-1 Summary of Liquefaction Settlement..... D4-1
Table D4-2 Lateral Spread Calculation (Kawaihae)..... D4-2

Figure D4-1 Stages of Reef Development and Classification of Coral Material..... D4-3
Figure D4-2 Pile Test Data in Coral Materials..... D4-4
Figure D4-3 Cementation Study of Coral..... D4-5
Figure D4-5 Liquefaction Settlement Calculations for B-18 (1988)..... D4-6
Figure D4-5 Liquefaction Settlement Calculations for B-28 (1988)..... D4-7
Figure D4-6 Liquefaction Settlement Calculations for B-35 (1989)..... D4-8
Figure D4-7 Liquefaction Settlement Calculations for B-36 (1989)..... D4-9
Figure D4-8 Liquefaction Settlement Calculations for B-A (1990)..... D4-10
Figure D4-9 Liquefaction Settlement Calculations for B-B (1990)..... D4-11
Figure D4-10 Liquefaction Settlement Calculations for B-E (1990) D4-12
Figure D4-11 Liquefaction Settlement Calculations for B-F (1990) D4-13
Figure D4-12 Liquefaction Settlement Calculations for B-8 (1999)..... D4-14
Figure D4-13 Liquefaction of Coral Sands D4-15
Figure D4-14 Comparison of Coral and Silica Sands D4-16
Figure D4-15 Blast Liquefaction Test at Maui Site D4-17
Figure D4-16 Shear Wave Data from Maui Blast Site..... D4-18

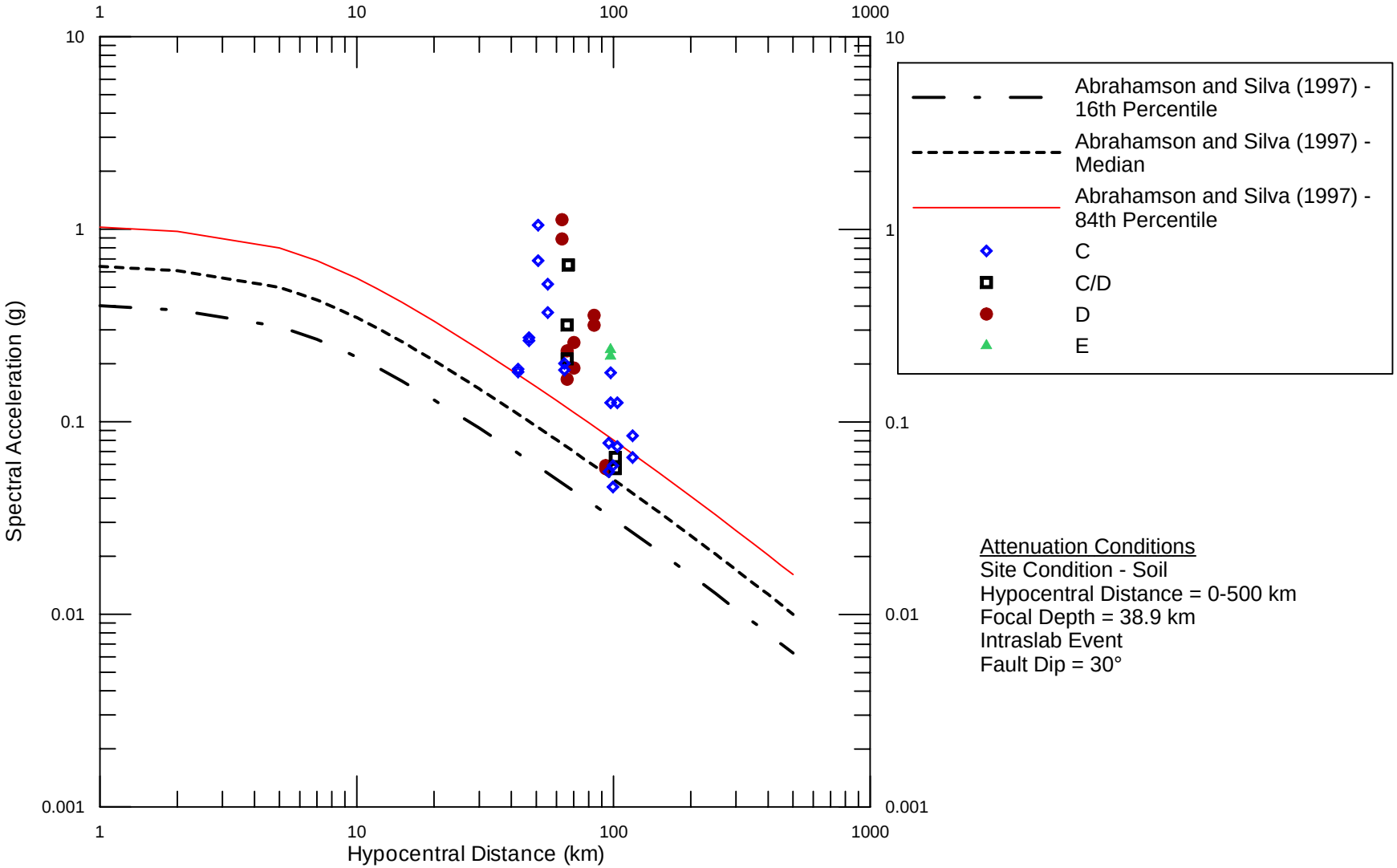
Appendix D1
Supplemental Site Seismicity Figures



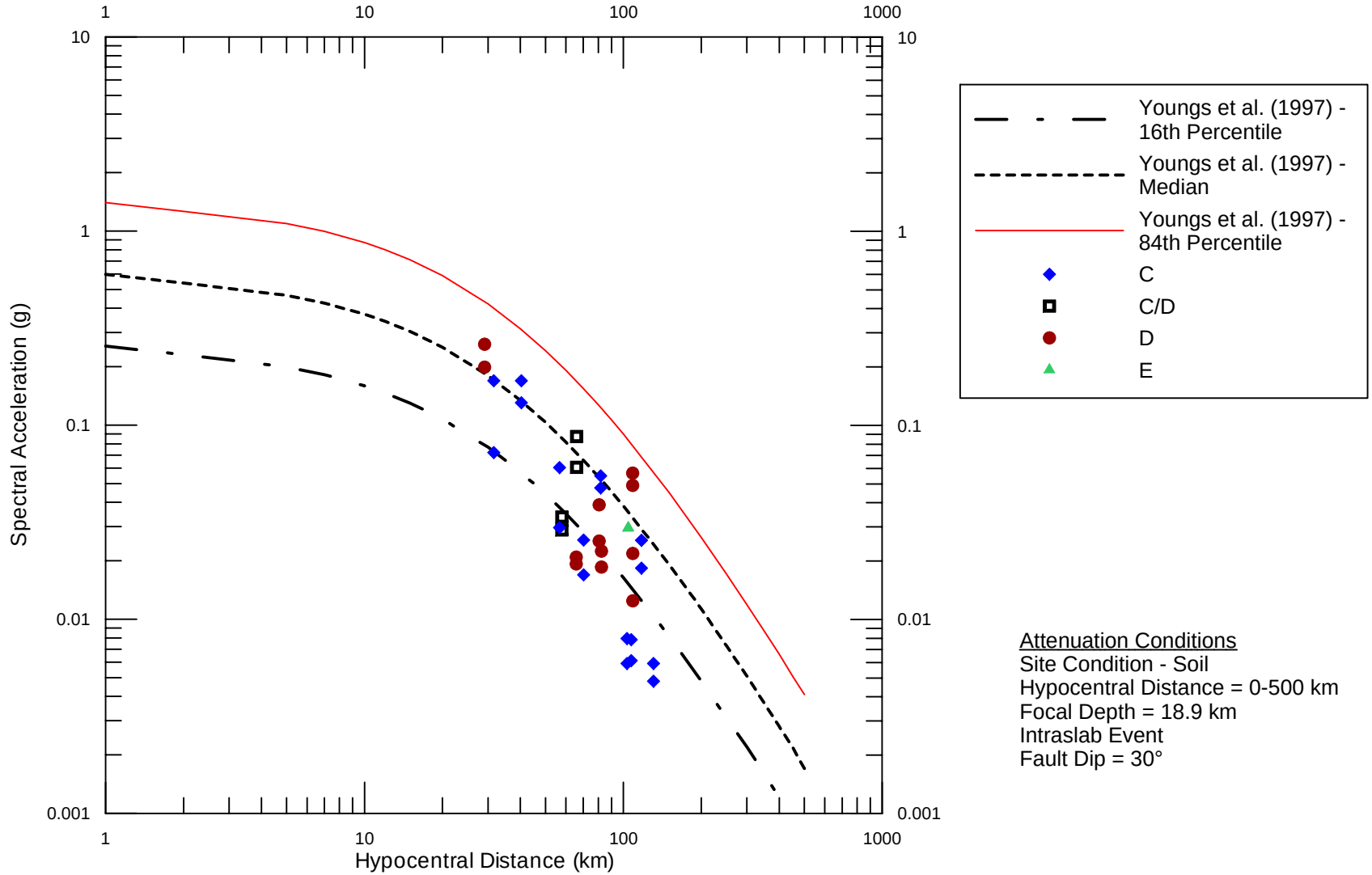
URS	Project No. 15298972 Kawaihae Harbor Hawaii	COMPARISON OF YOUNGS ET AL. (1997) ATTENUATION RELATIONSHIP FOR SOIL AND **M** 6.7 KIHOLO BAY EARTHQUAKE STRONG MOTION DATA AND NEHRP SITE CLASS	Figure D1-1



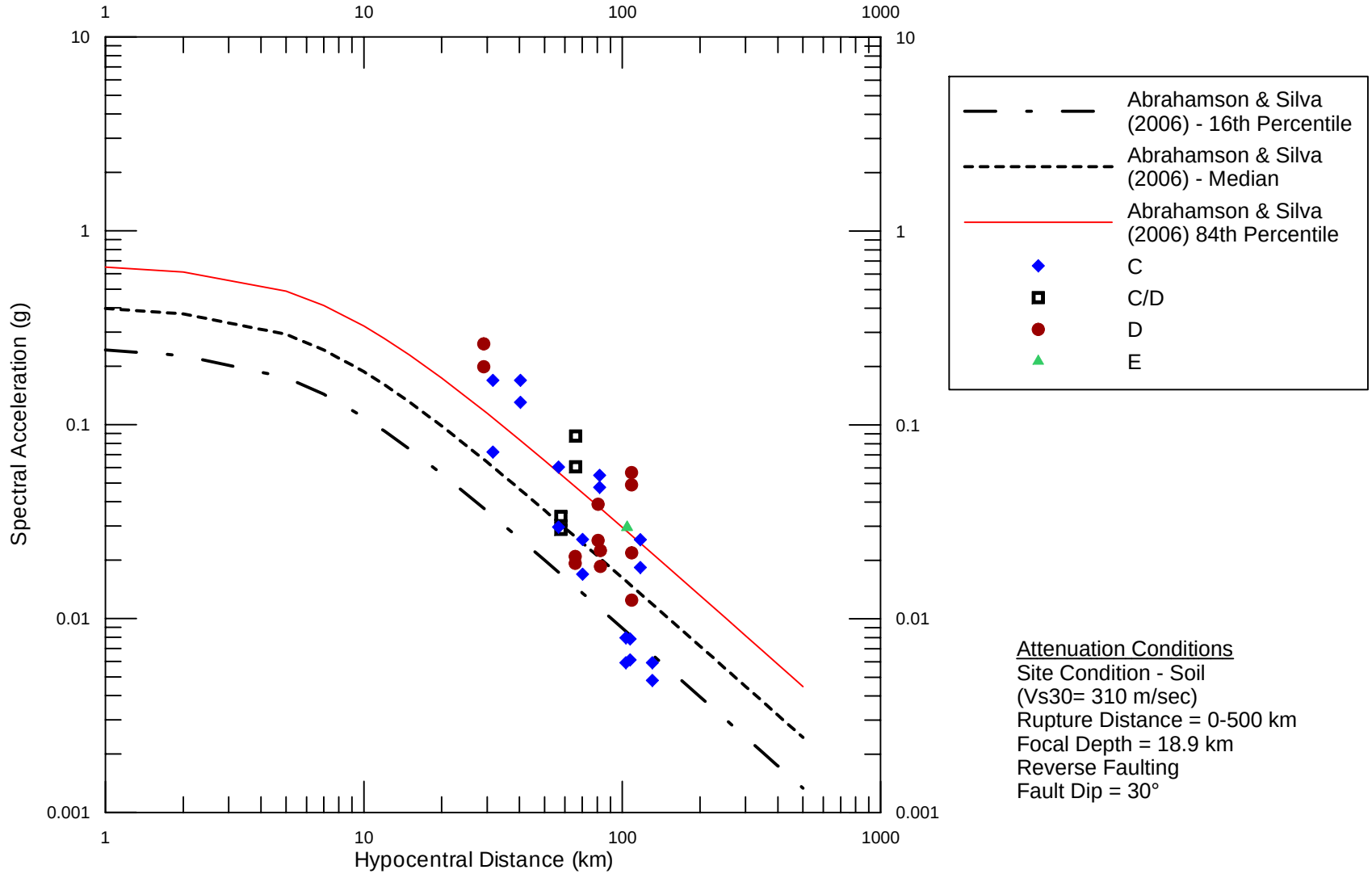
URS	Project No. 15298972 Kawaihae Harbor Hawaii	COMPARISON OF MUNSON AND THURBER (1997) ATTENUATION RELATIONSHIP FOR ASH AND **M** 6.7 KIHOLO BAY EARTHQUAKE STRONG MOTION DATA	Figure D1-2



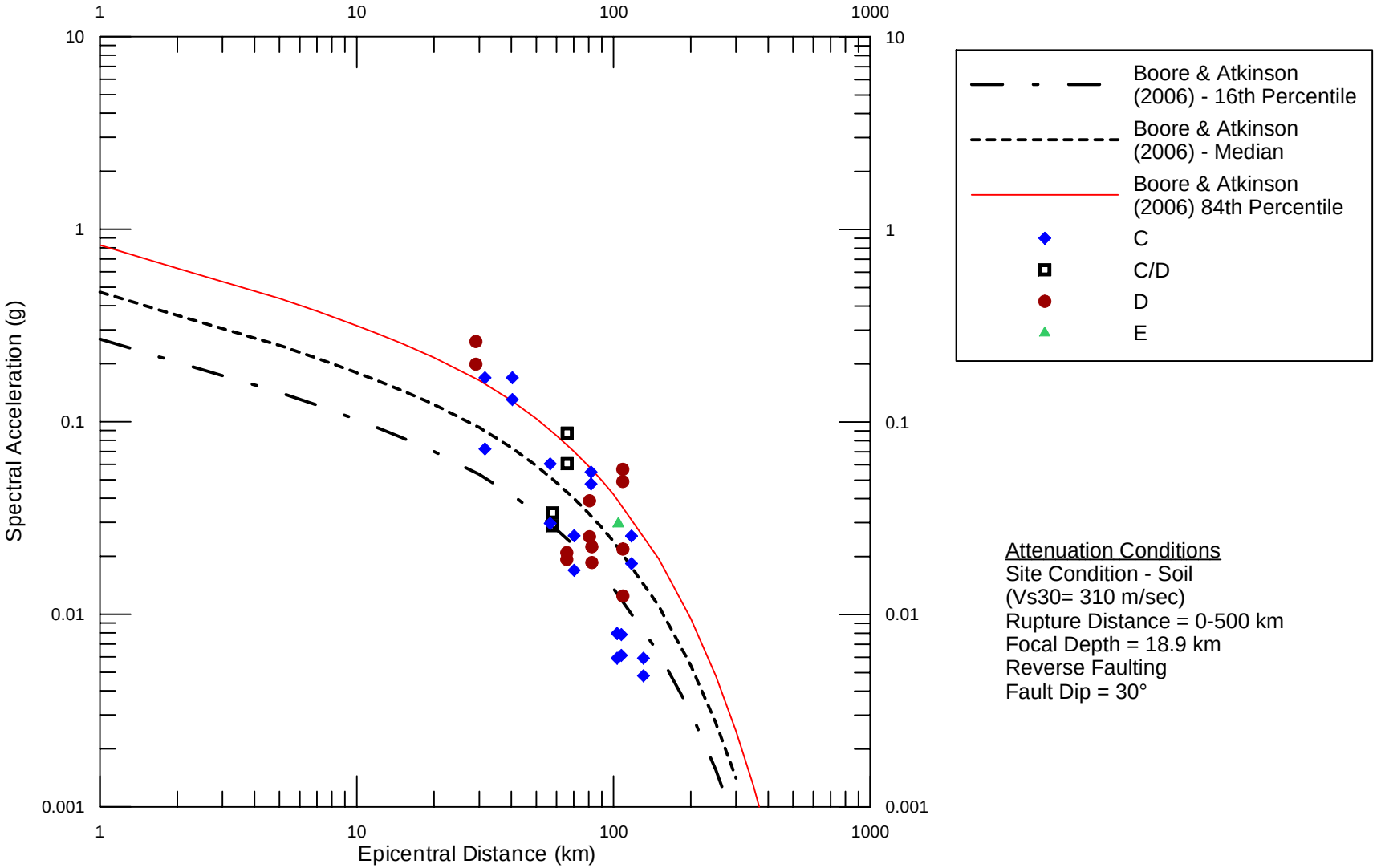
URS	Project No. 15298972 Kawaihae Harbor Hawaii	COMPARISON OF ABRAHAMSON AND SILVA (1997) ATTENUATION RELATIONSHIP FOR SOIL AND **M** 6.7 KIHOLO BAY EARTHQUAKE STRONG MOTION DATA	Figure D1-3



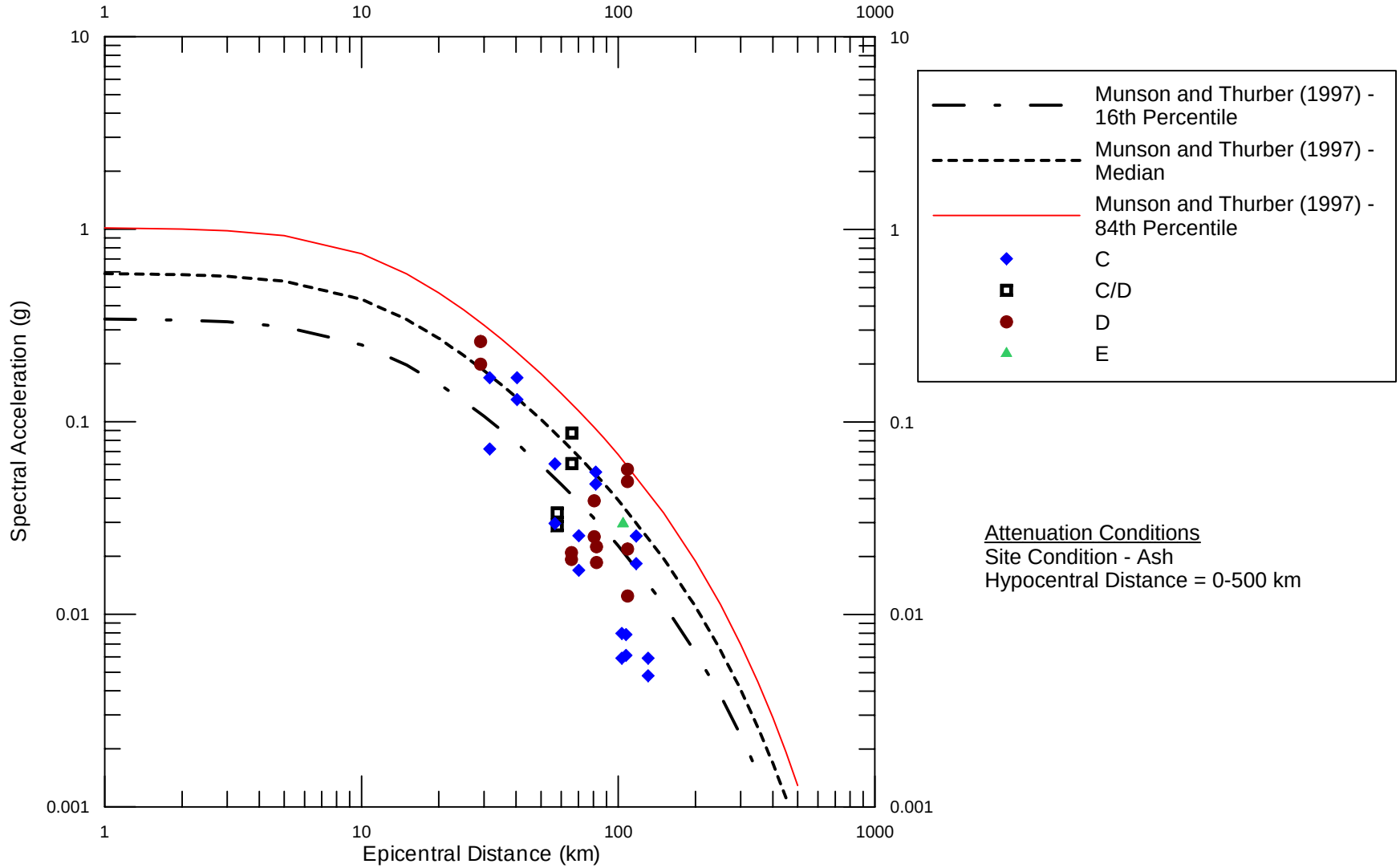
URS	Project No. 15298972 Kawaihae Harbor Hawaii	COMPARISON OF YOUNGS ET AL. (1997) ATTENUATION RELATIONSHIP FOR SOIL AND **M** 6.0 MAHUKONA EARTHQUAKE STRONG MOTION DATA	Figure D1-4



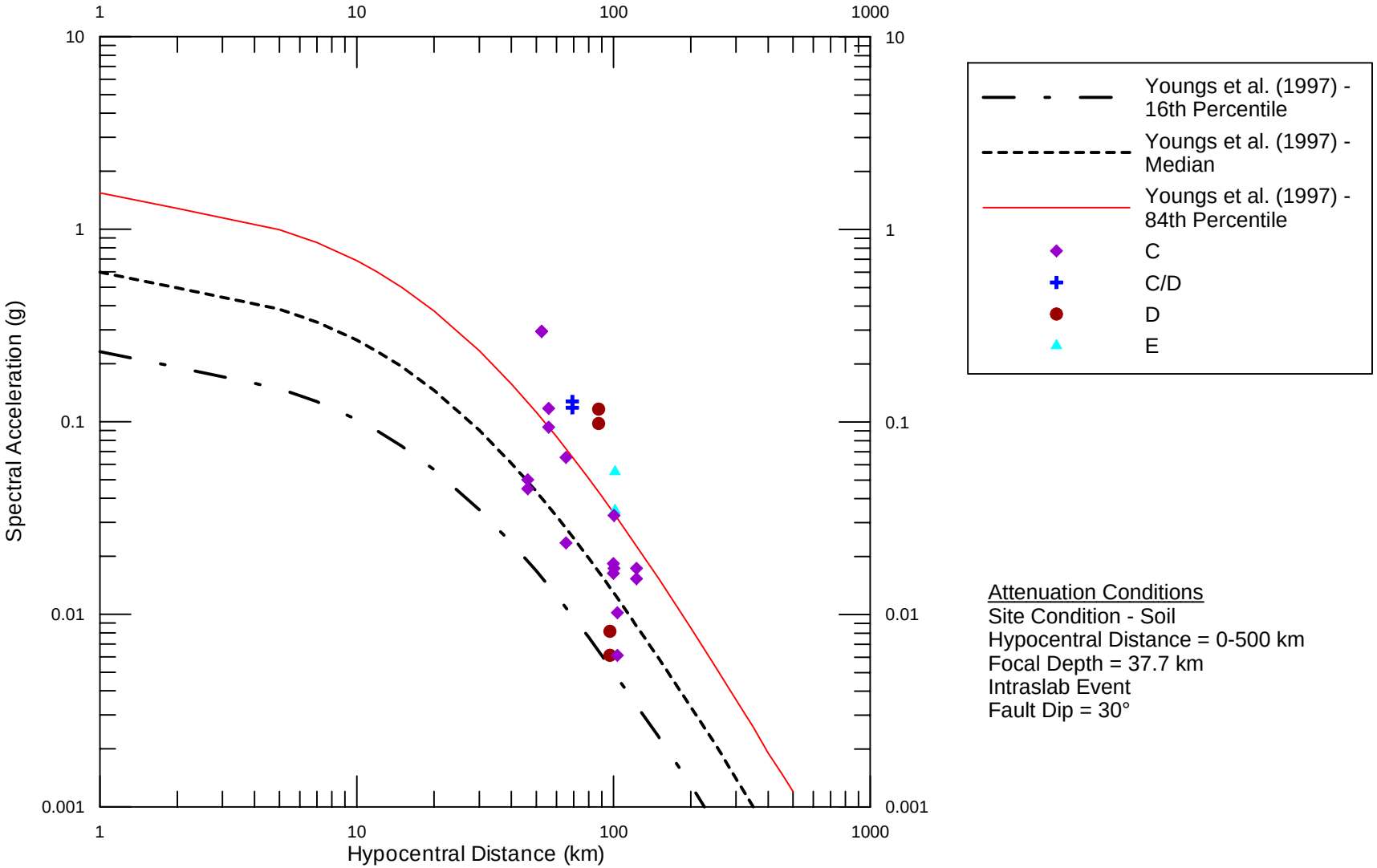
URS	Project No. 15298972 Kawaihae Harbor Hawaii	COMPARISON OF ABRAHAMSON AND SILVA (2007) ATTENUATION RELATIONSHIP FOR SOIL AND **M** 6.0 MAHUKONA EARTHQUAKE STRONG MOTION DATA	Figure D1-5



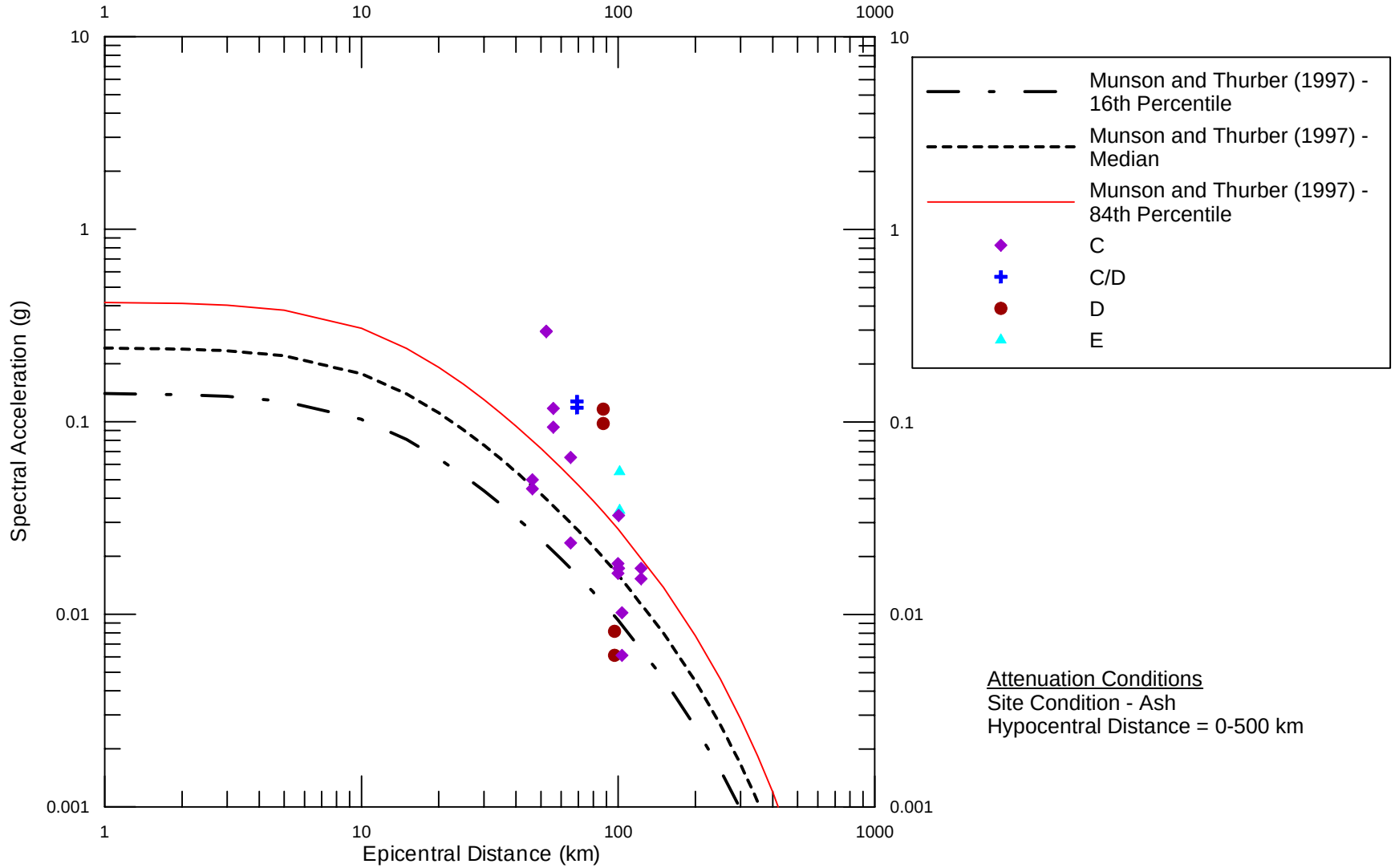
URS	Project No. 15298792 Kawaehae Harbor Hawaii	COMPARISON OF BOORE AND ATKINSON (2006) ATTENUATION RELATIONSHIP FOR SOIL AND **M** 6.0 MAHUKONA EARTHQUAKE STRONG MOTION DATA	Figure D1-6



URS	Project No. 15298972 Kawaihea Harbor Hawaii	COMPARISON OF MUNSON AND THURBER (1997) ATTENUATION RELATIONSHIP FOR ASH AND **M** 6.0 MAHUKONA EARTHQUAKE STRONG MOTION DATA	Figure D1-7

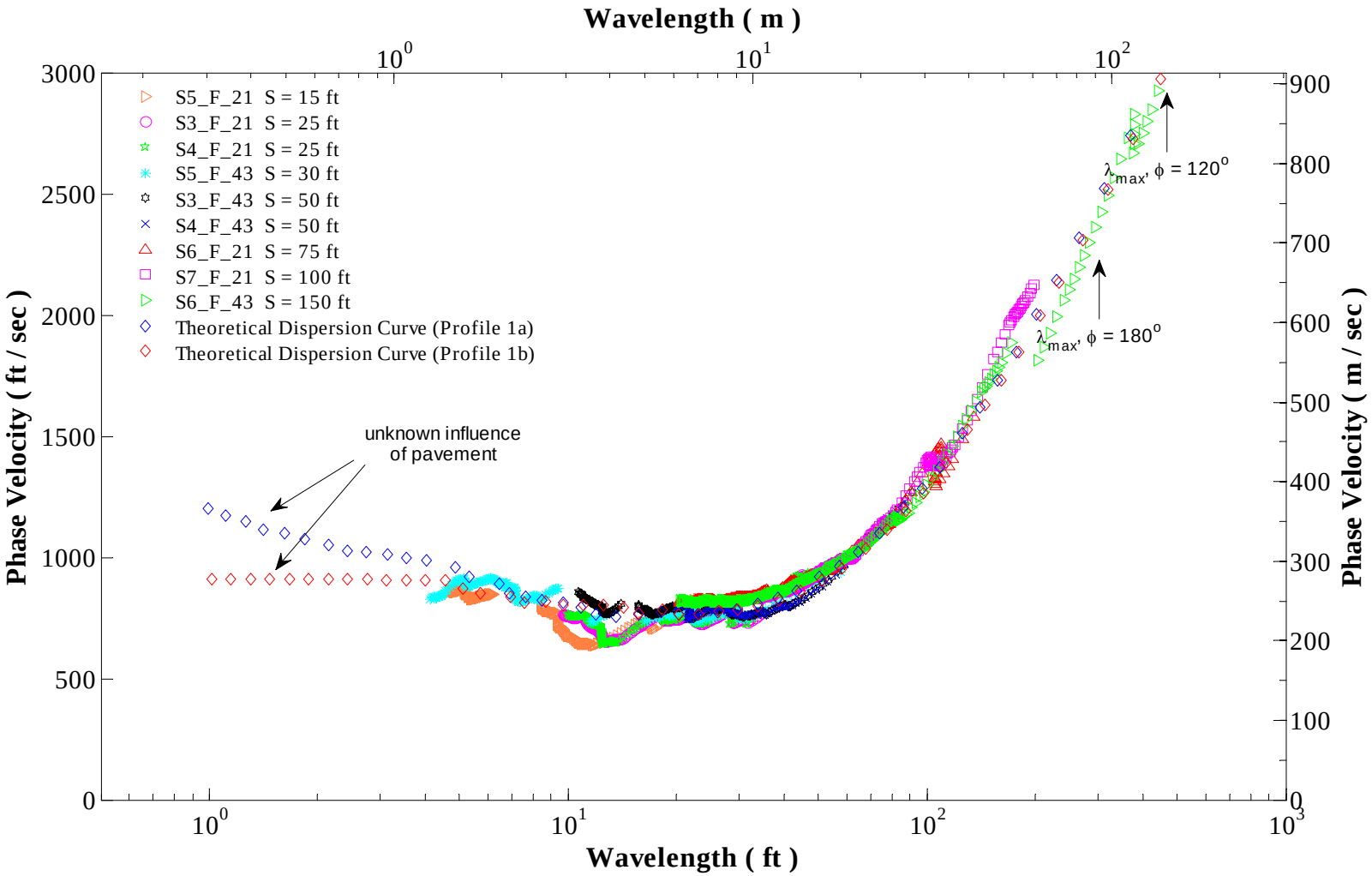


URS	Project No. 15298972 Kawaihae Harbor Hawaii	COMPARISON OF YOUNGS ET AL. (1997) ATTENUATION RELATIONSHIP FOR SOIL AND **M** 5.0 PUAKO EARTHQUAKE STRONG MOTION DATA	Figure D1-8

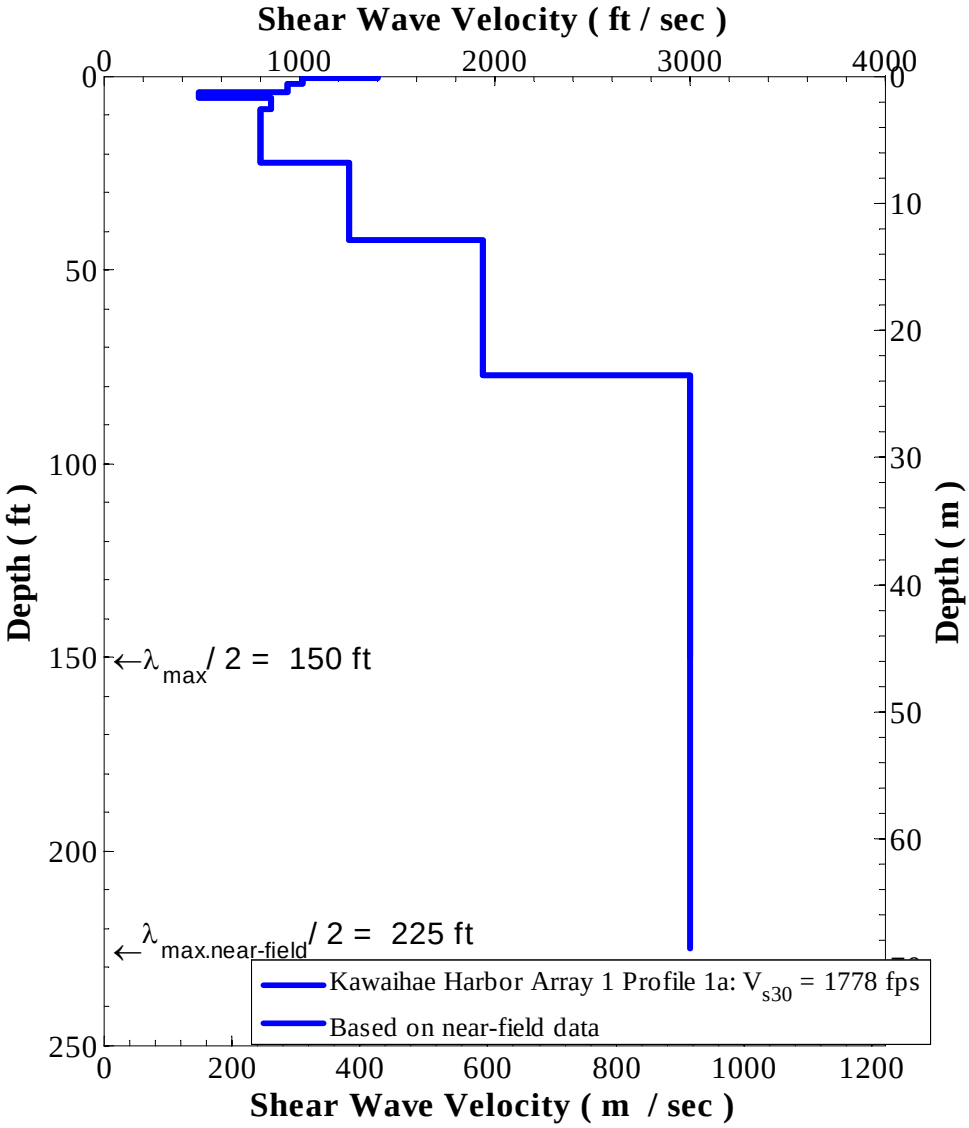


URS	Project No. 15298972 Kawaihae Harbor Hawaii	COMPARISON OF MUNSON AND THURBER (1997) ATTENUATION RELATIONSHIP FOR ASH AND **M** 5.0 PUAKO EARTHQUAKE STRONG MOTION DATA	Figure D1-9

Appendix D2
Shear-Wave Velocity Profiling of Kawaihae Harbor and the USGS Strong Motion Stations



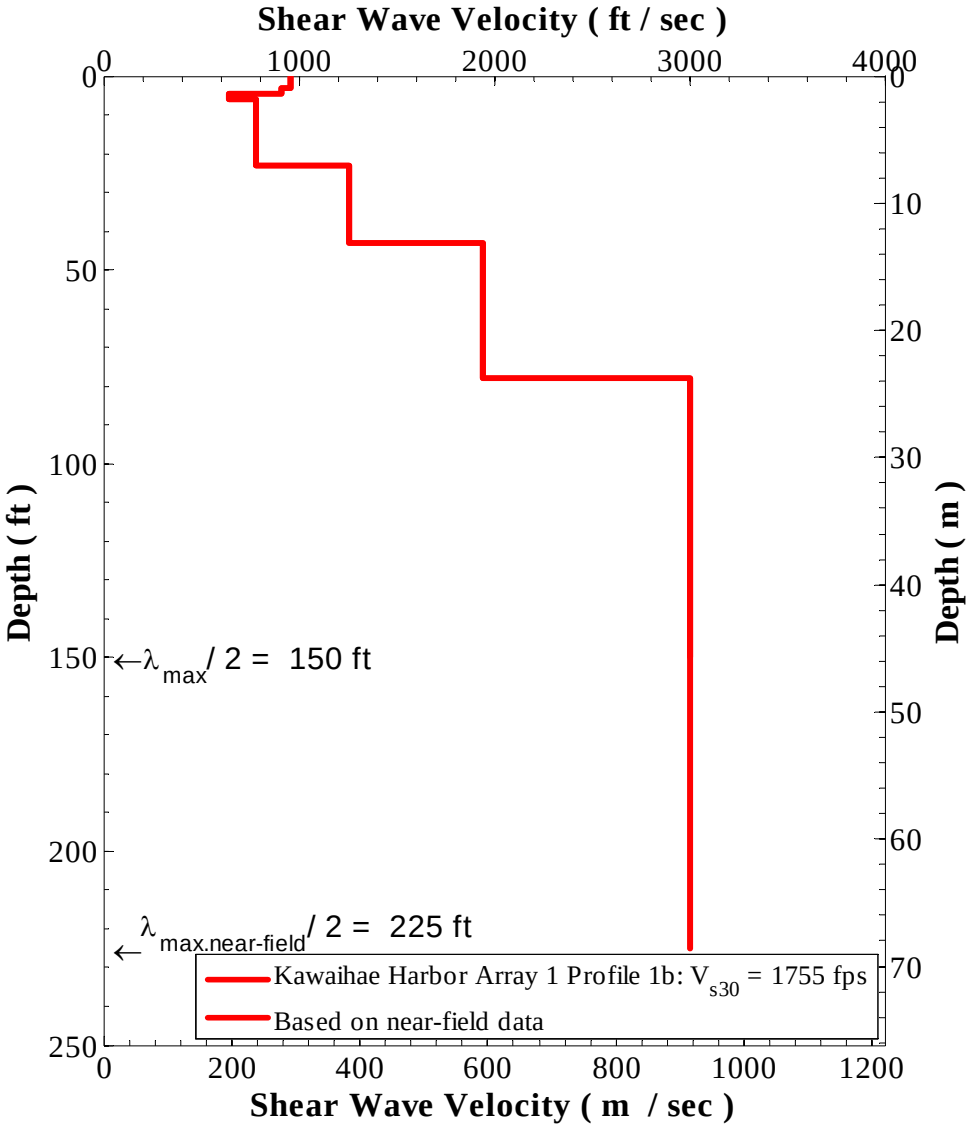
Kawaihae Harbor, Array 1



Kawaihae Harbor Array 1 Profile 1a

Profile Parameter for Kawaihae Harbor Array 1 Profile 1a

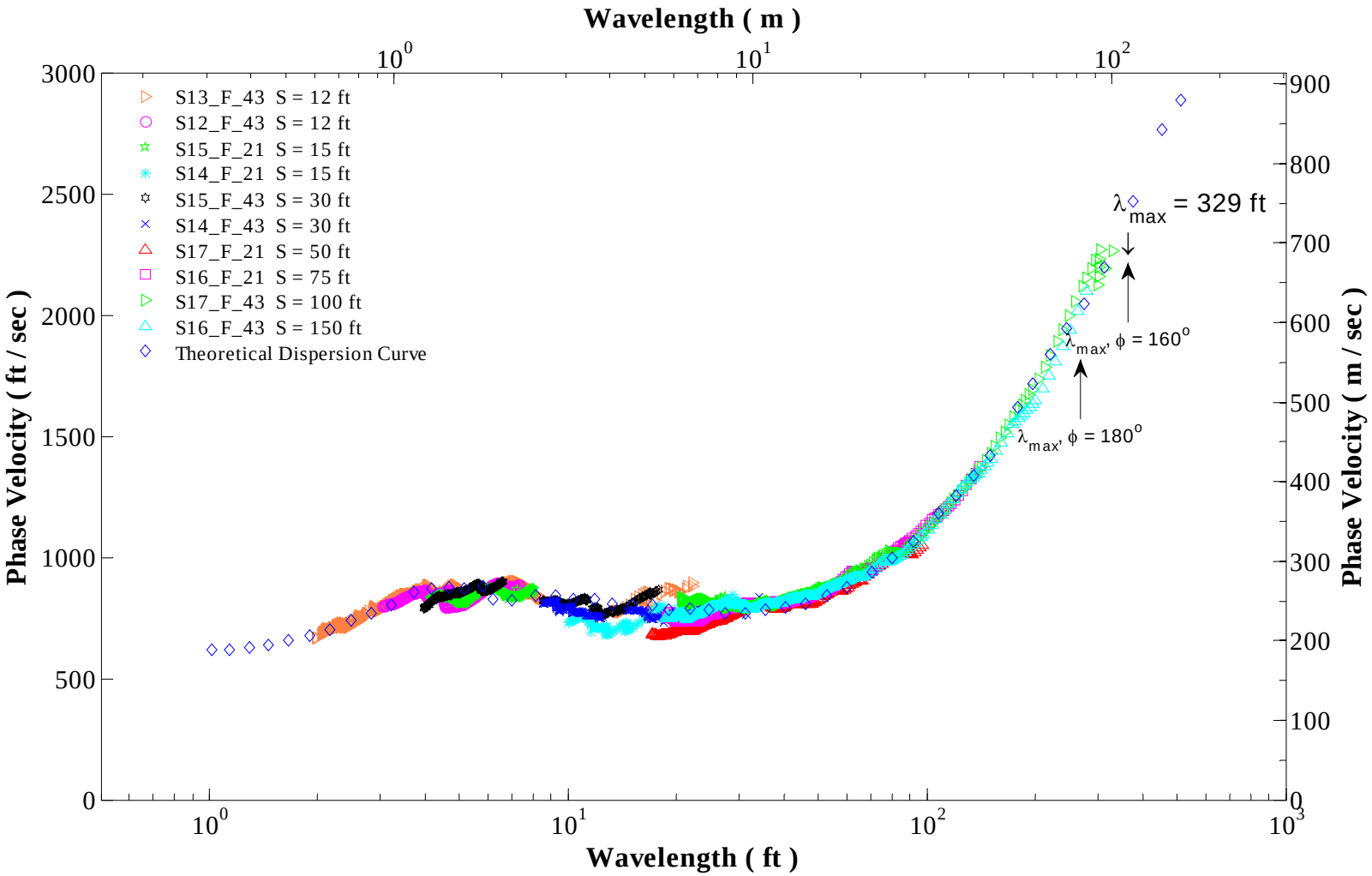
Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	0.5	2779.3	1400	0.33	120
2	1.5	2024.9	1020	0.33	120
3	2	1866.1	940	0.33	120
4	1.7	972.8	490	0.33	120
5	3	1707.3	860	0.33	120
6	13.5	5000	800	0.4869	120
7	20	5000	1260	0.4661	120
8	35	5000	1940	0.4114	120
9	99999	5000	3000	0.2188	130



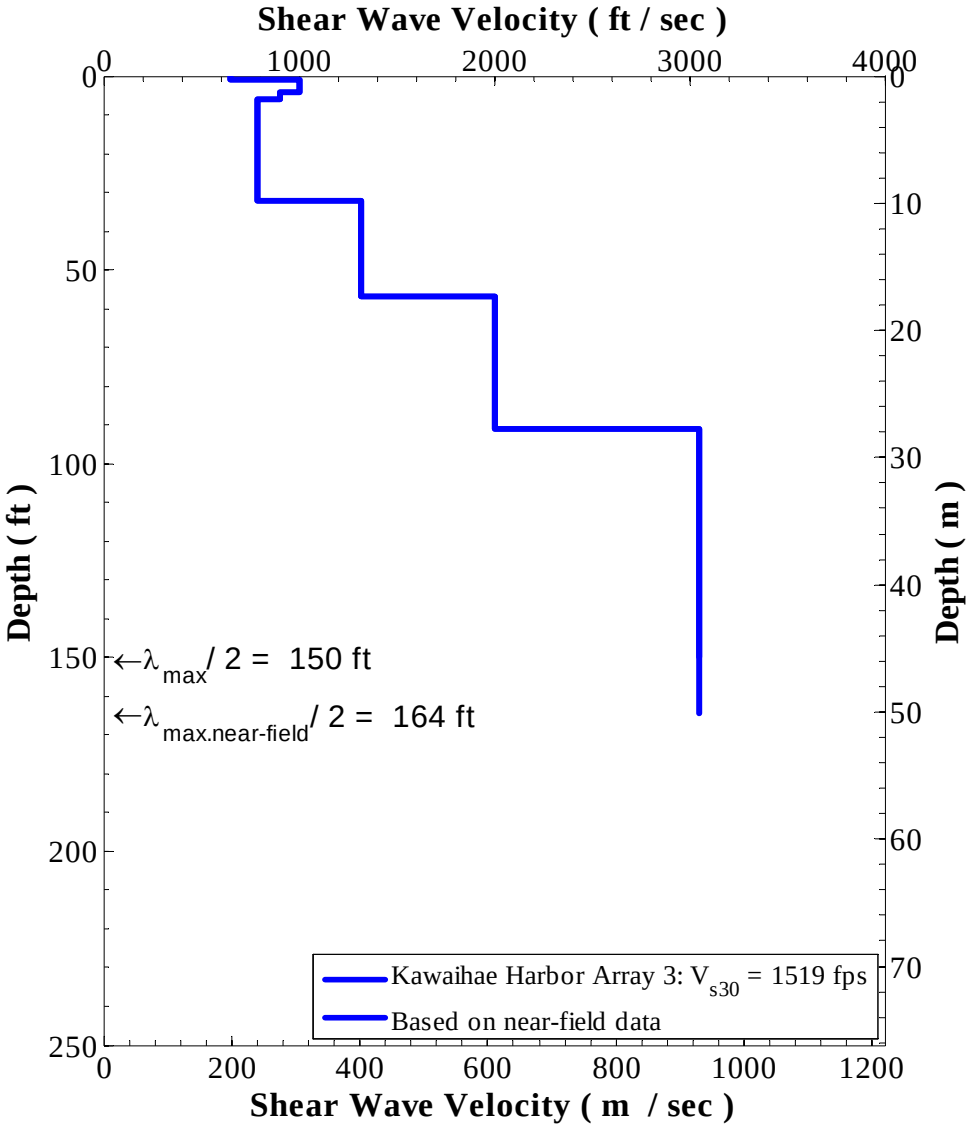
Kawaihae Harbor Array 1 Profile 1b

Profile Parameter for Kawaihae Harbor Array 1 Profile 1b

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	3	1905.8	960	0.33	120
2	1.5	1806.6	910	0.33	120
3	1.5	1270.6	640	0.33	120
4	17	5000	780	0.4875	120
5	20	5000	1260	0.4661	120
6	35	5000	1940	0.4114	120
7	99999	5000	3000	0.2188	130



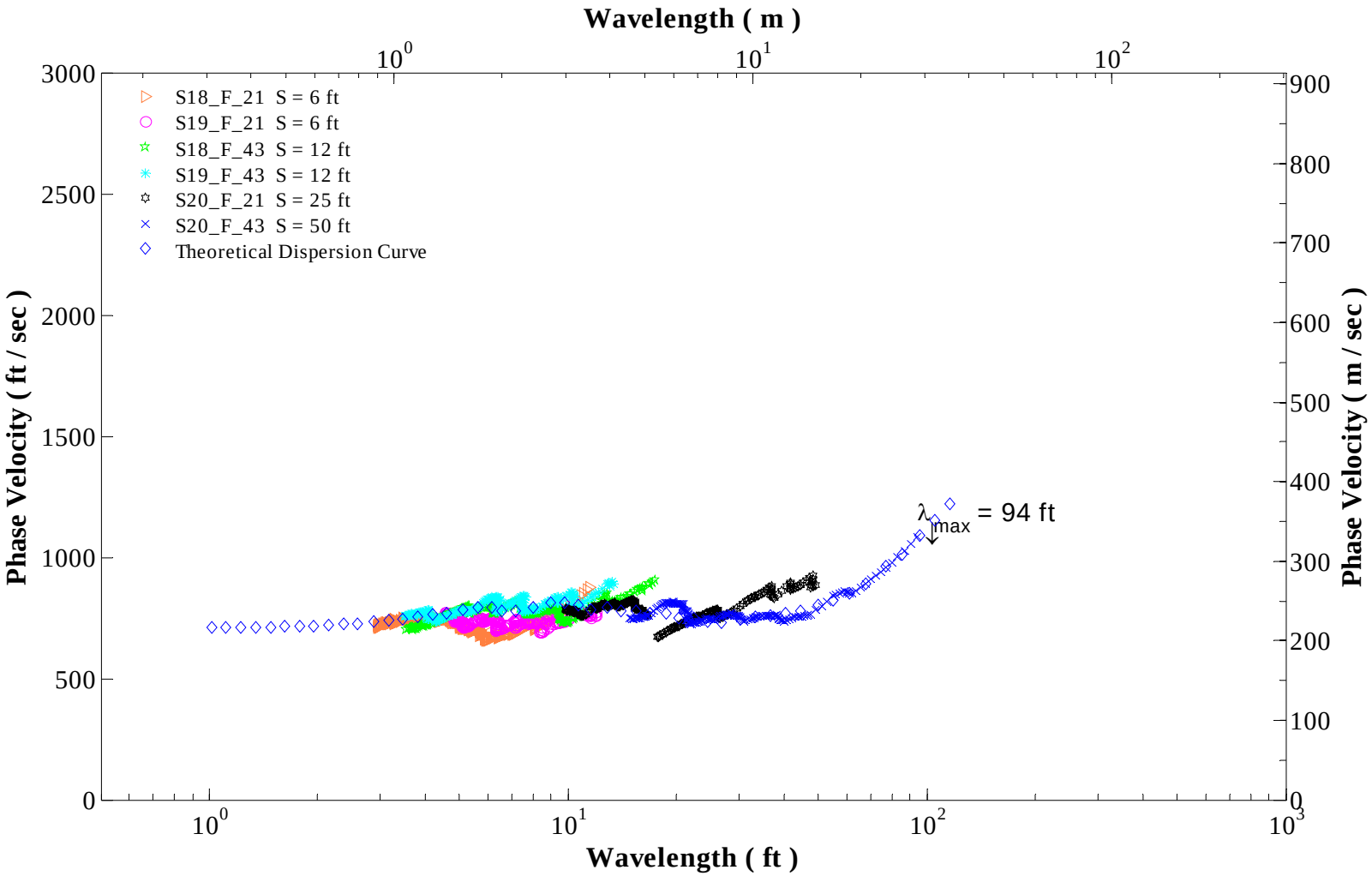
Kawaihae Harbor, Array 3



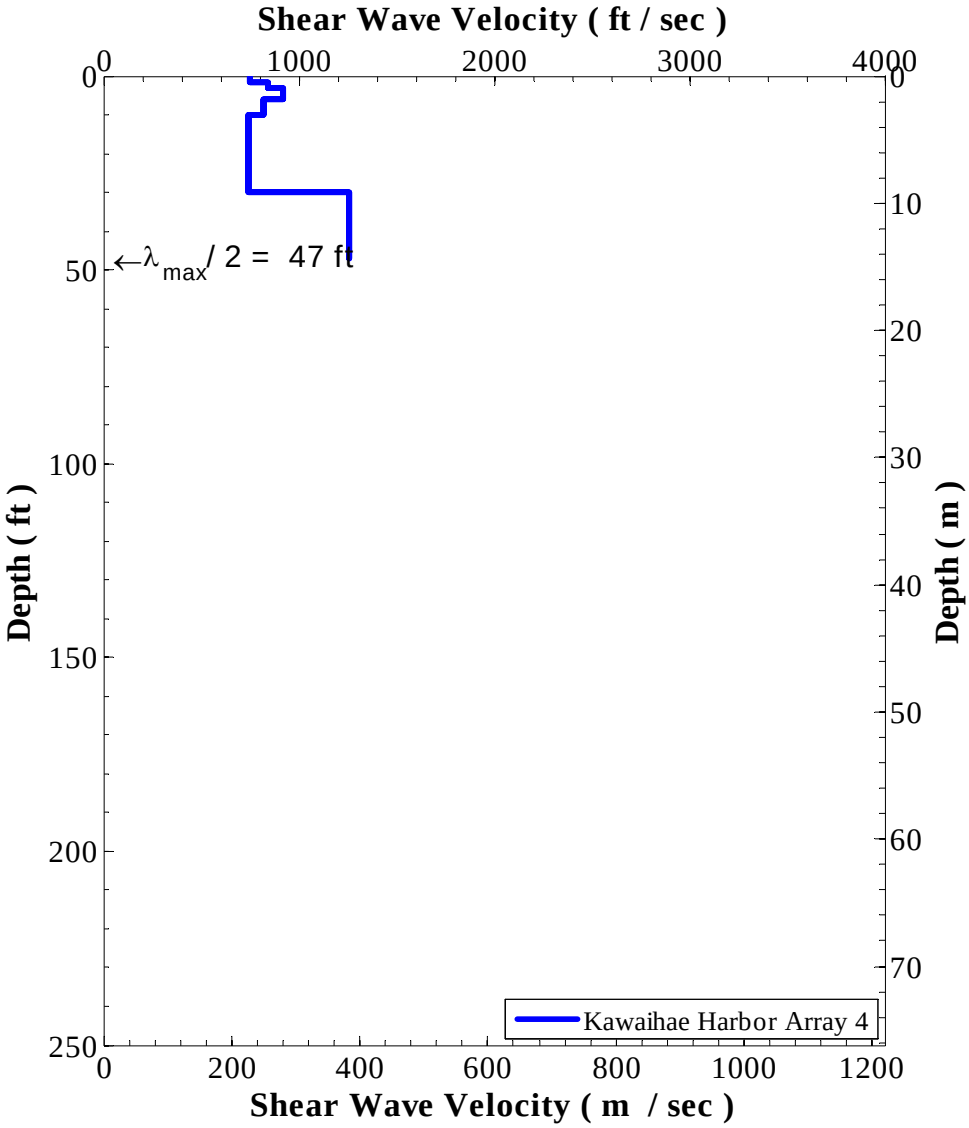
Kawaihae Harbor Array 3

Profile Parameter for Kawaihae Harbor Array 3

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	1	1905.8	650	0.33	120
2	3	1806.6	1000	0.33	120
3	2	1270.6	900	0.33	120
4	26	5000	790	0.4875	120
5	25	5000	1320	0.4625	120
6	34	5000	2000	0.4048	120
7	99999	5000	3050	0.2037	130



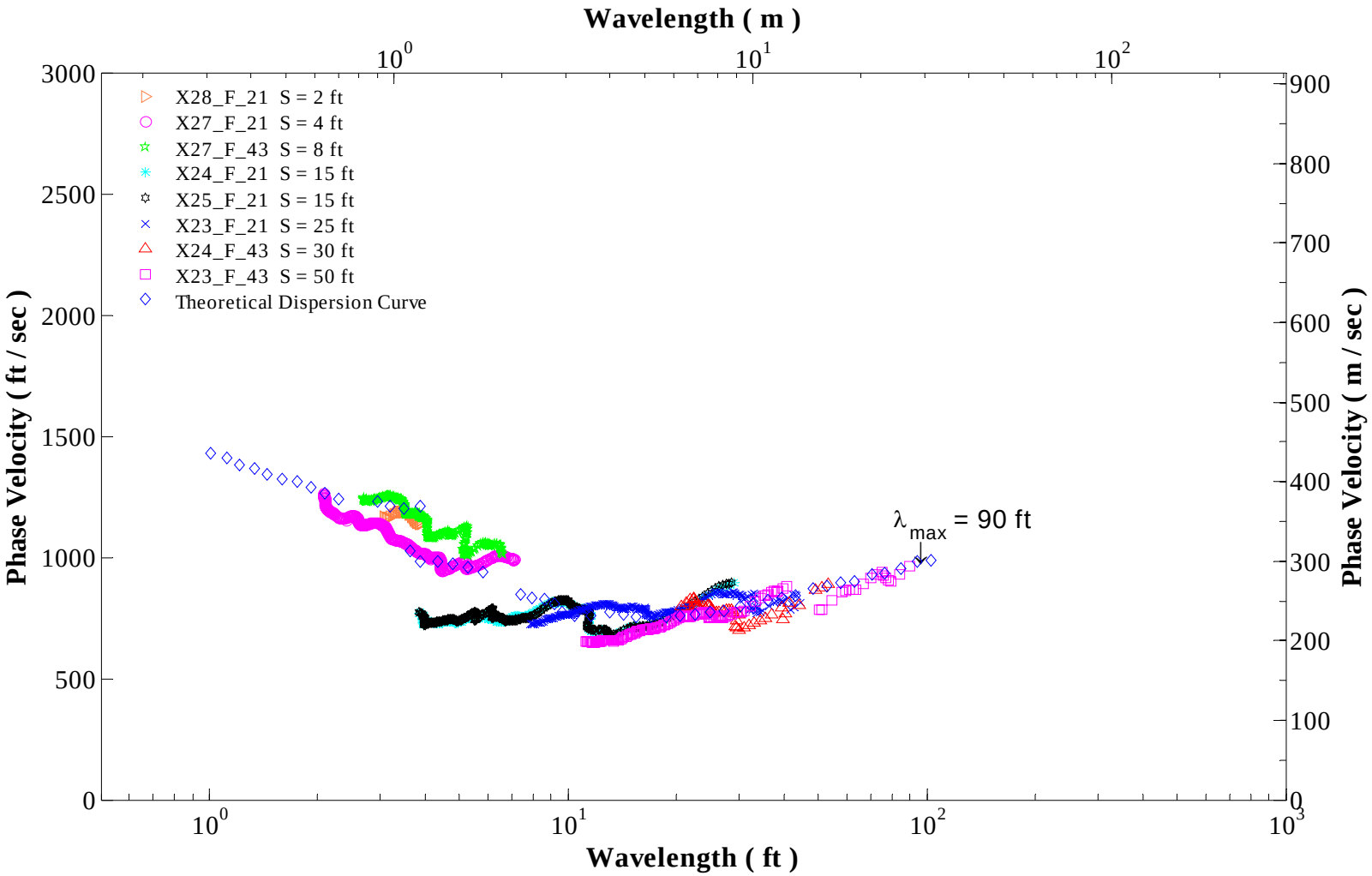
Kawaihae Harbor, Array 4



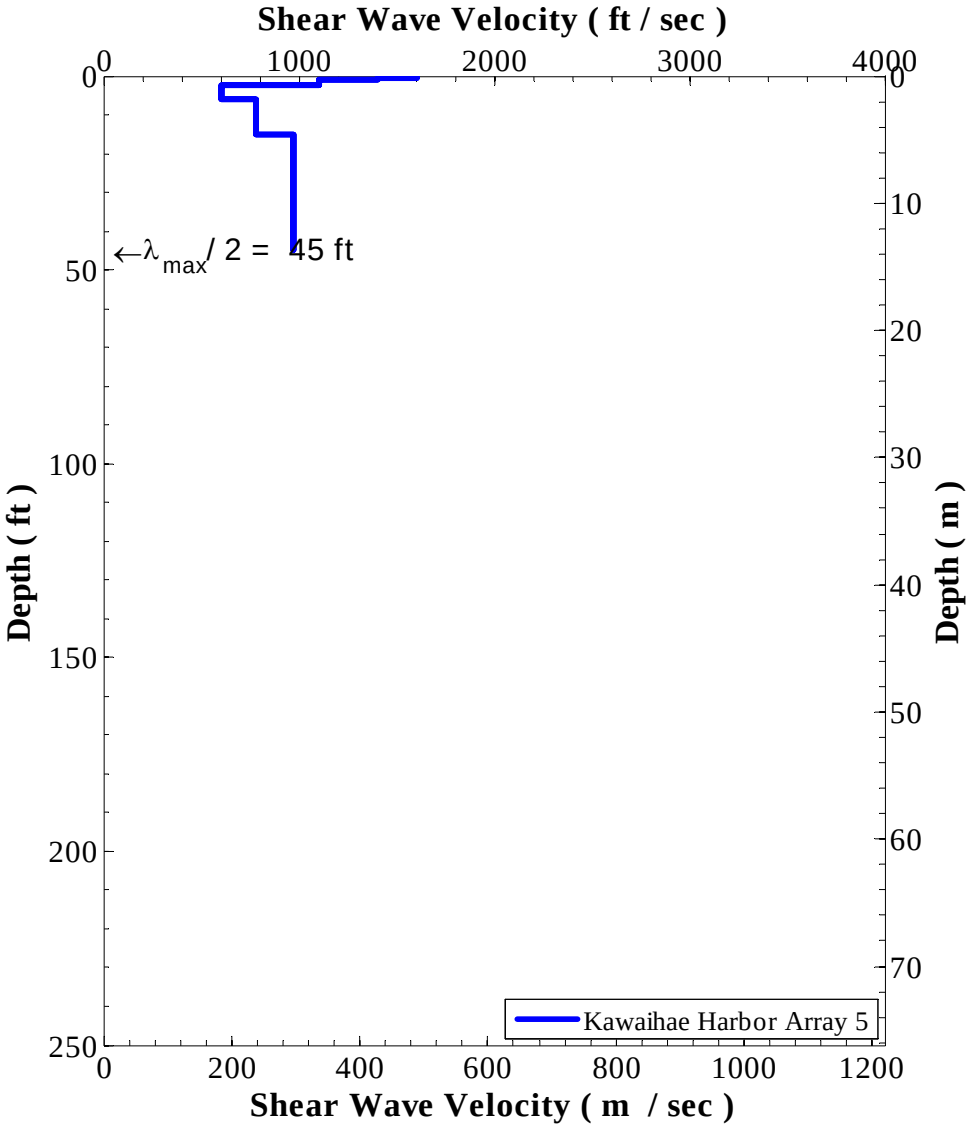
Kawaihae Harbor Array 4

Profile Parameter for Kawaihae Harbor Array 4

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	1.5	1588.2	750	0.33	120
2	1.5	1667.6	840	0.33	120
3	3	1826.4	920	0.33	120
4	4	5000	820	0.4862	120
5	20	5000	740	0.4888	120
6	25	5000	1260	0.4661	130
7	99999	5000	2500	0.3333	130



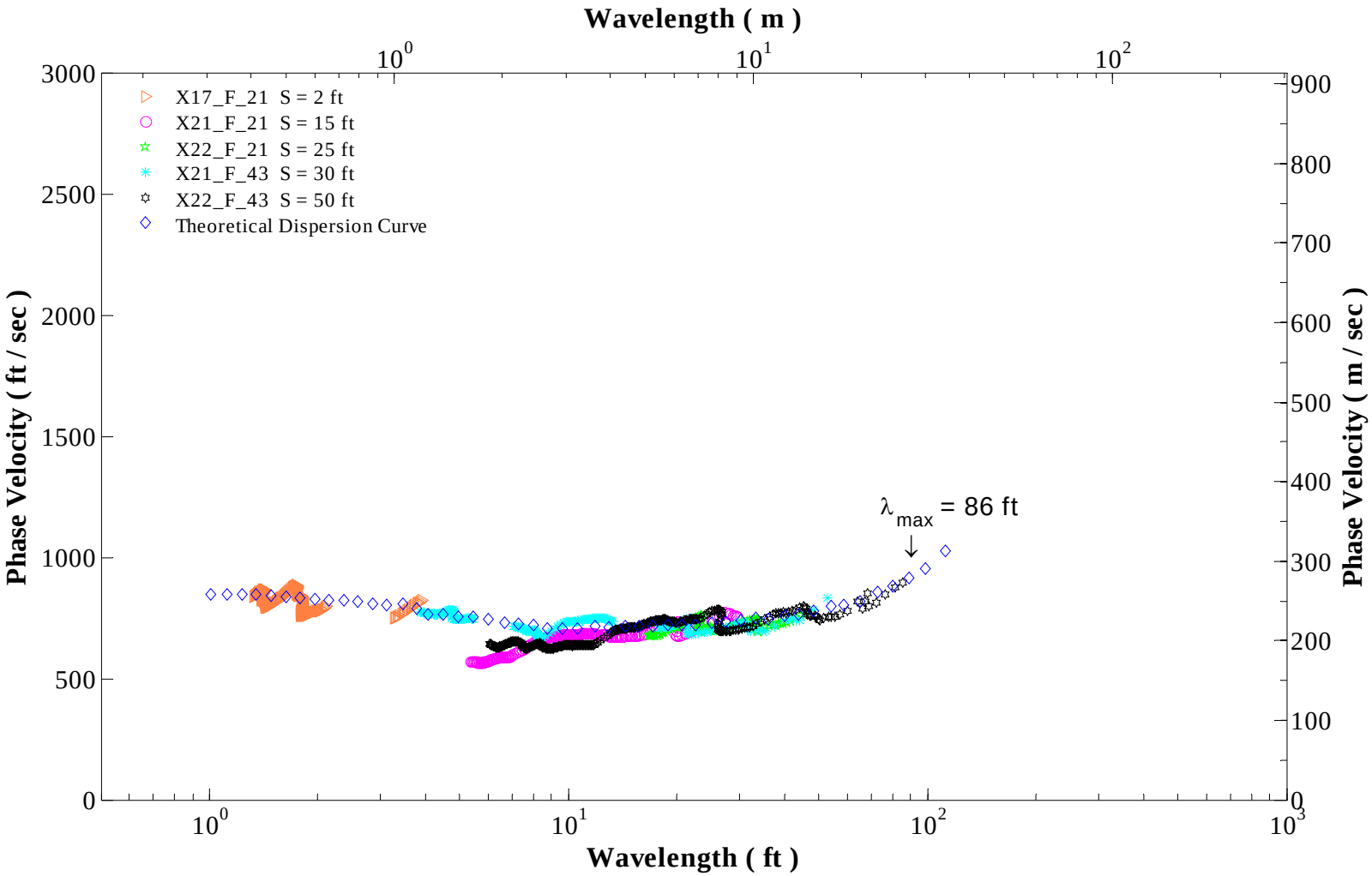
Kawaihae Harbor, Array 5



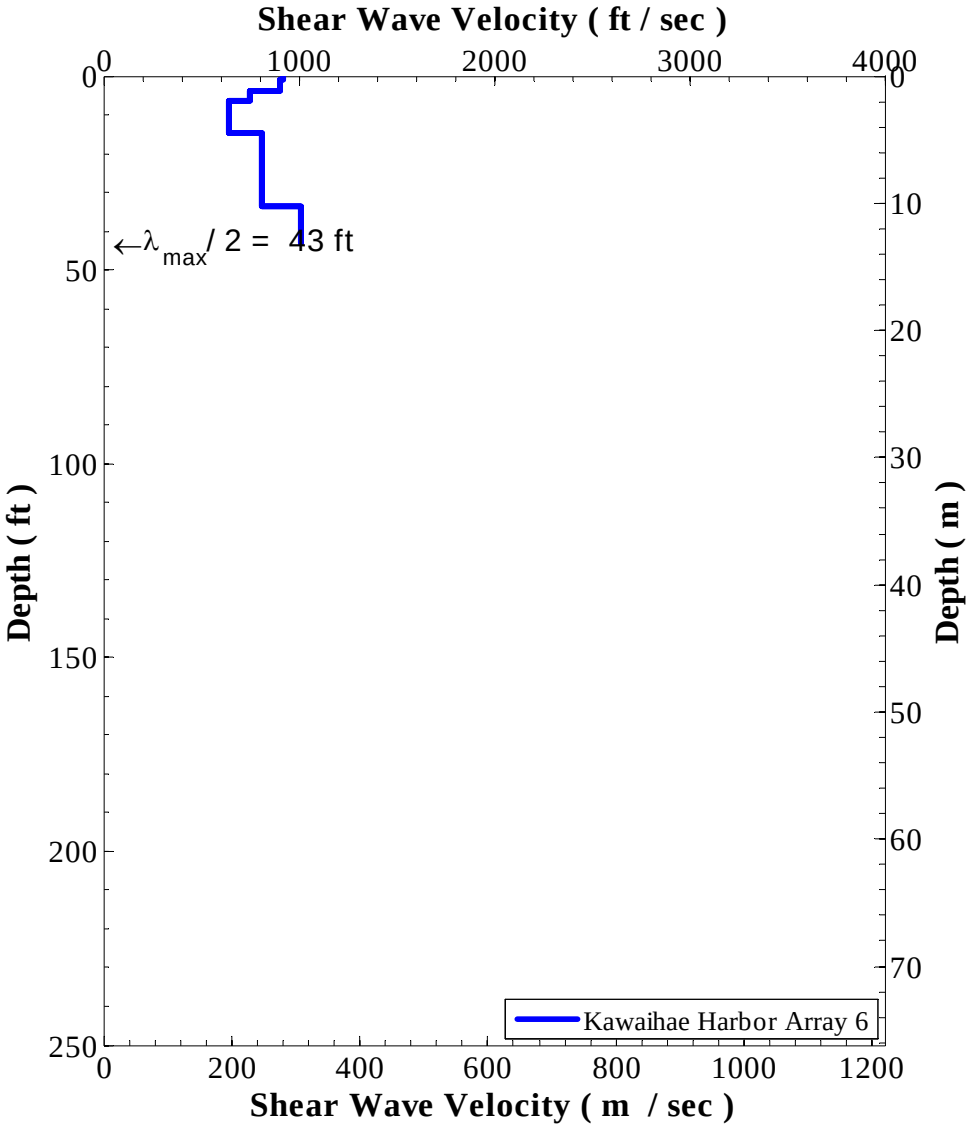
Kawaihae Harbor Array 5

Profile Parameter for Kawaihae Harbor Array 5

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	0.5	3176.4	1600	0.33	130
2	0.5	2779.3	1400	0.33	120
3	1.5	2183.8	1100	0.33	120
4	3.5	1191.1	600	0.33	120
5	9	5000	780	0.4875	120
6	30	5000	970	0.4804	120
7	9999	5000	1100	0.4746	130



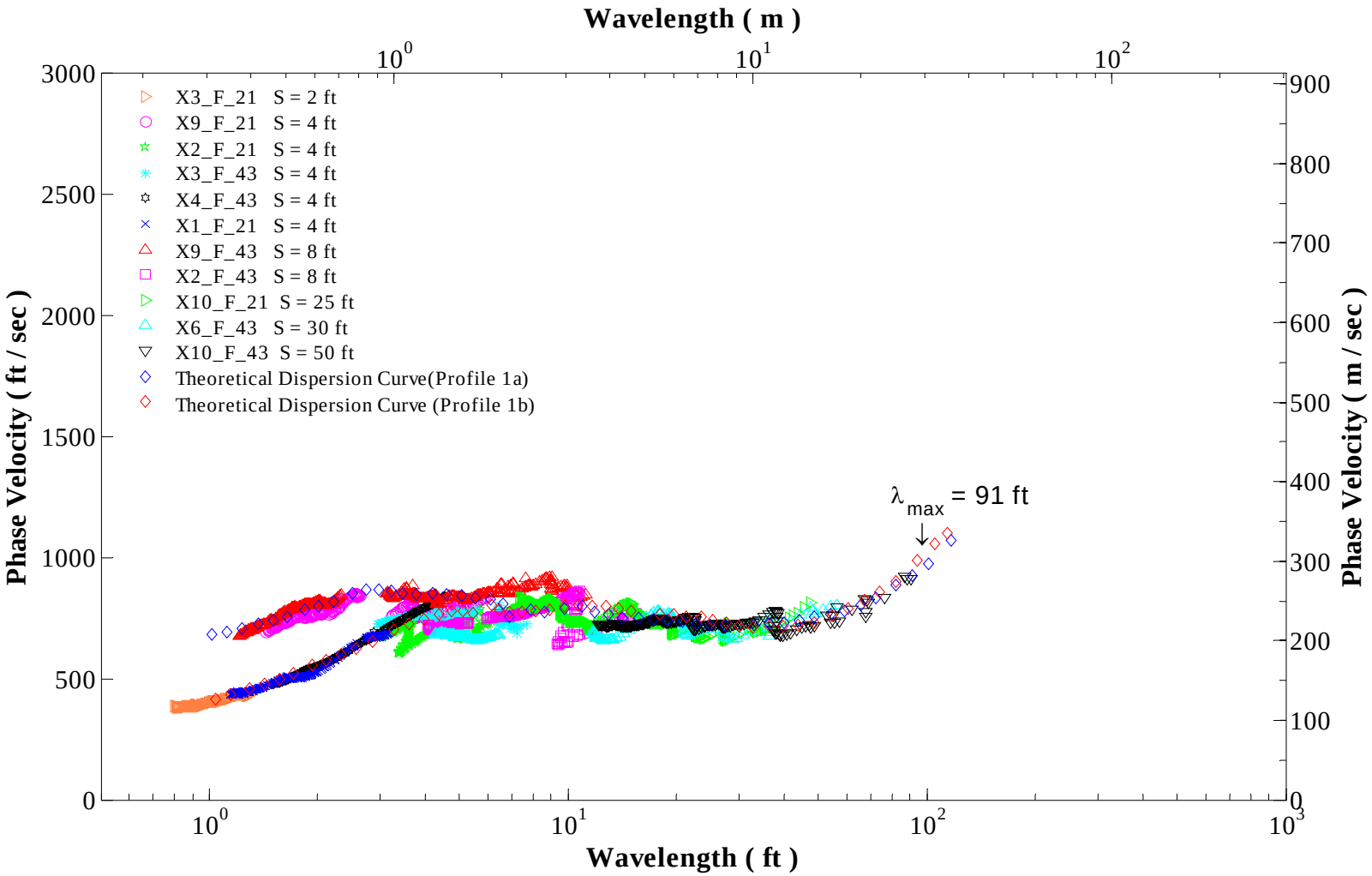
Kawaihae Harbor, Array 6



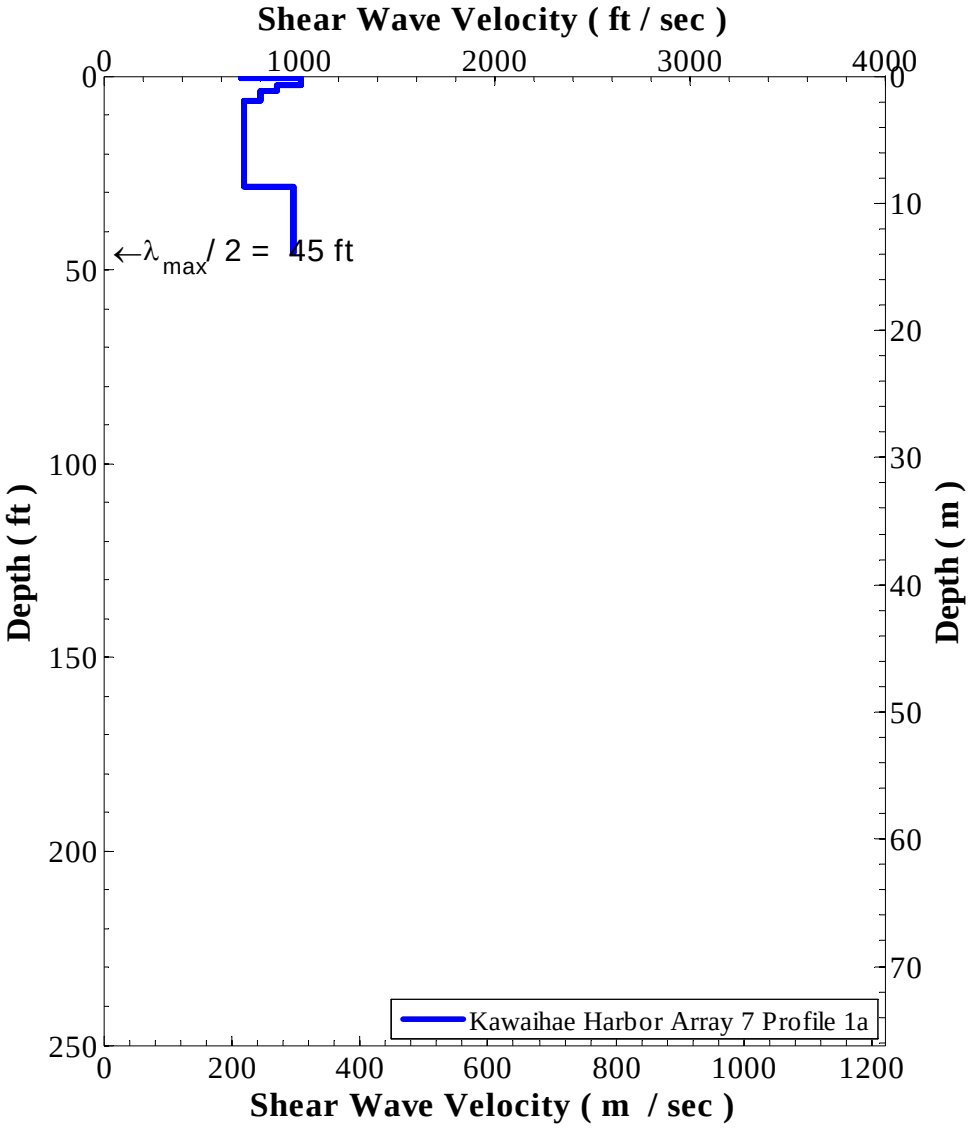
Kawaihae Harbor Array 6

Profile Parameter for Kawaihae Harbor Array 6

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	1	1786.7	920	0.33	130
2	2.8	1786.7	900	0.33	120
3	2.5	1429.4	750	0.33	120
4	8.3	5000	640	0.4919	120
5	19	5000	810	0.4869	120
6	29	5000	1010	0.4792	120
7	99999	5000	3000	0.1878	130



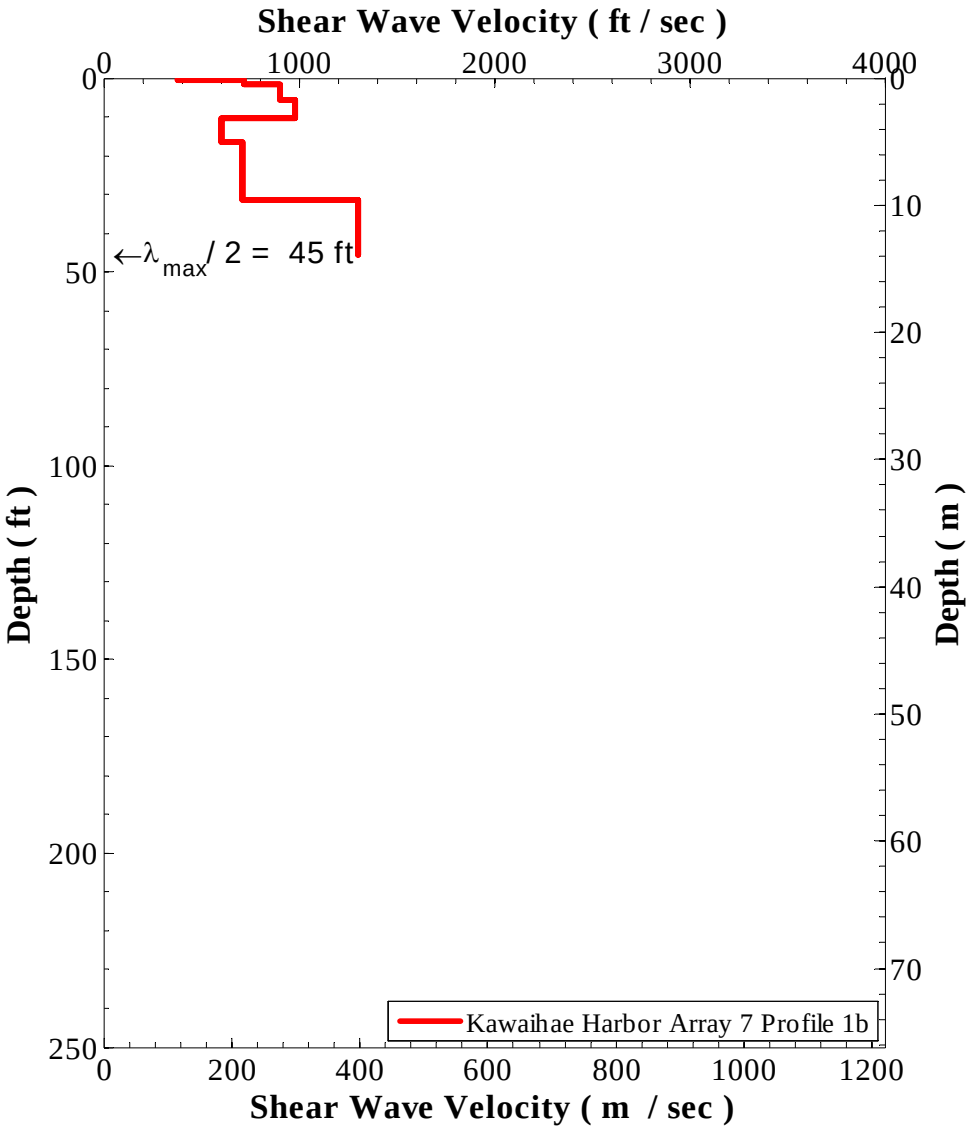
Kawaihae Harbor, Array 7



Kawaihae Harbor Array 7 Profile 1a

Profile Parameter for Kawaihae Harbor Array 7 Profile 1a

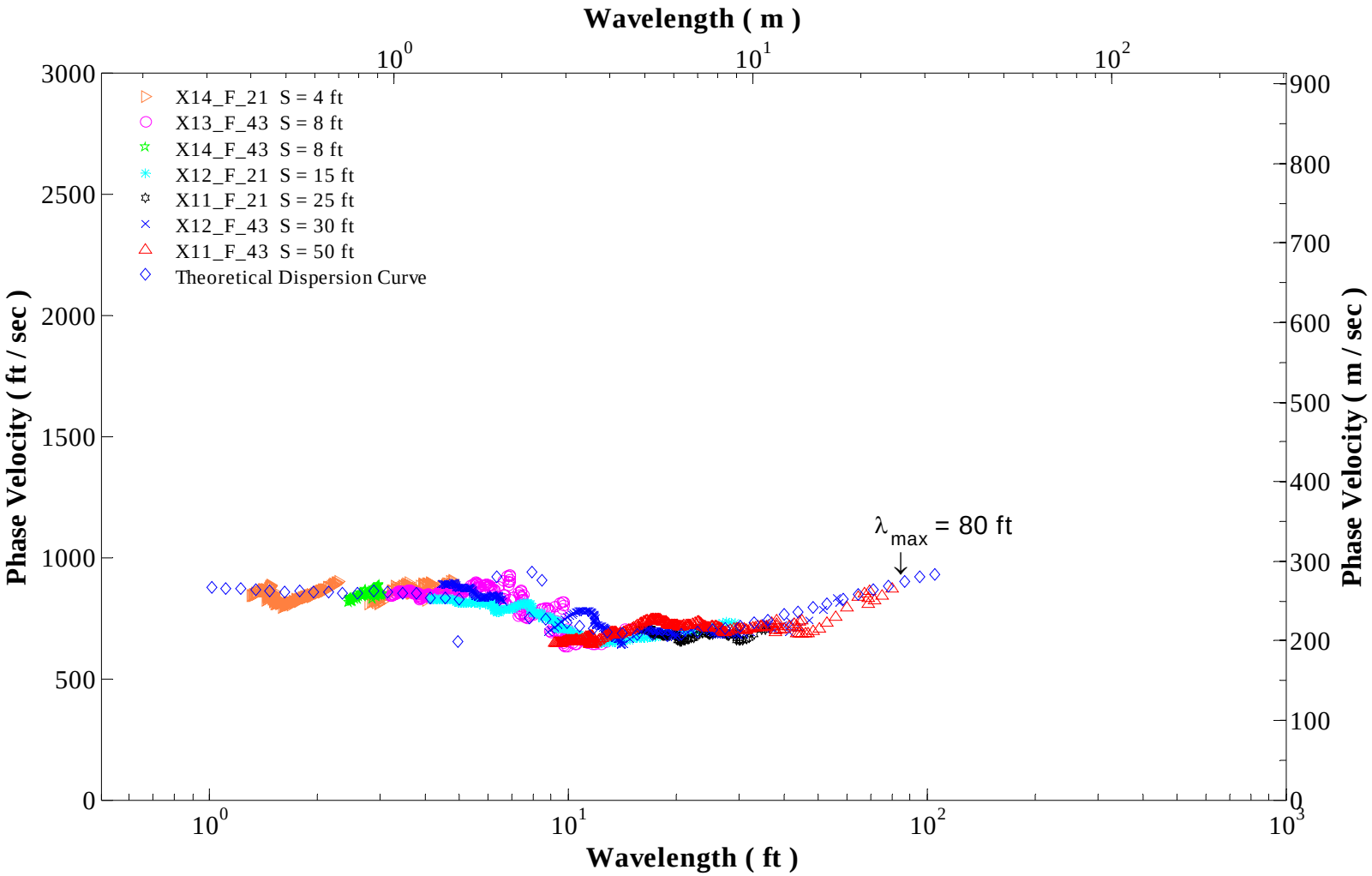
Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	0.7	1786.7	700	0.33	130
2	1.5	1647.8	1010	0.33	120
3	1.5	1191.1	890	0.33	120
4	2.5	1588.2	800	0.33	120
5	22.5	5000	720	0.4894	120
6	31	5000	970	0.4804	120
7	9999	5000	2500	0.3333	130



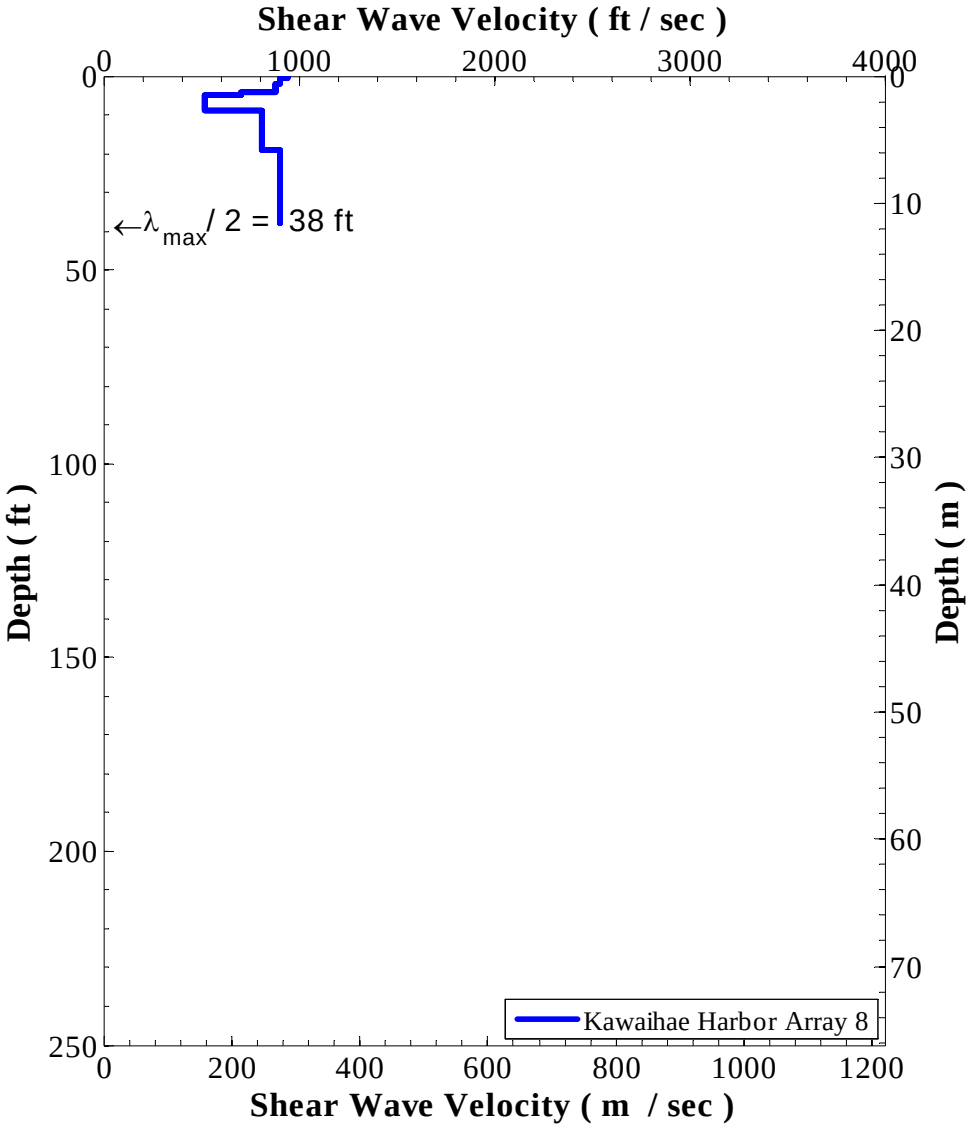
Kawaihae Harbor Array 7 Profile 1b

Profile Parameter for Kawaihae Harbor Array 7 Profile 1b

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	0.5	754.4	380	0.33	130
2	1	1429.4	720	0.33	120
3	4	1786.7	900	0.33	120
4	5	1945.5	980	0.33	120
5	6	5000	600	0.4927	120
6	15	5000	710	0.4897	120
7	9999	5000	1300	0.4637	120



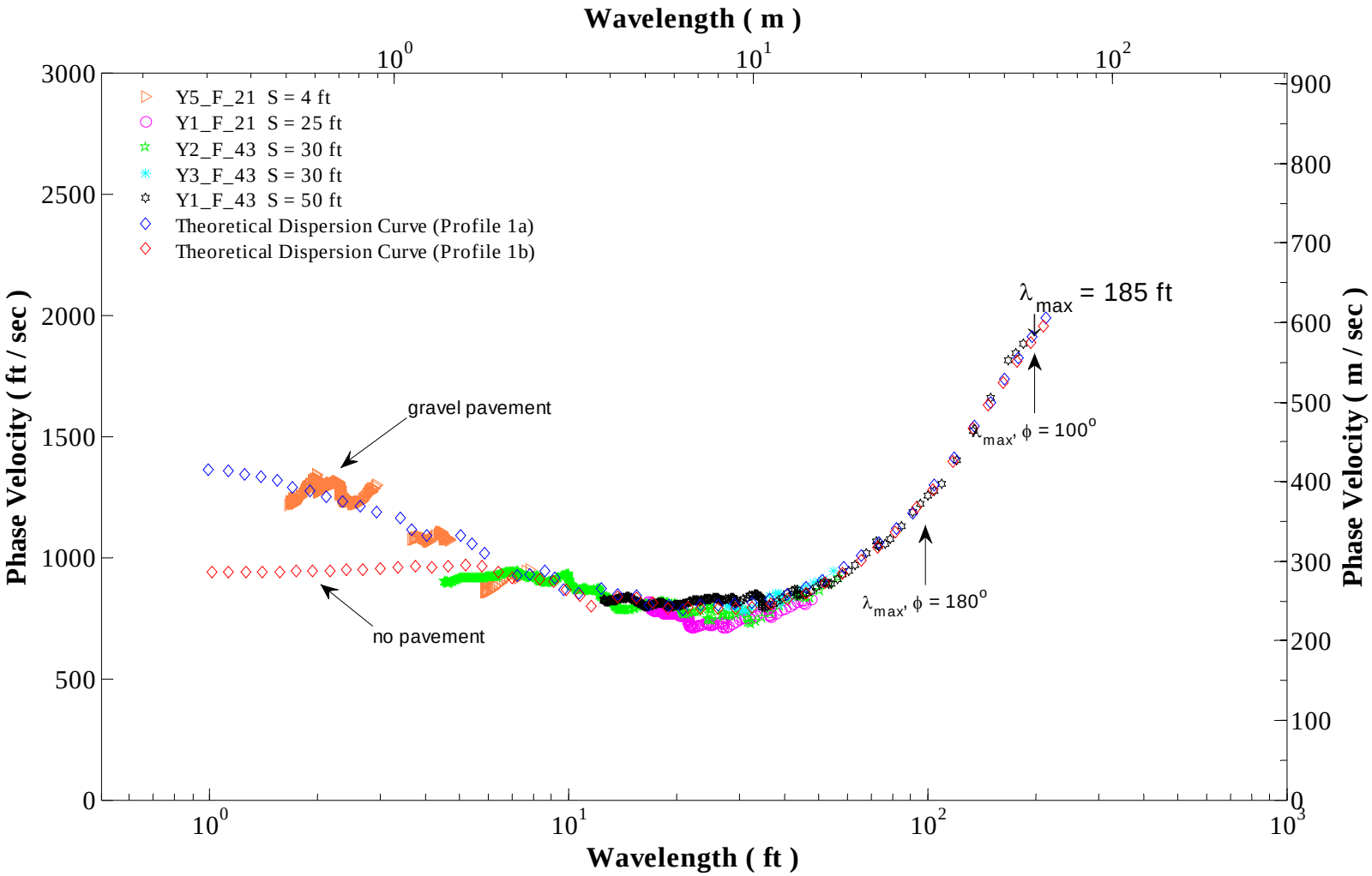
Kawaihae Harbor, Array 8



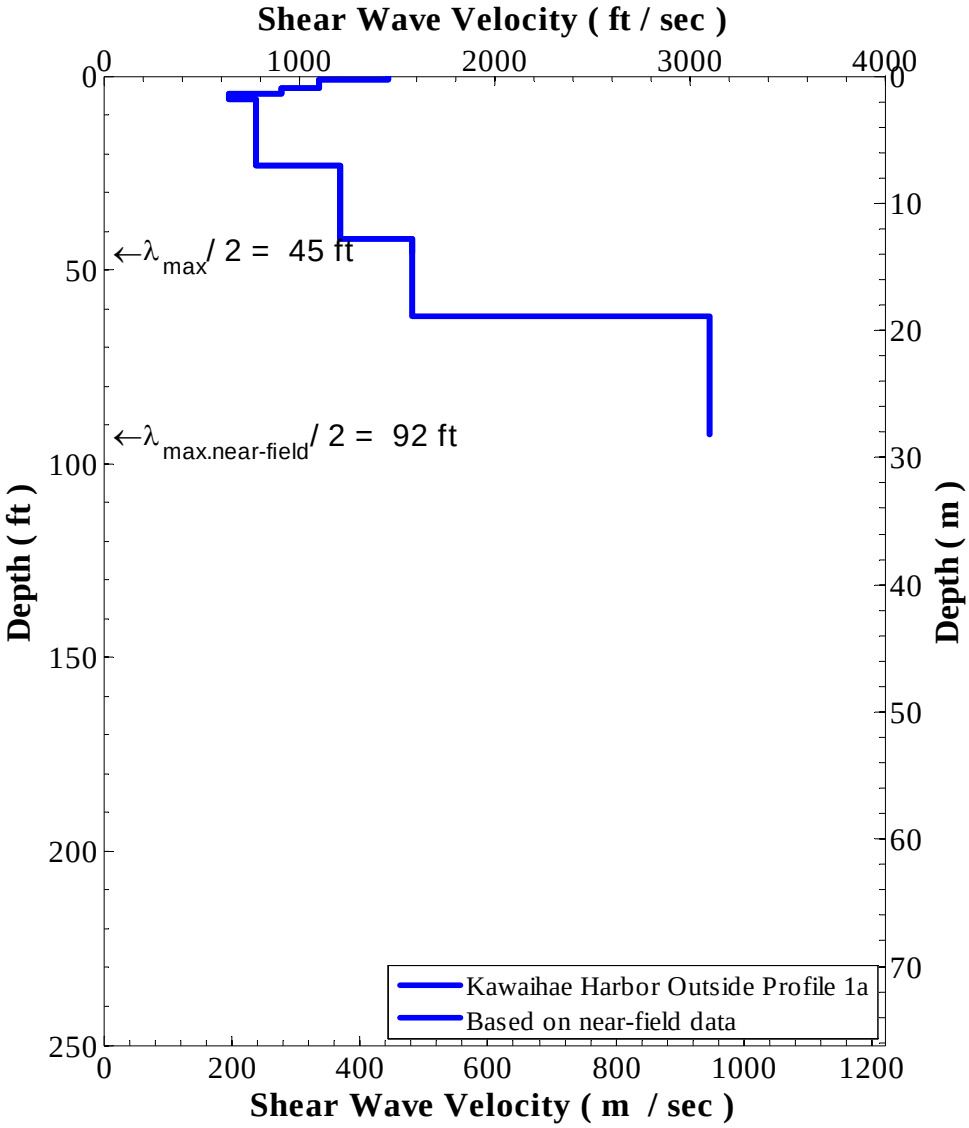
Kawaihae Harbor Array 8

Profile Parameter for Kawaihae Harbor Array 8

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	0.5	1866.1	940	0.33	130
2	1.5	1786.7	900	0.33	120
3	2	1747	880	0.33	120
4	1	1389.7	700	0.33	120
5	4	5000	520	0.4945	120
6	10	5000	810	0.4865	120
7	20	5000	900	0.4833	120
8	9999	5000	1000	0.4792	120



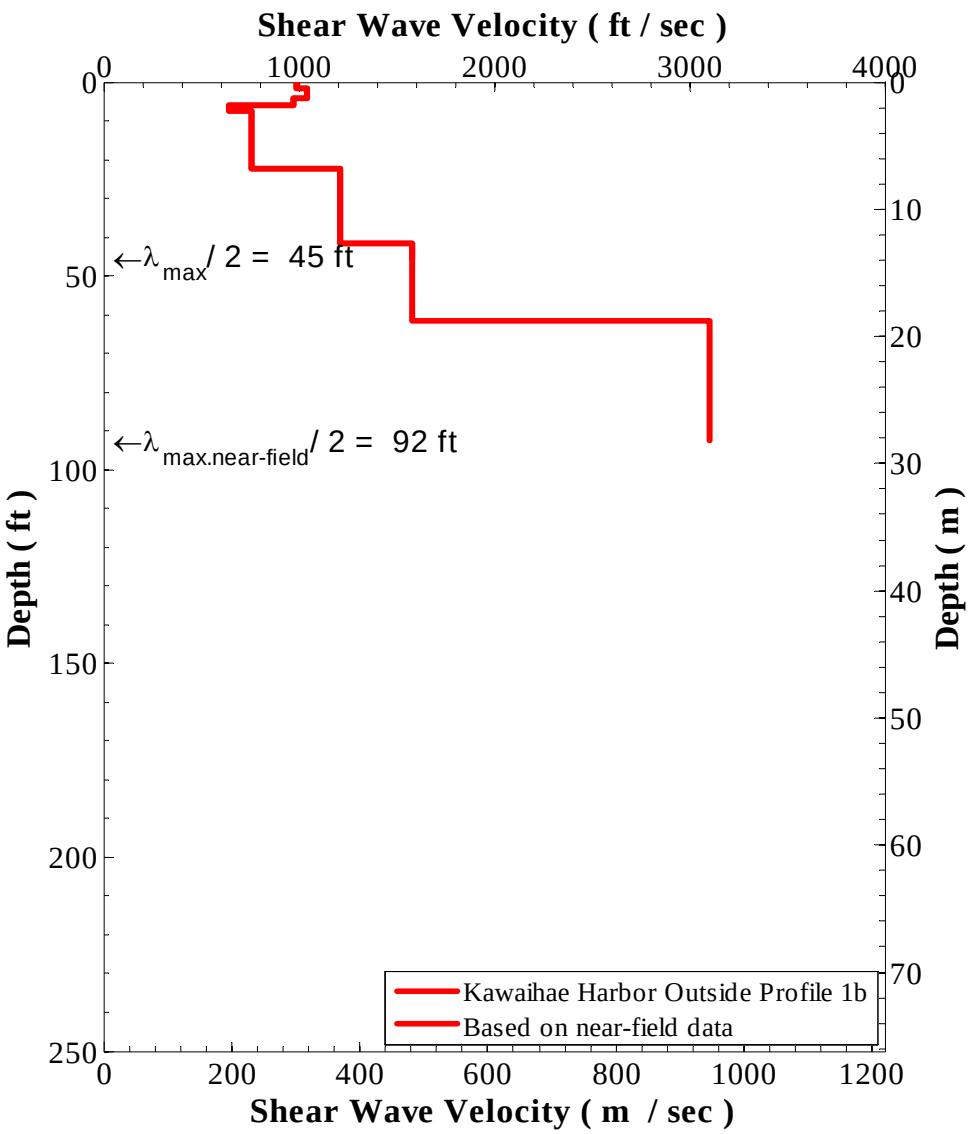
Kawaihae Harbor, Outside



Kawaihae Harbor Outside Profile 1a

Profile Parameter for Kawaihae Harbor Outside, Profile 1a

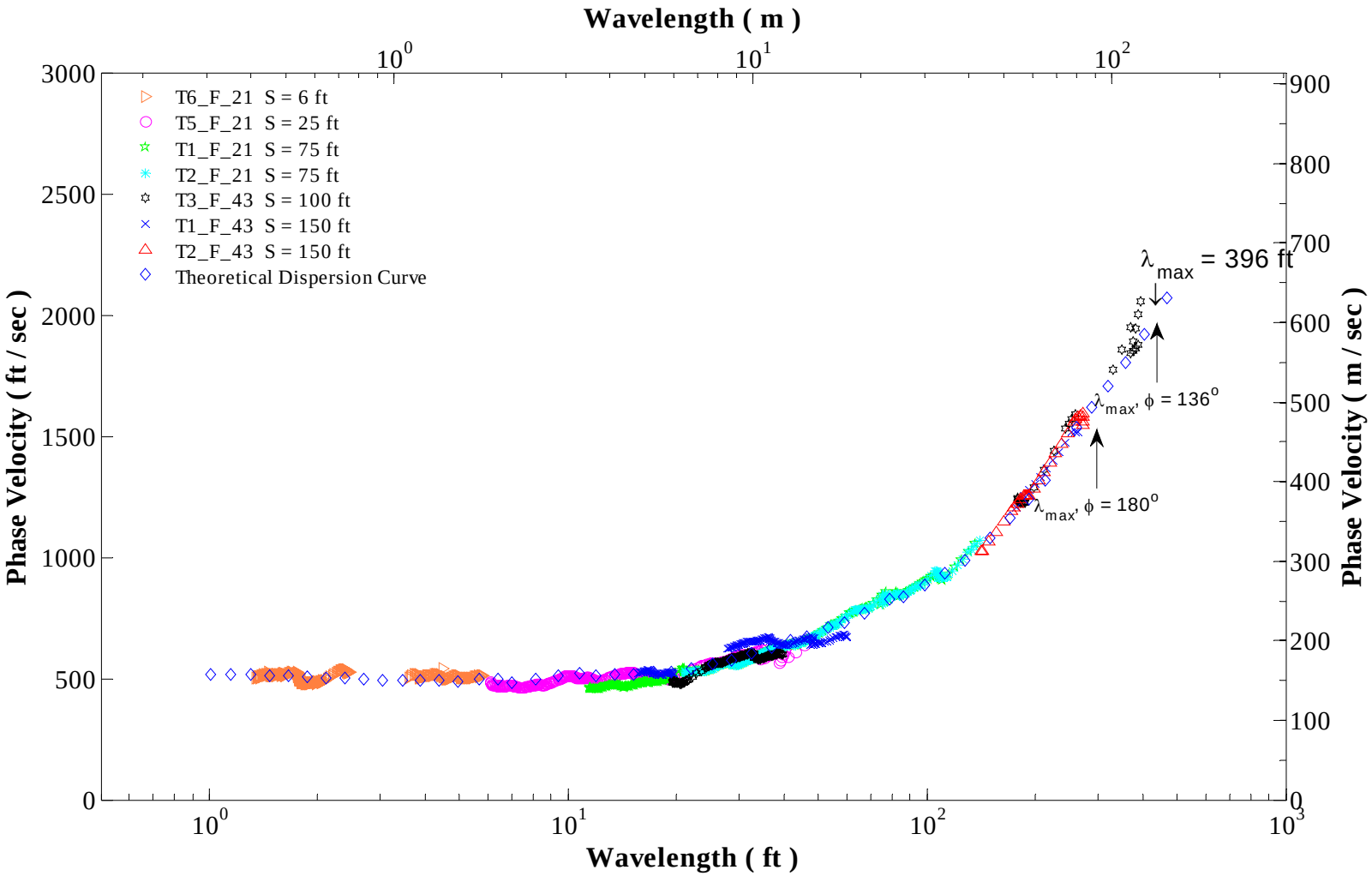
Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	1	2898.5	1460	0.33	120
2	2	2183.8	1100	0.33	120
3	1.5	1806.6	910	0.33	120
4	1.5	1270.6	640	0.33	120
5	17	5000	780	0.4875	120
6	19	5000	1210	0.4689	120
7	20	5000	1580	0.4445	120
8	99999	5000	3100	0.1878	130



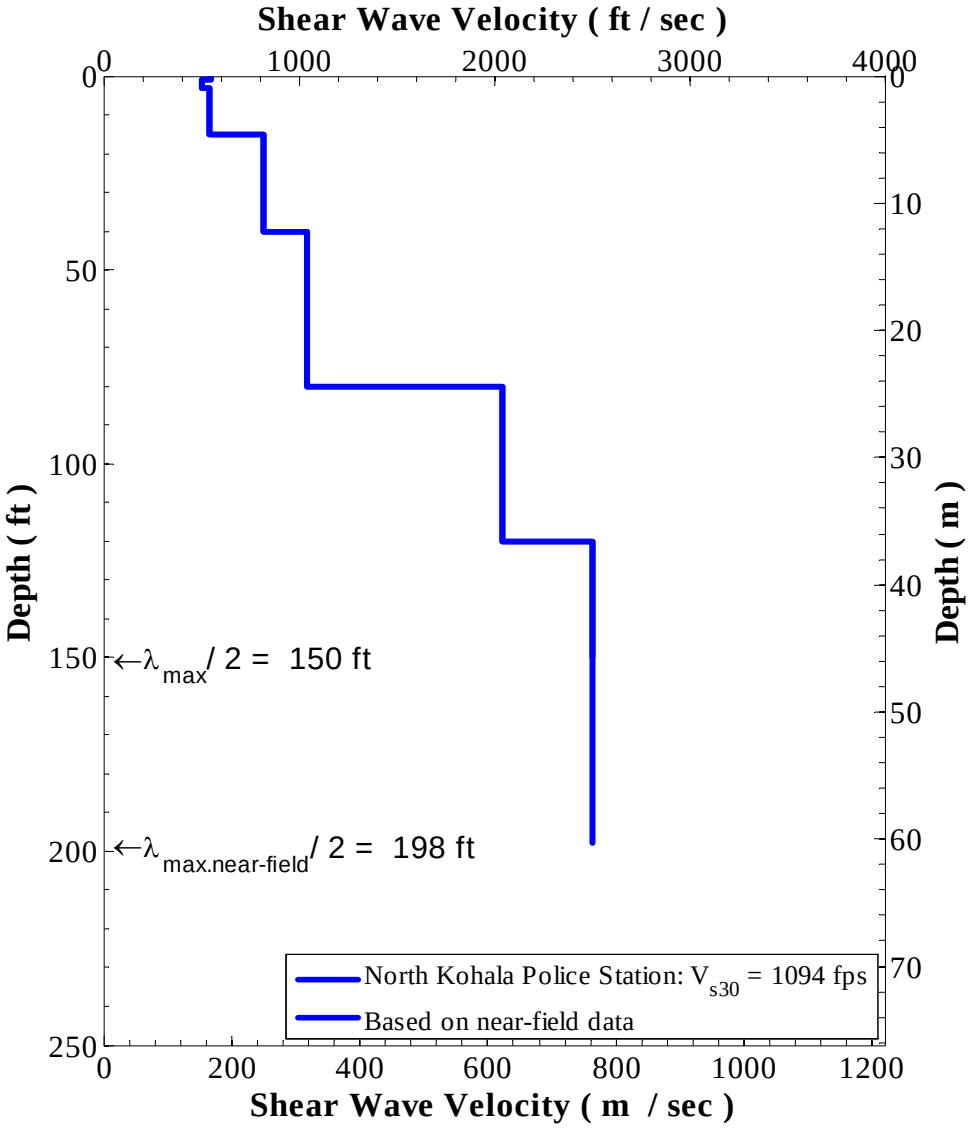
Kawaihae Harbor Outside Profile 1b

Profile Parameter for Kawaihae Harbor Outside, Profile 1b

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	1	2898.5	1460	0.33	120
2	2	2183.8	1100	0.33	120
3	1.5	1806.6	910	0.33	120
4	1.5	1270.6	640	0.33	120
5	15	5000	780	0.4875	120
6	19	5000	1210	0.4689	120
7	20	5000	1580	0.4445	120
8	99999	5000	3100	0.1878	130



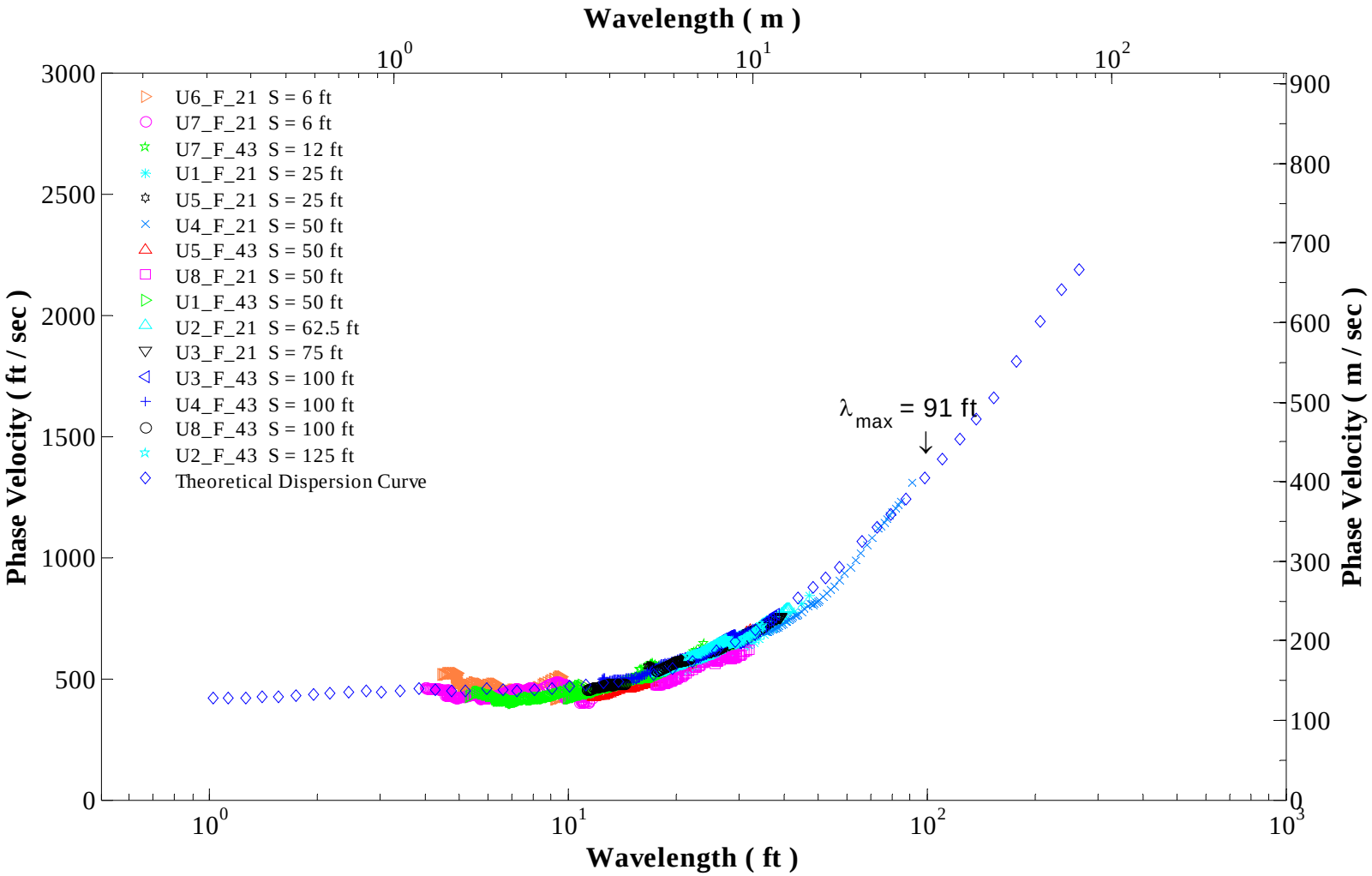
North Kohala Police Station



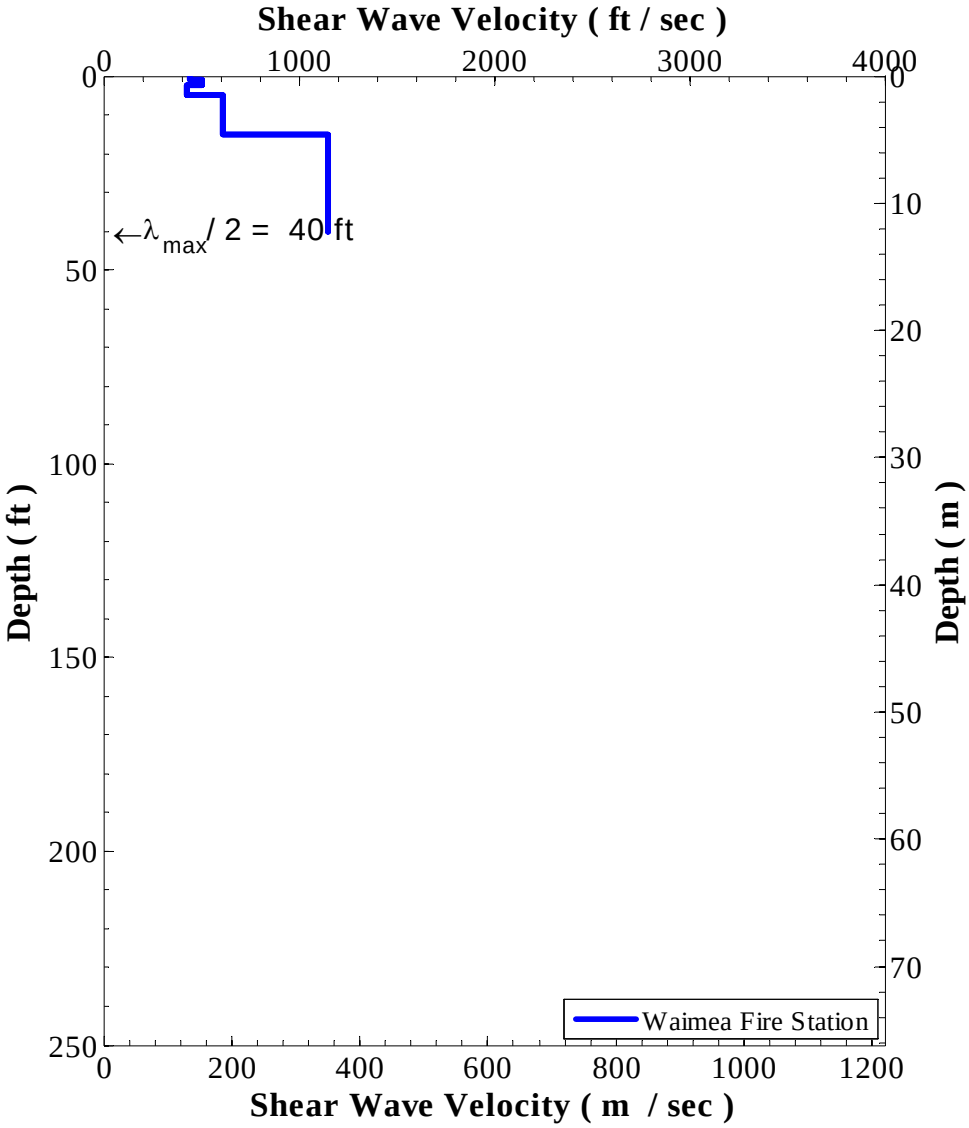
North Kohala Police Station

Profile Parameter for North Kohala Police Station

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	1	1091.9	550	0.33	120
2	2	992.6	500	0.33	120
3	3	1072	540	0.33	120
4	9	5000	540	0.4941	120
5	25	5000	820	0.4862	120
6	40	5000	1040	0.4774	120
7	40	5000	2040	0.4001	120
8	99999	5000	2500	0.3333	130



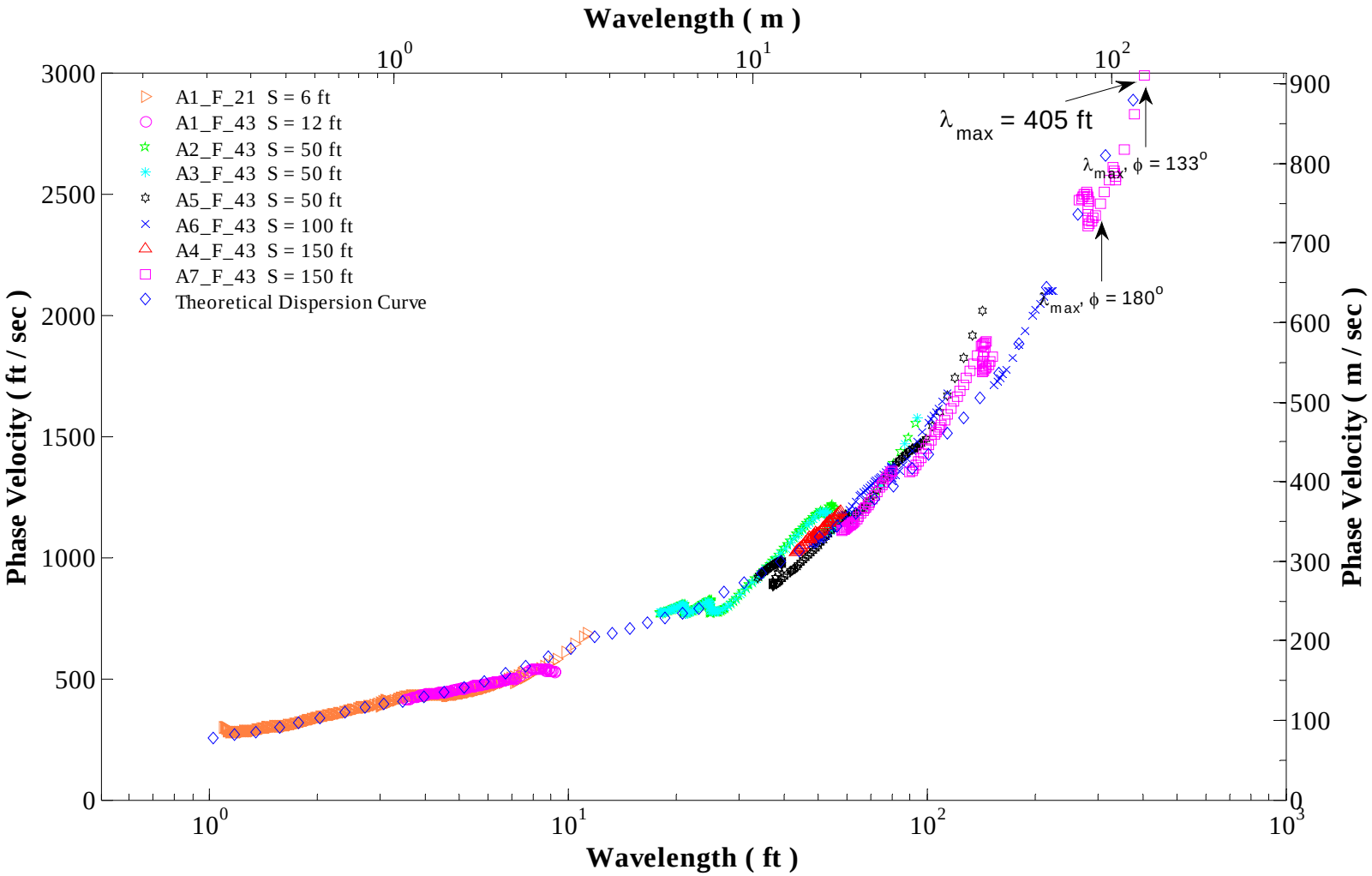
Waimea Fire Station



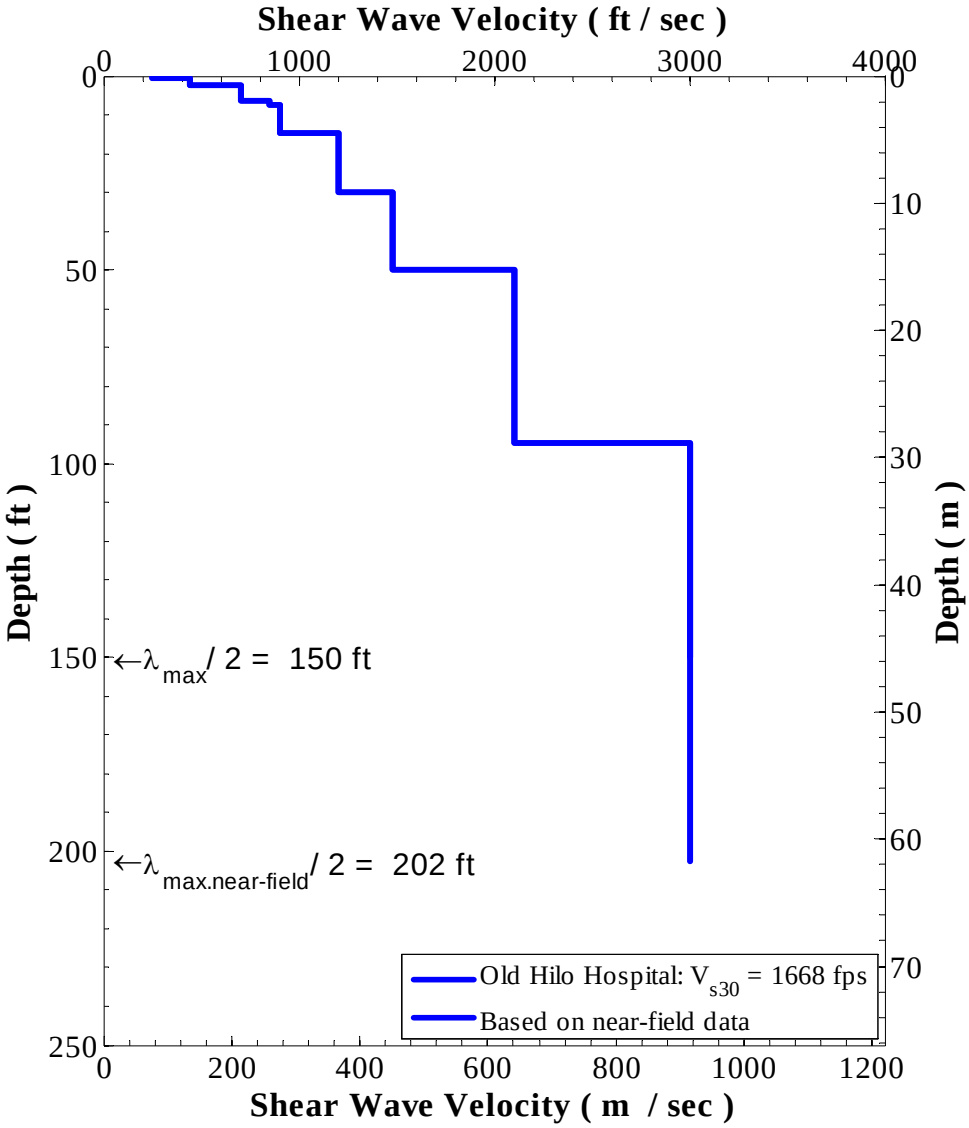
Waimea Fire Station

Profile Parameter for Waimea Fire Station

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	1	873.5	440	0.33	120
2	1.5	992.6	500	0.33	120
3	2.5	5000	430	0.4963	120
4	10	5000	610	0.4924	120
5	25	5000	1150	0.4721	120
6	99999	5000	2000	0.25	130



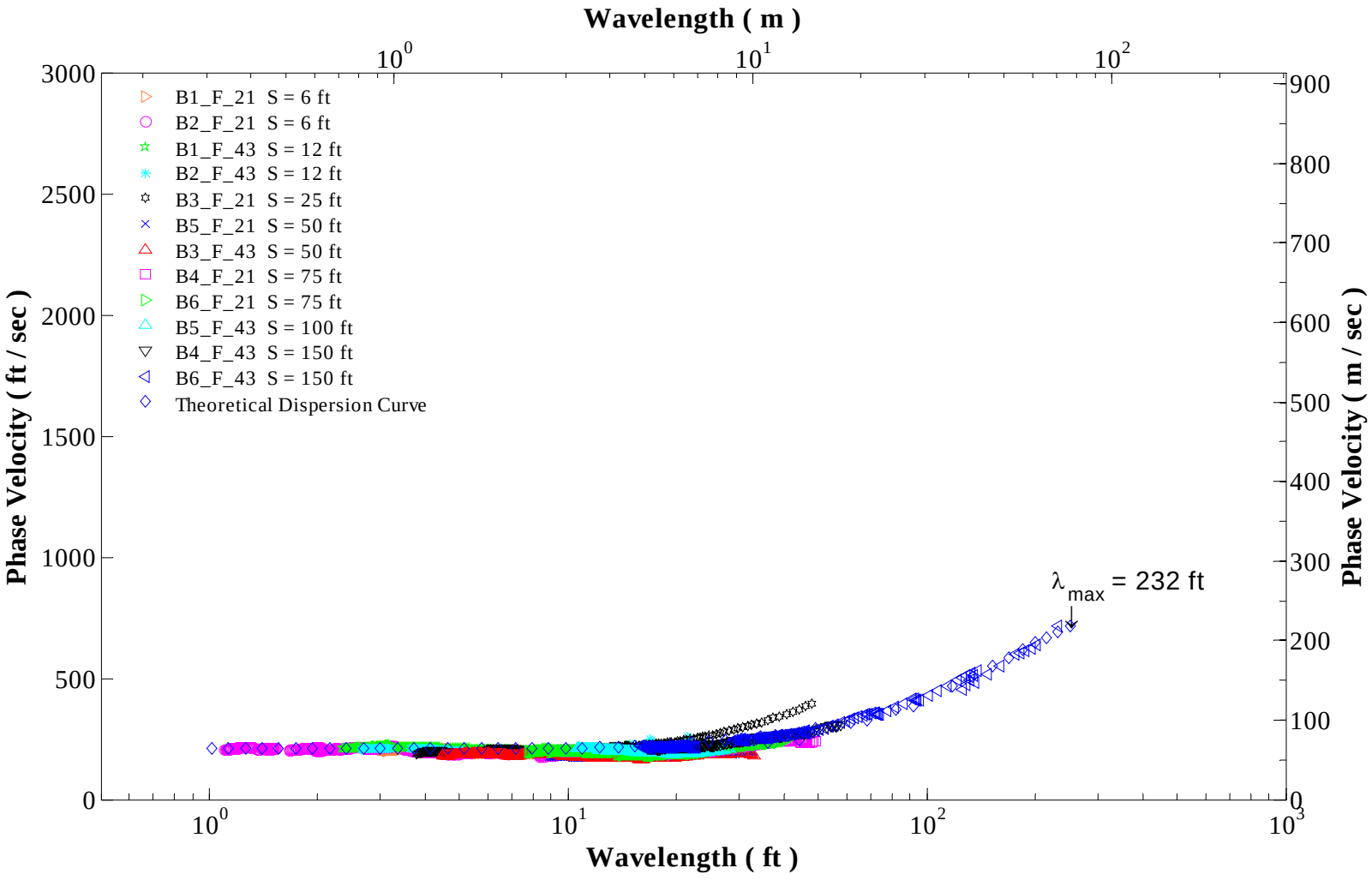
Old Hilo Hospital



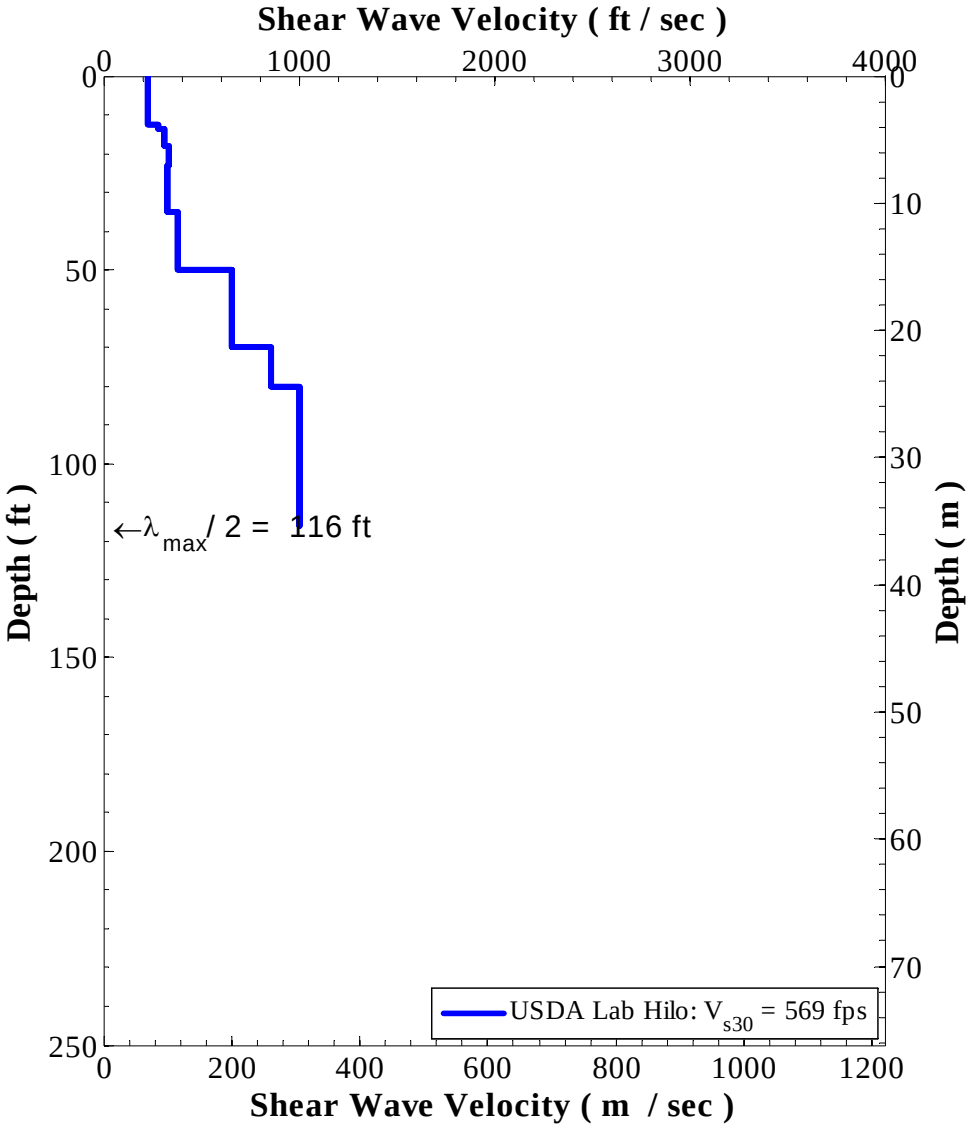
Old Hilo Hospital

Profile Parameter for Old Hilo Hospital

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	0.6	496.3	250	0.33	120
2	1.7	873.5	440	0.33	120
3	4	1389.7	700	0.33	120
4	1	5000	850	0.4833	120
5	7.5	5000	900	0.4833	120
6	15	5000	1200	0.4746	120
7	20	5000	1480	0.452	120
8	45	5000	2100	0.3929	130
9	99999	5000	3000	0.2188	130



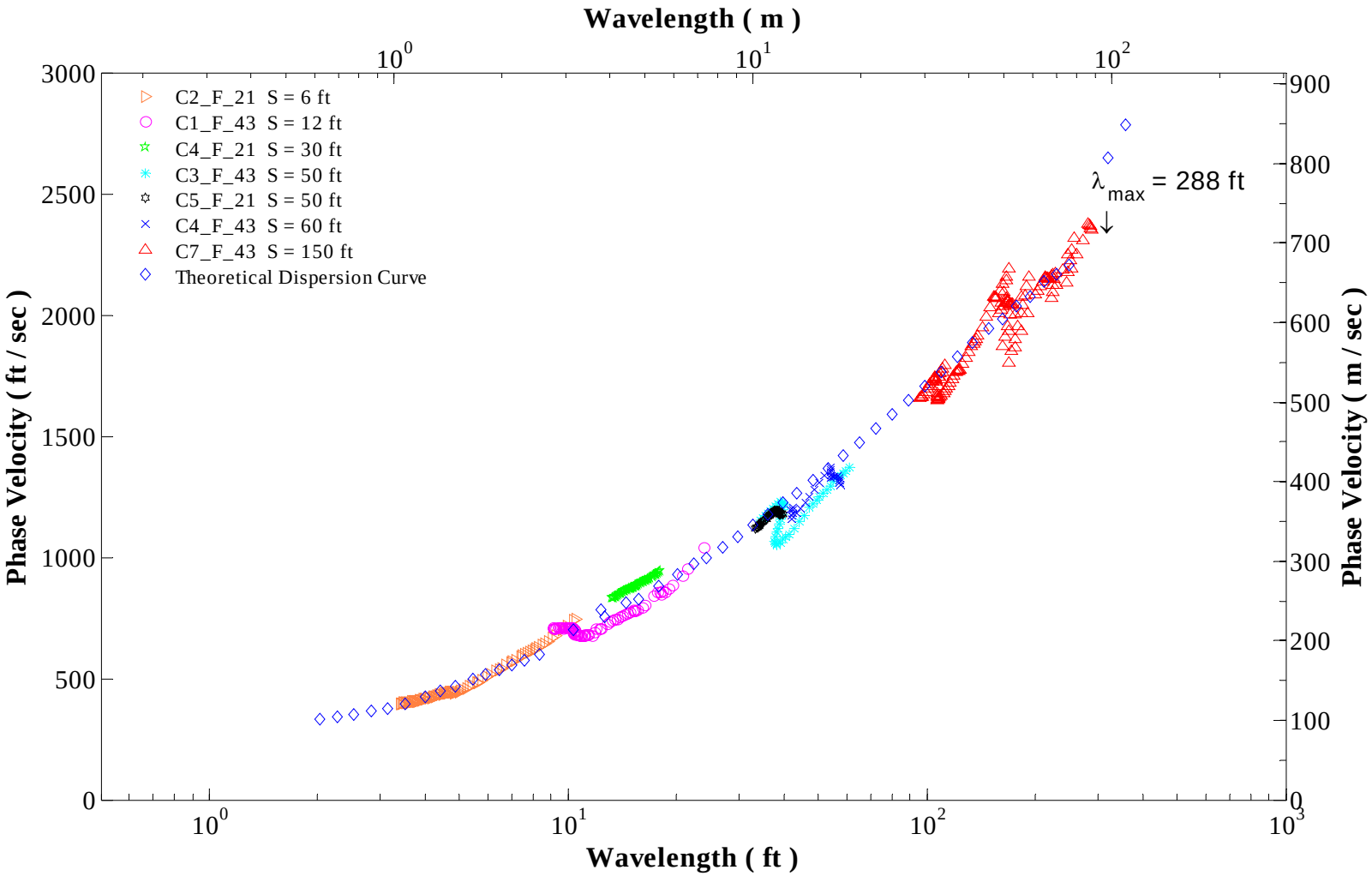
USDA Lab Hilo



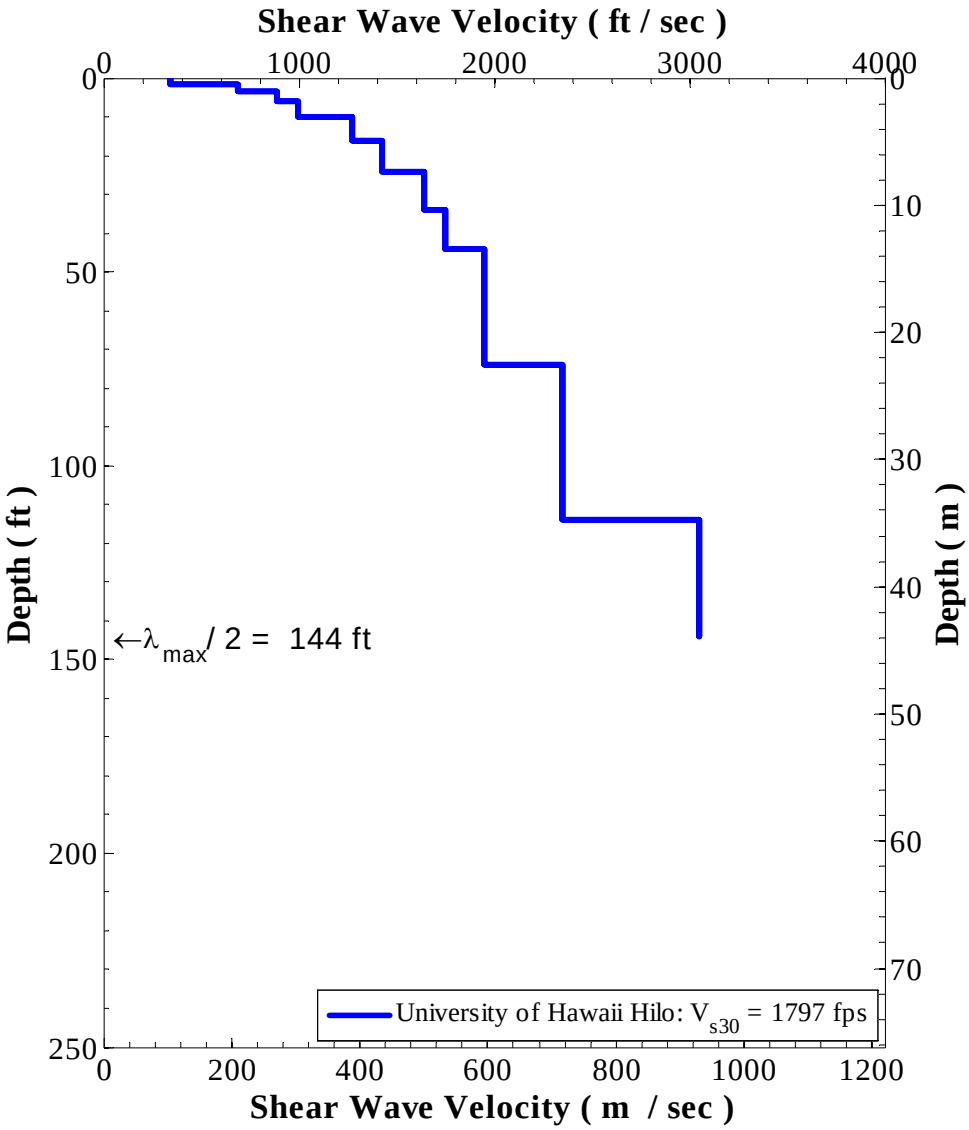
USDA Lab Hilo

Profile Parameter for USDA Lab Hilo

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	6	476.5	225	0.33	120
2	6.5	5000	225	0.4989	120
3	1	5000	280	0.4984	120
4	4.5	5000	310	0.4981	120
5	5	5000	335	0.4977	120
6	12	5000	330	0.4978	120
7	15	5000	380	0.4971	120
8	20	5000	660	0.4911	120
9	10	5000	860	0.4809	120
10	99999	5000	1000	0.4792	130



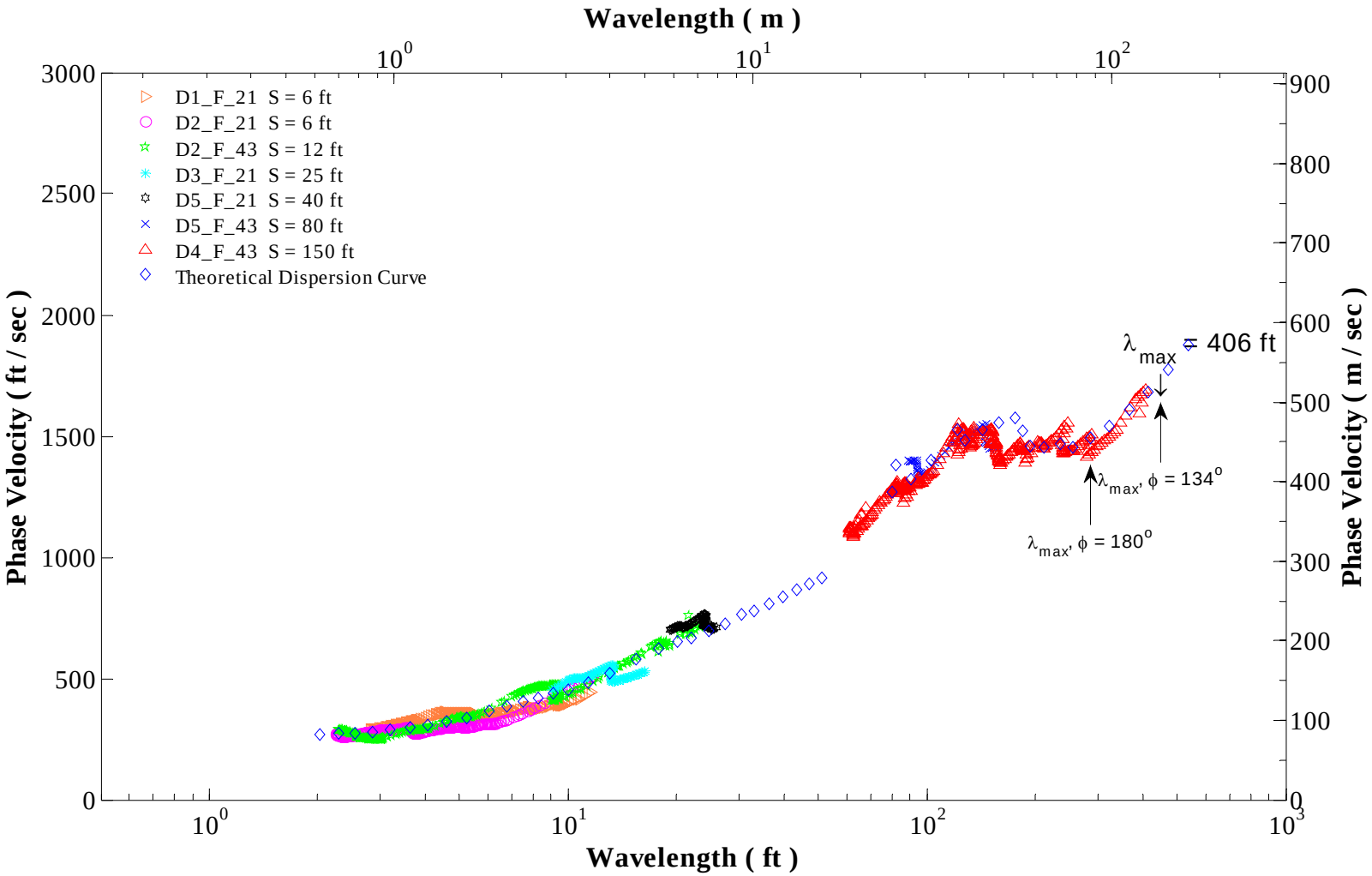
University of Hawaii Hilo



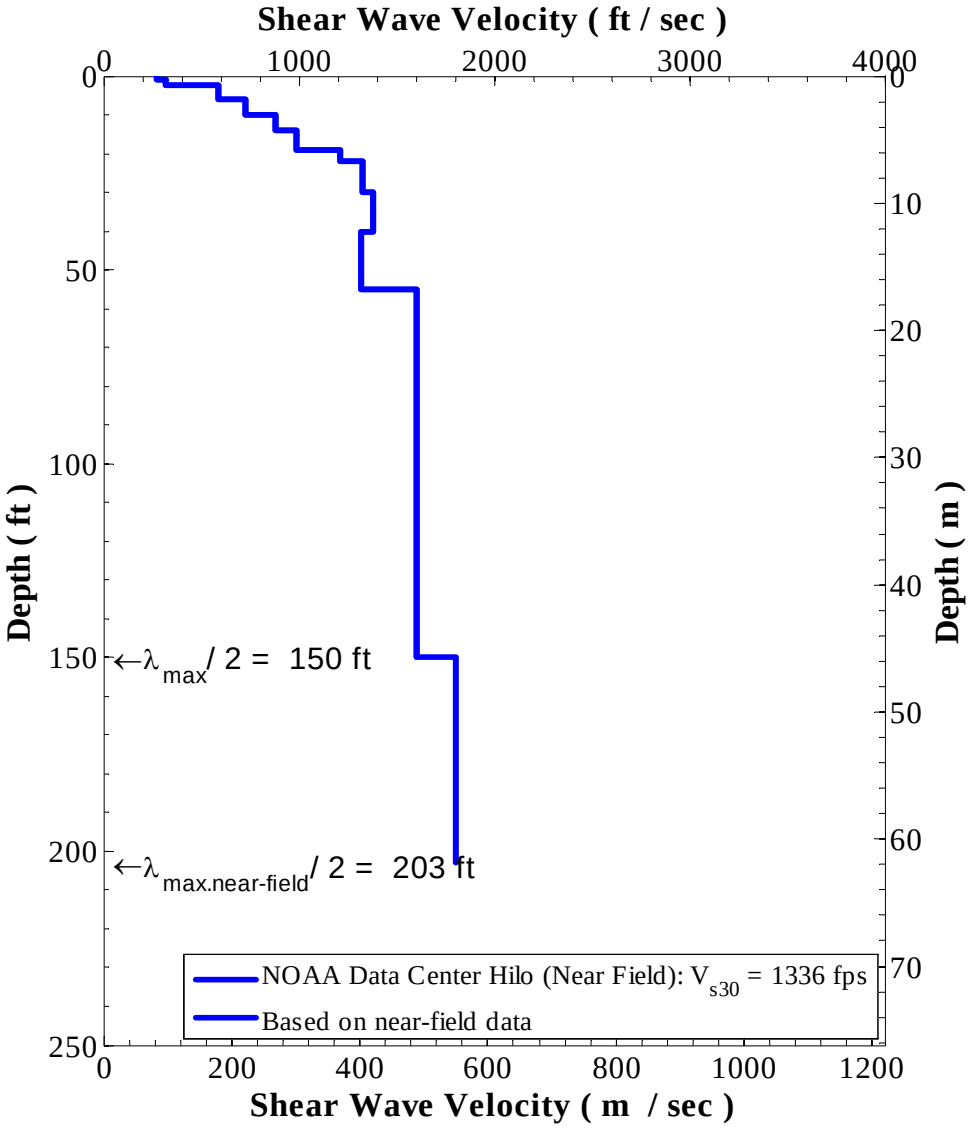
University of Hawaii Hilo

Profile Parameter for University of Hawaii Hilo

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	1.5	615.4	340	0.33	120
2	2	1369.8	690	0.33	120
3	2.5	1747	890	0.33	120
4	4	5000	999	0.4792	120
5	6	5000	1270	0.4917	120
6	8	5000	1430	0.484	120
7	10	5000	1640	0.4689	120
8	10	5000	1750	0.4389	120
9	30	5000	1950	0.4103	120
10	40	5000	2350	0.3582	120
11	99999	5000	3050	0.2037	130



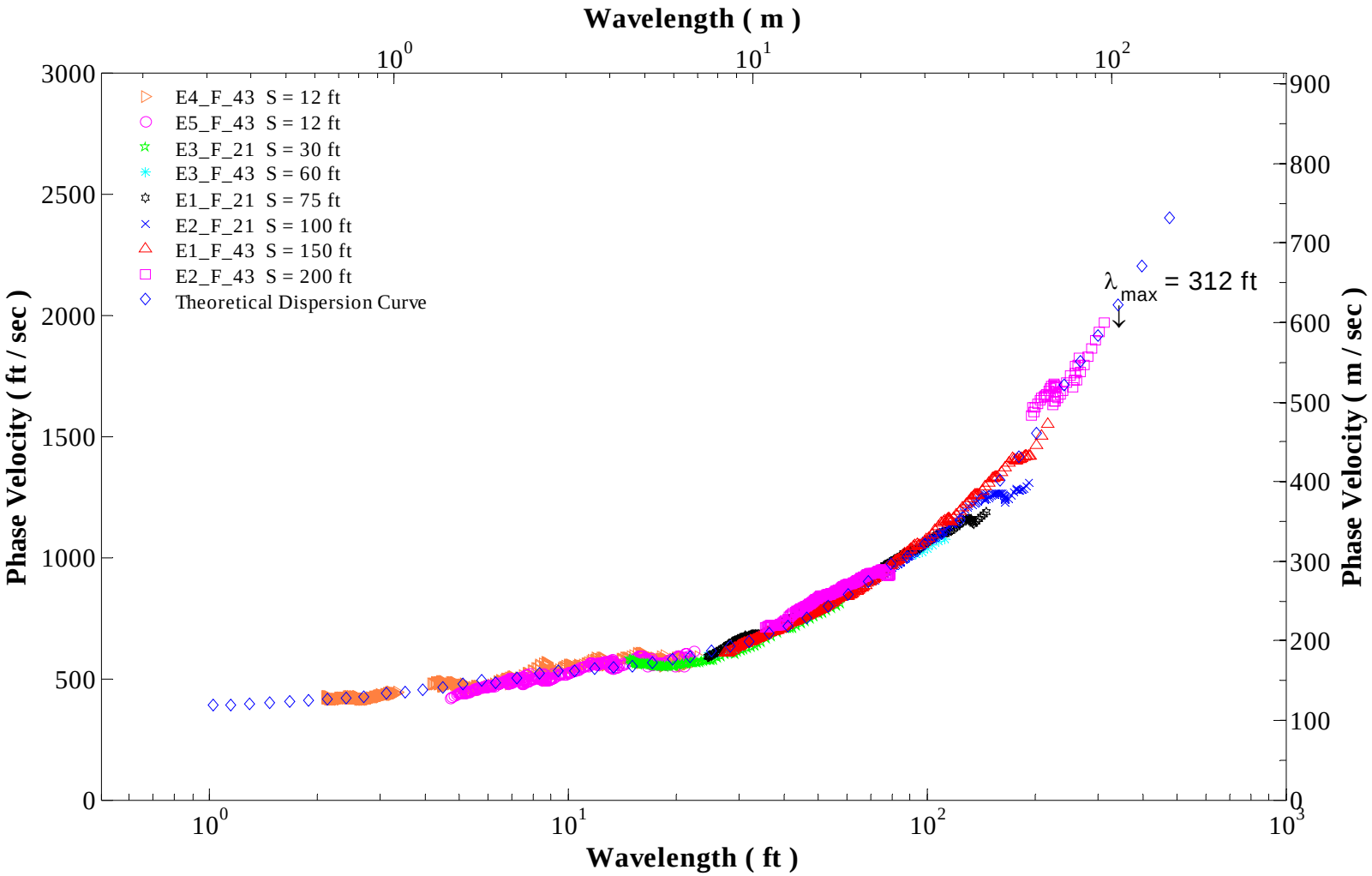
NOAA Data Center Hilo



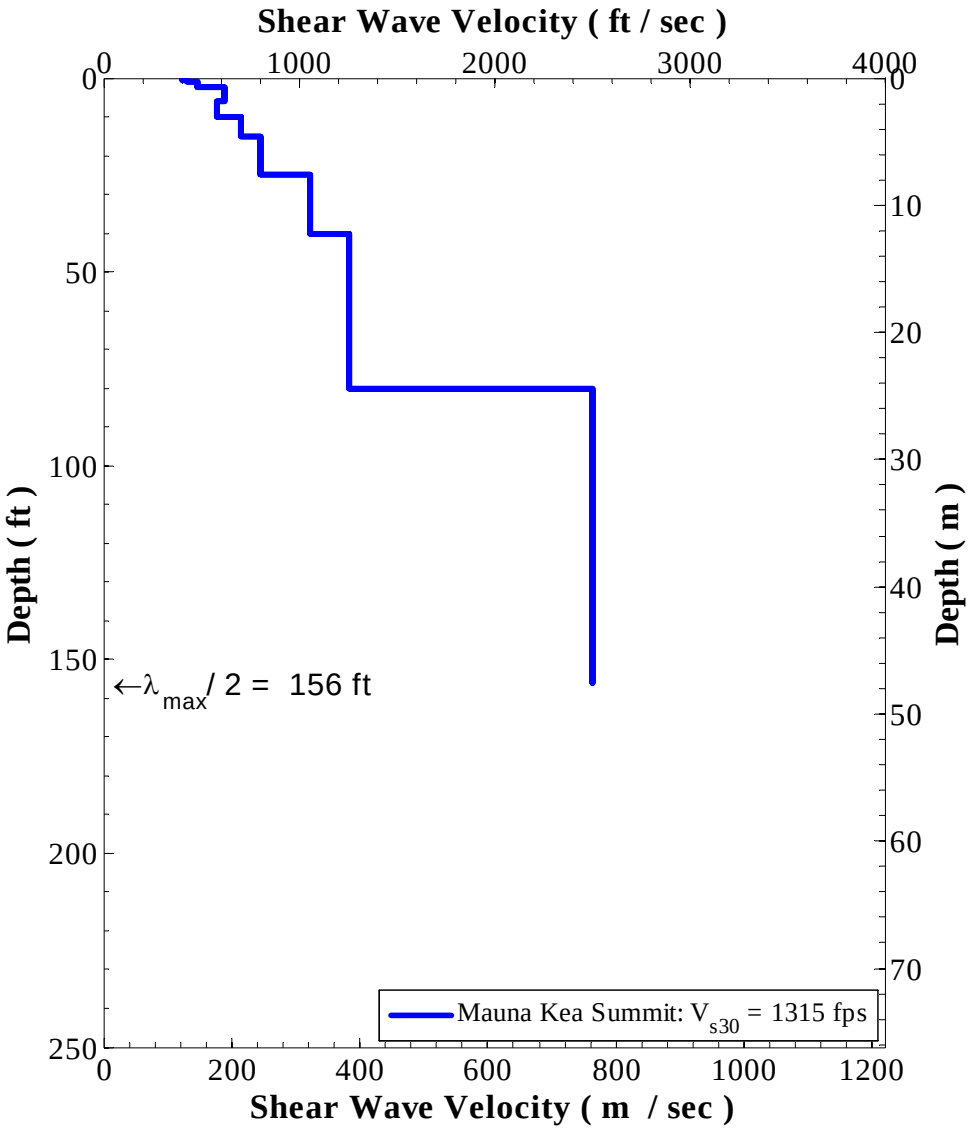
NOAA Data Center Hilo (Near Field)

Profile Parameter for NOAA Data Center Hilo

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	1	536	270	0.33	120
2	1.5	635.3	320	0.33	120
3	3.5	1171.3	590	0.33	120
4	4	1449.2	730	0.33	120
5	10	5000	999	0.48	120
6	10	5000	1150	0.4721	120
7	10	5000	1320	0.4625	120
8	10	5000	1900	0.4156	120
9	40	5000	1300	0.4637	120
10	160	5000	1700	0.4346	130
11	99999	5000	3000	0.2188	130



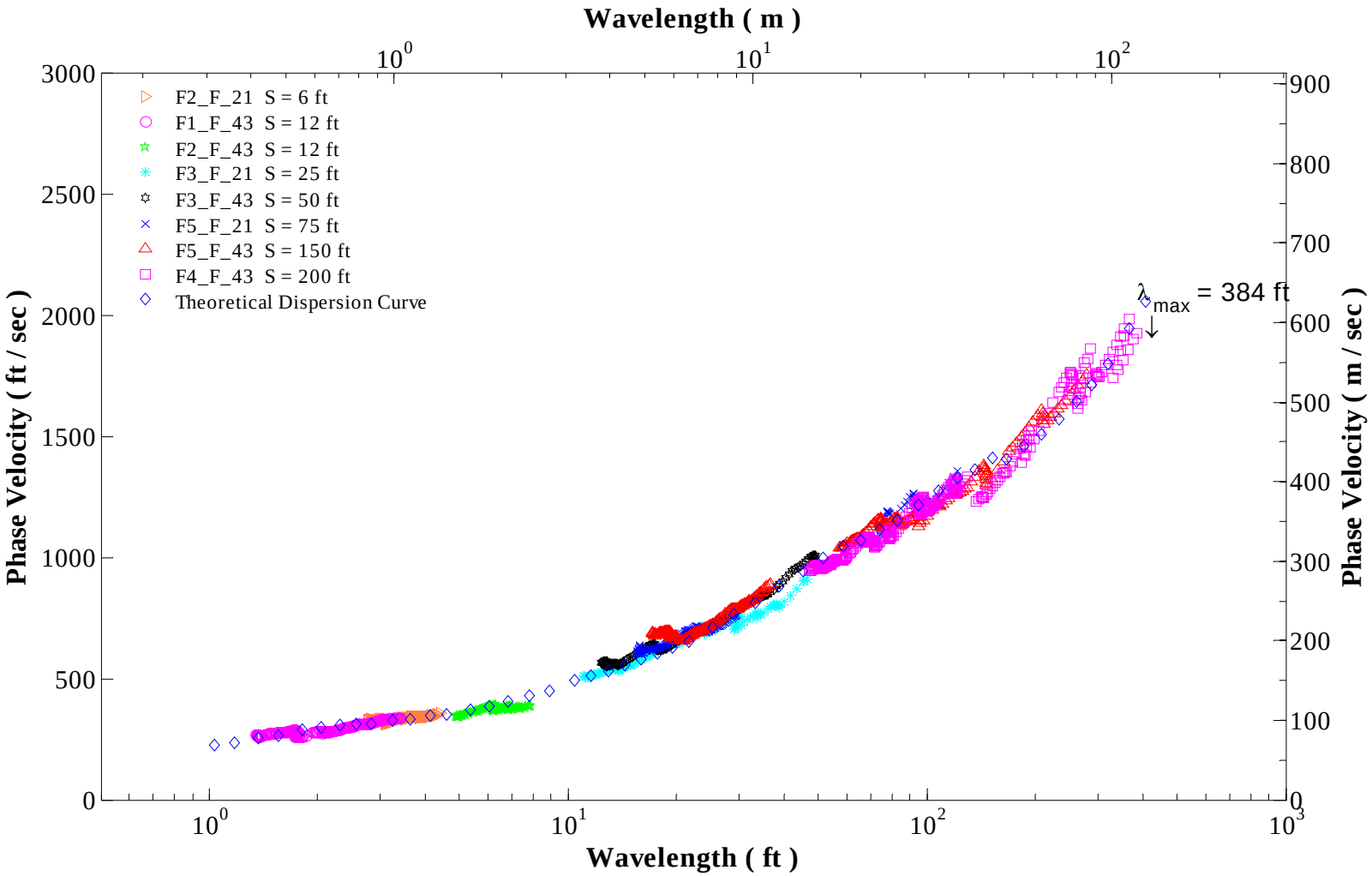
Mauna Kea Summit



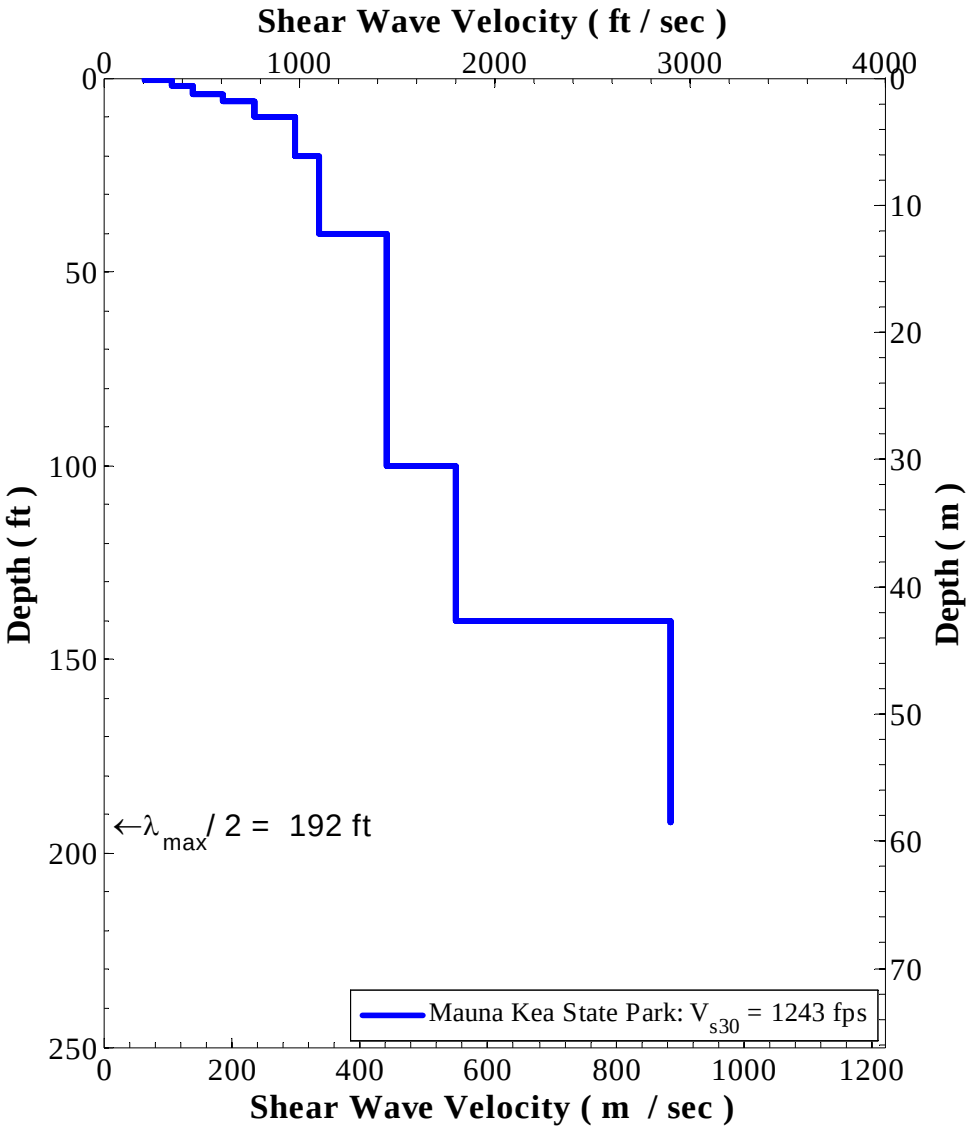
Mauna Kea Summit

Profile Parameter for Mauna Kea Summit

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	0.5	794.1	400	0.33	120
2	0.5	853.7	430	0.33	120
3	1.5	952.9	480	0.33	120
4	3.5	1230.9	620	0.33	120
5	4	5000	580	0.4932	120
6	5	5000	700	0.49	120
7	10	5000	800	0.49	120
8	15	5000	1060	0.4765	120
9	40	5000	1260	0.4661	120
10	99999	5000	2500	0.3333	130



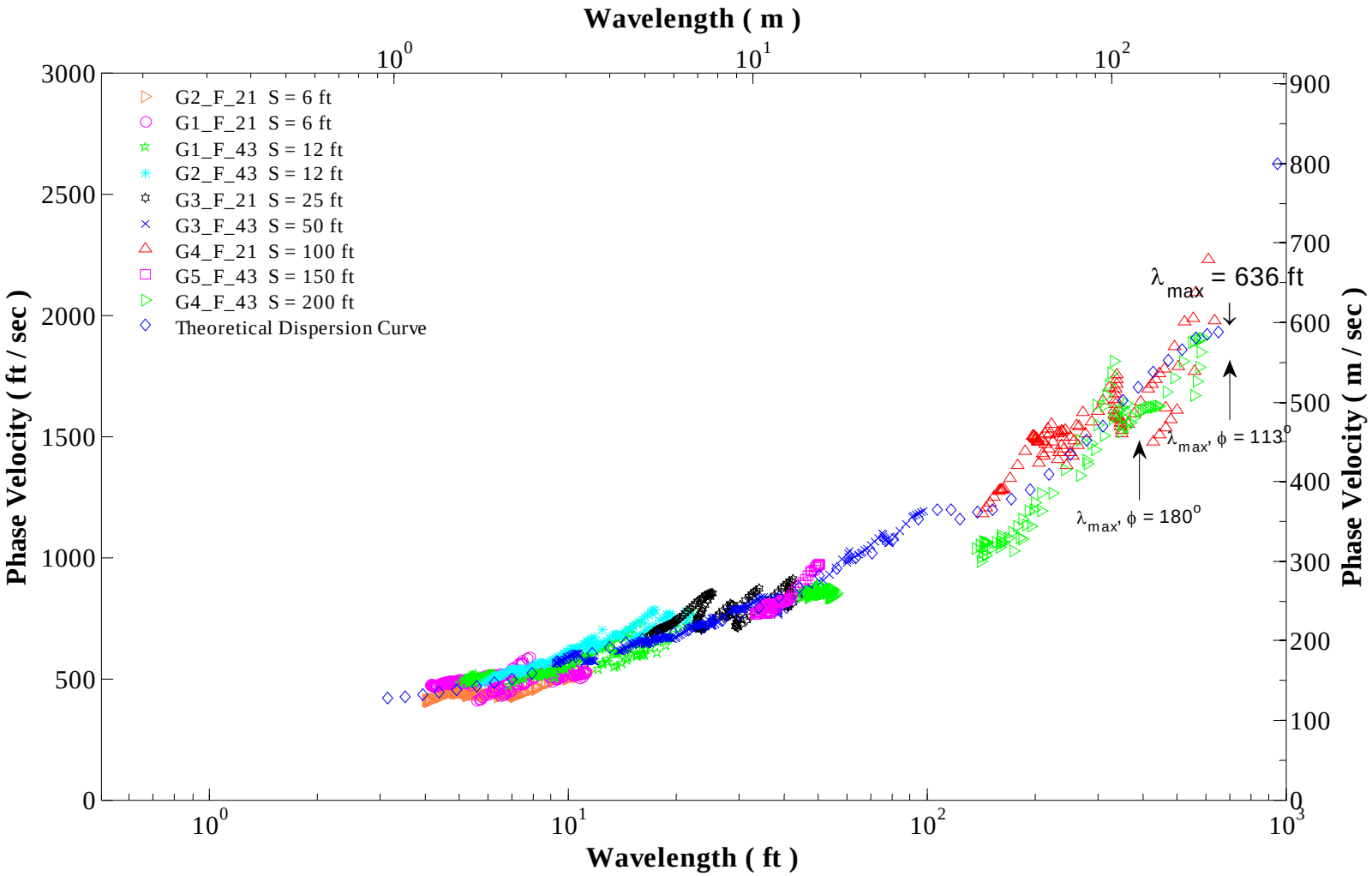
Mauna Kea State Park



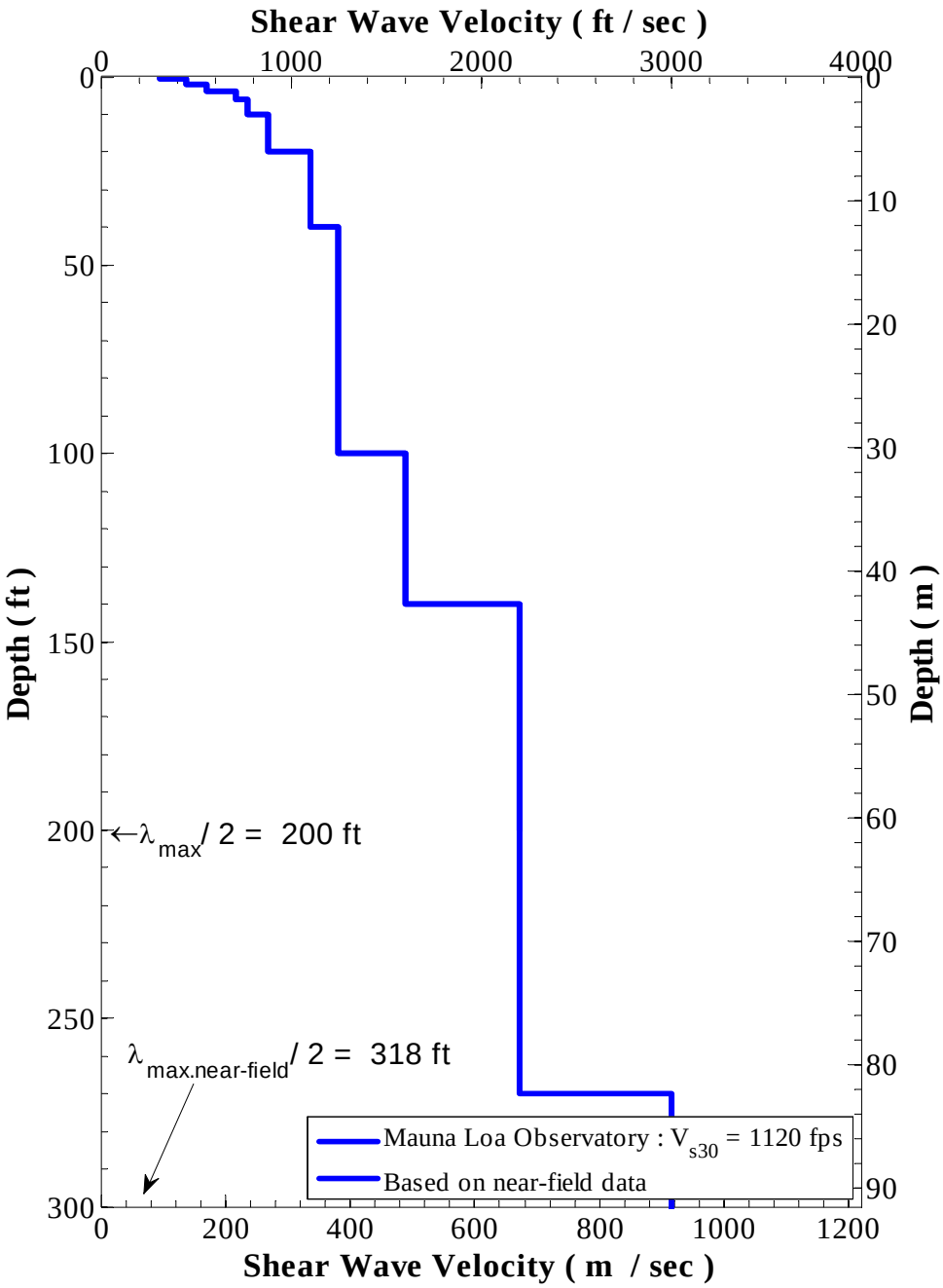
Mauna Kea State Park

Profile Parameter for Mauna Kea State Park

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	0.5	416.9	210	0.33	120
2	1.5	694.8	350	0.33	120
3	2	913.2	460	0.33	120
4	2	1211	610	0.33	120
5	4	5000	770	0.4879	120
6	10	5000	980	0.48	120
7	20	5000	1100	0.4746	120
8	60	5000	1450	0.4541	120
9	40	5000	1800	0.4256	120
10	99999	5000	2900	0.2465	130



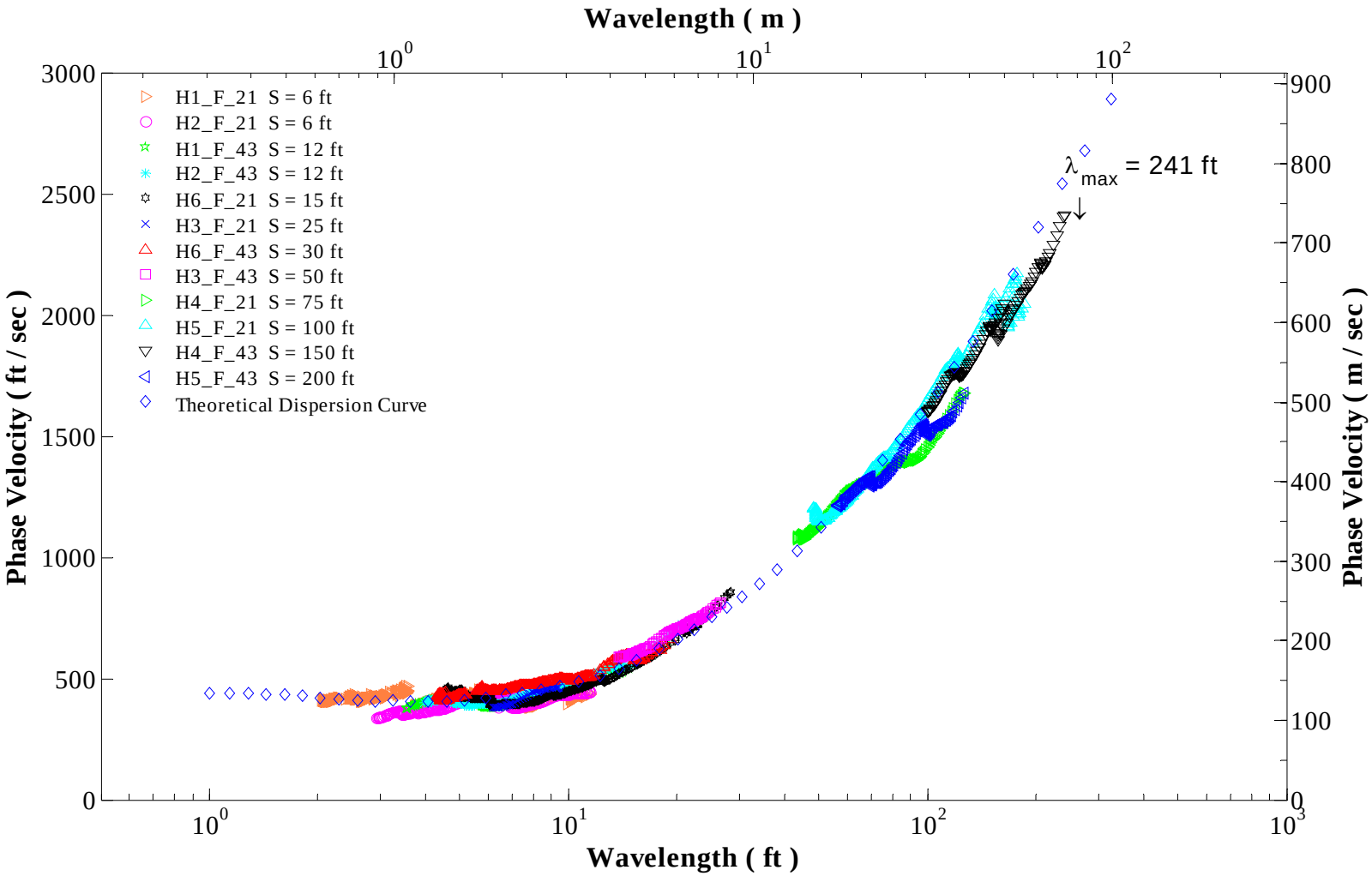
Mauna Loa Observatory



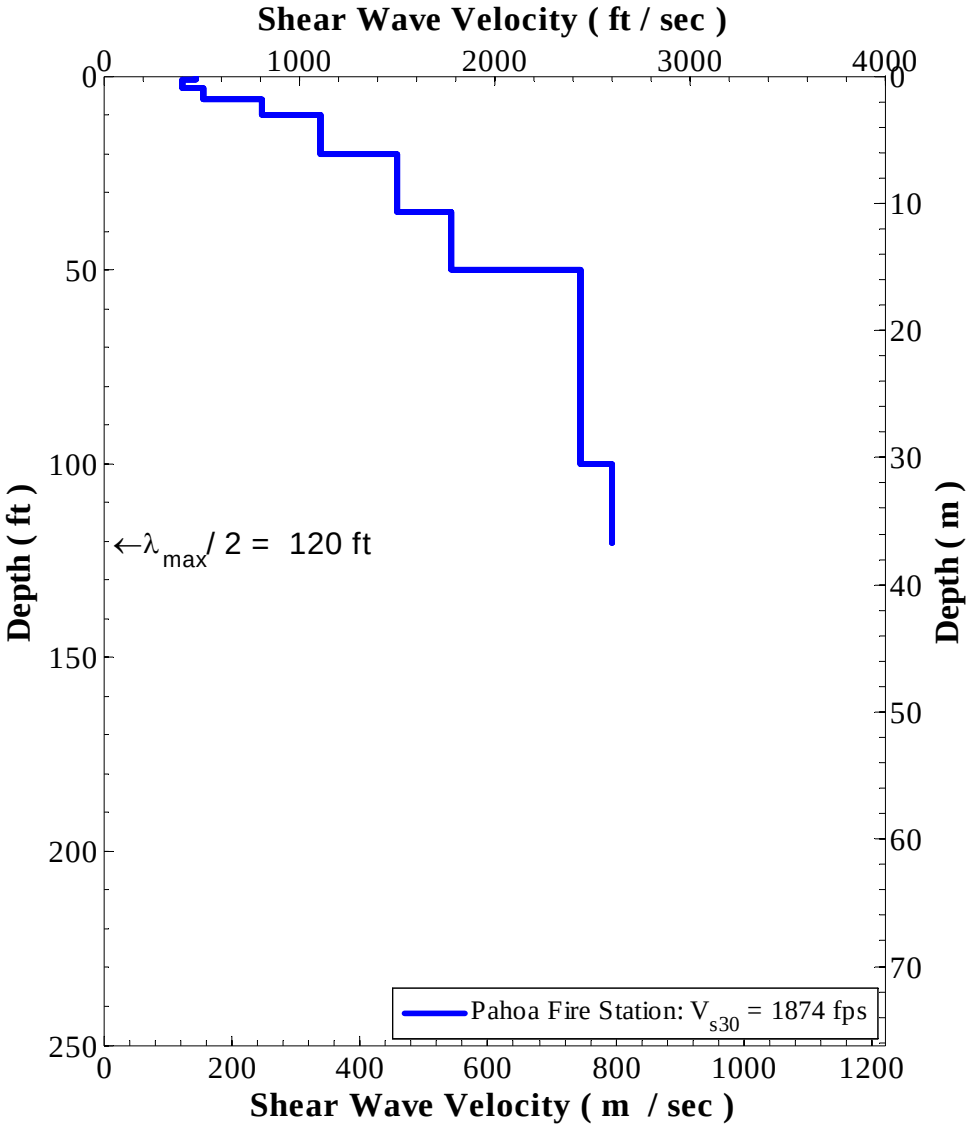
Mauna Loa Observatory

Profile Parameter for Mauna Loa Observatory

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	0.5	416.9	310	0.33	120
2	1.5	694.8	450	0.33	120
3	2	913.2	560	0.33	120
4	2	1211	710	0.33	120
5	4	5000	770	0.4879	120
6	10	5000	880	0.48	120
7	20	5000	1100	0.4746	120
8	60	5000	1250	0.4667	120
9	40	5000	1600	0.4505	120
10	130	5000	2200	0.38	120
11	99999	5000	3000	0.38	130



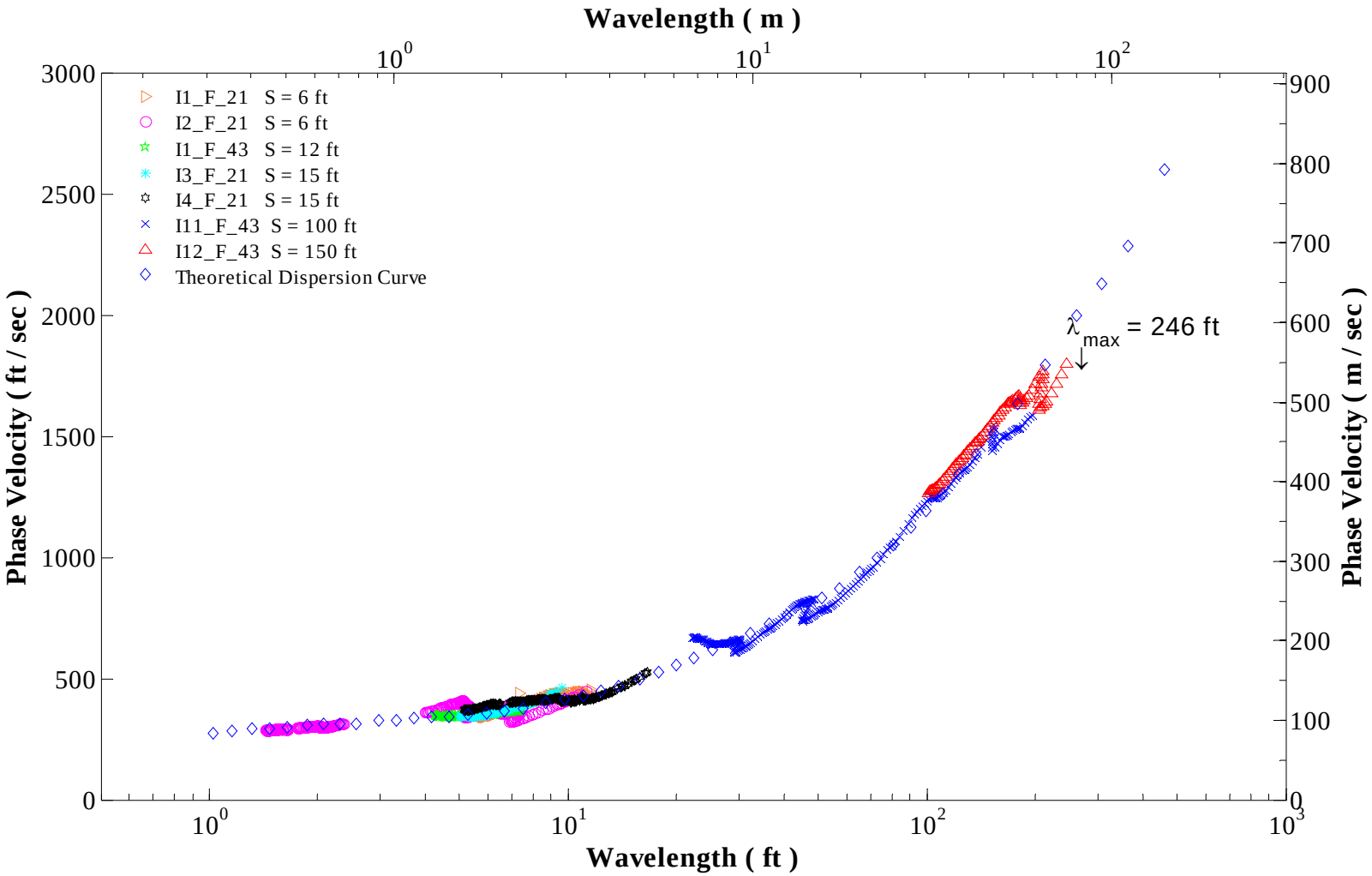
Pahoa Fire Station



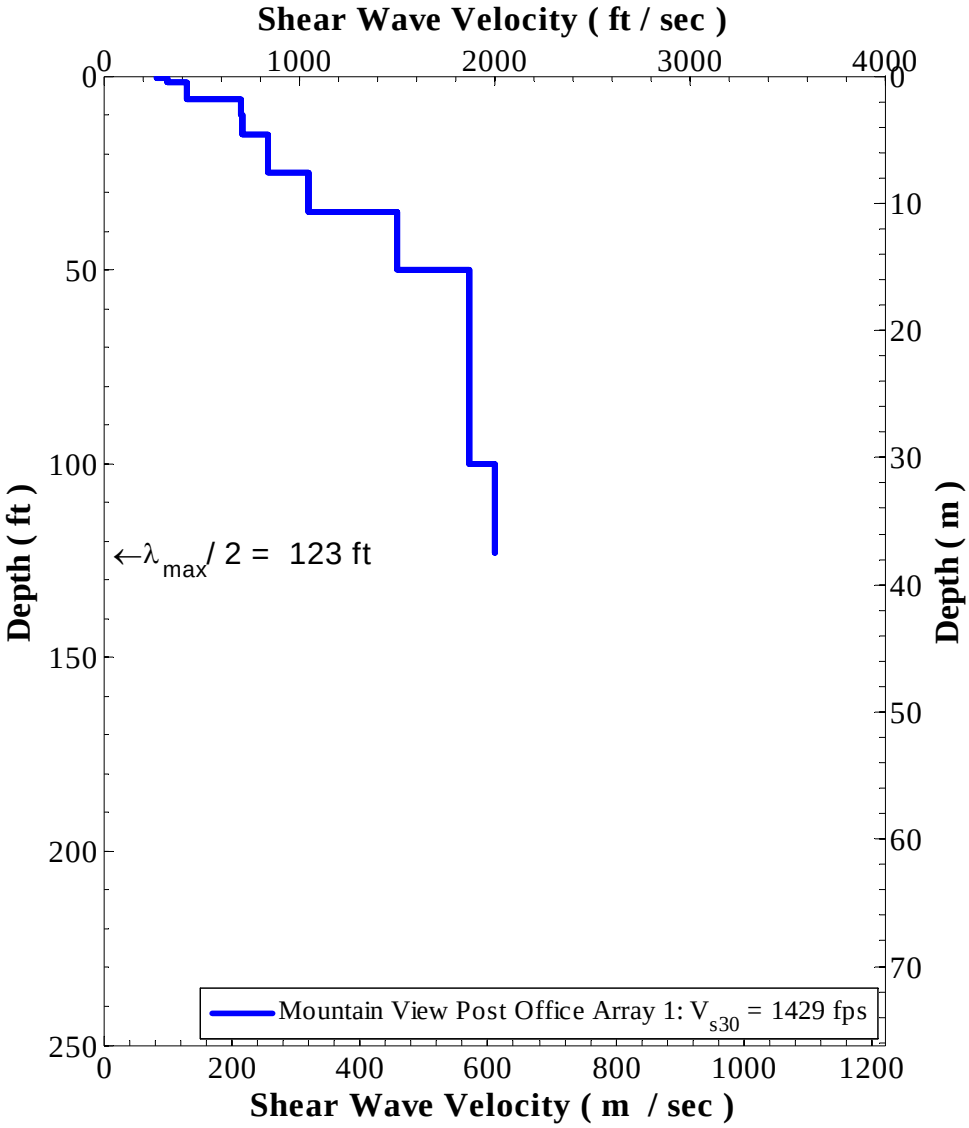
Pahoa Fire Station

Profile Parameter for Pahoa Fire Station

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	1	933.1	470	0.33	120
2	2	794.1	400	0.33	120
3	3	1012.5	510	0.33	120
4	4	5000	810	0.4865	120
5	10	5000	1110	0.4741	120
6	15	5000	1500	0.4505	120
7	15	5000	1780	0.4274	120
8	50	5000	2440	0.3437	130
9	99999	5000	2600	0.3147	130



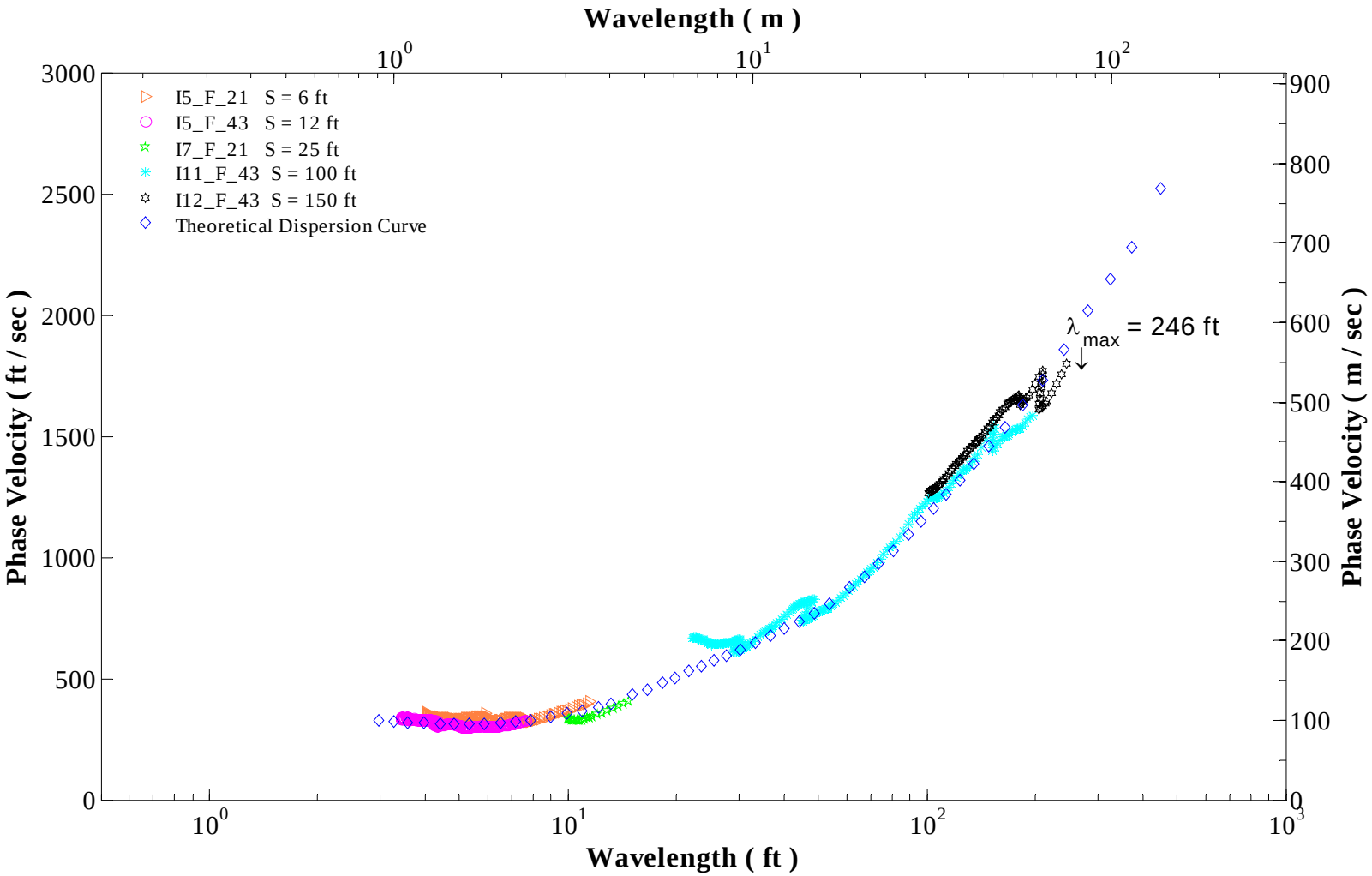
Mountain View Post Office (Array 1)



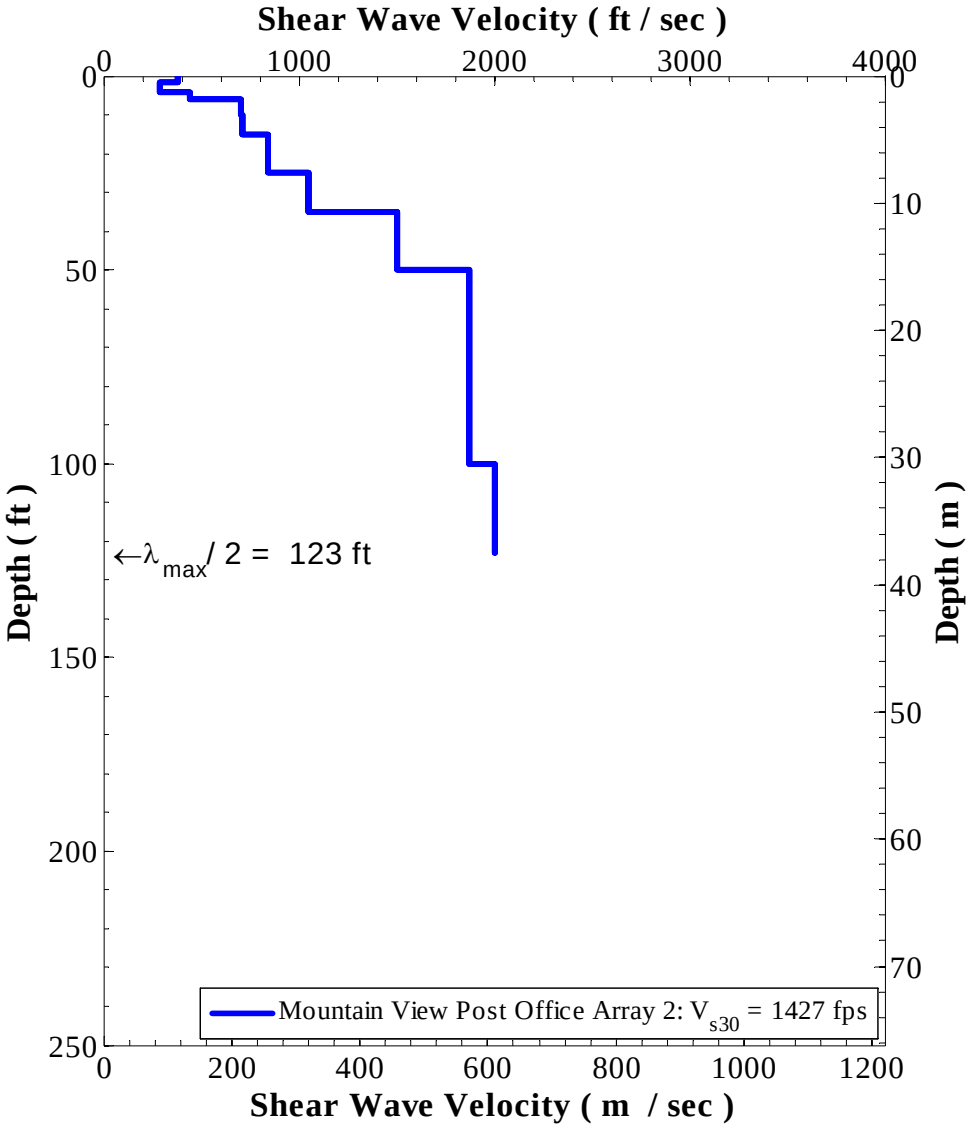
Mountain View Post Office Array 1

Profile Parameter for Mountain View Post Office Array 1

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	0.5	536	270	0.33	120
2	1	655.1	330	0.33	120
3	4.5	853.7	430	0.33	120
4	4	5000	700	0.49	120
5	5	5000	710	0.4897	120
6	10	5000	840	0.4862	120
7	10	5000	1050	0.4769	120
8	15	5000	1500	0.4505	120
9	50	5000	1870	0.4187	120
10	99999	5000	2000	0.4048	130



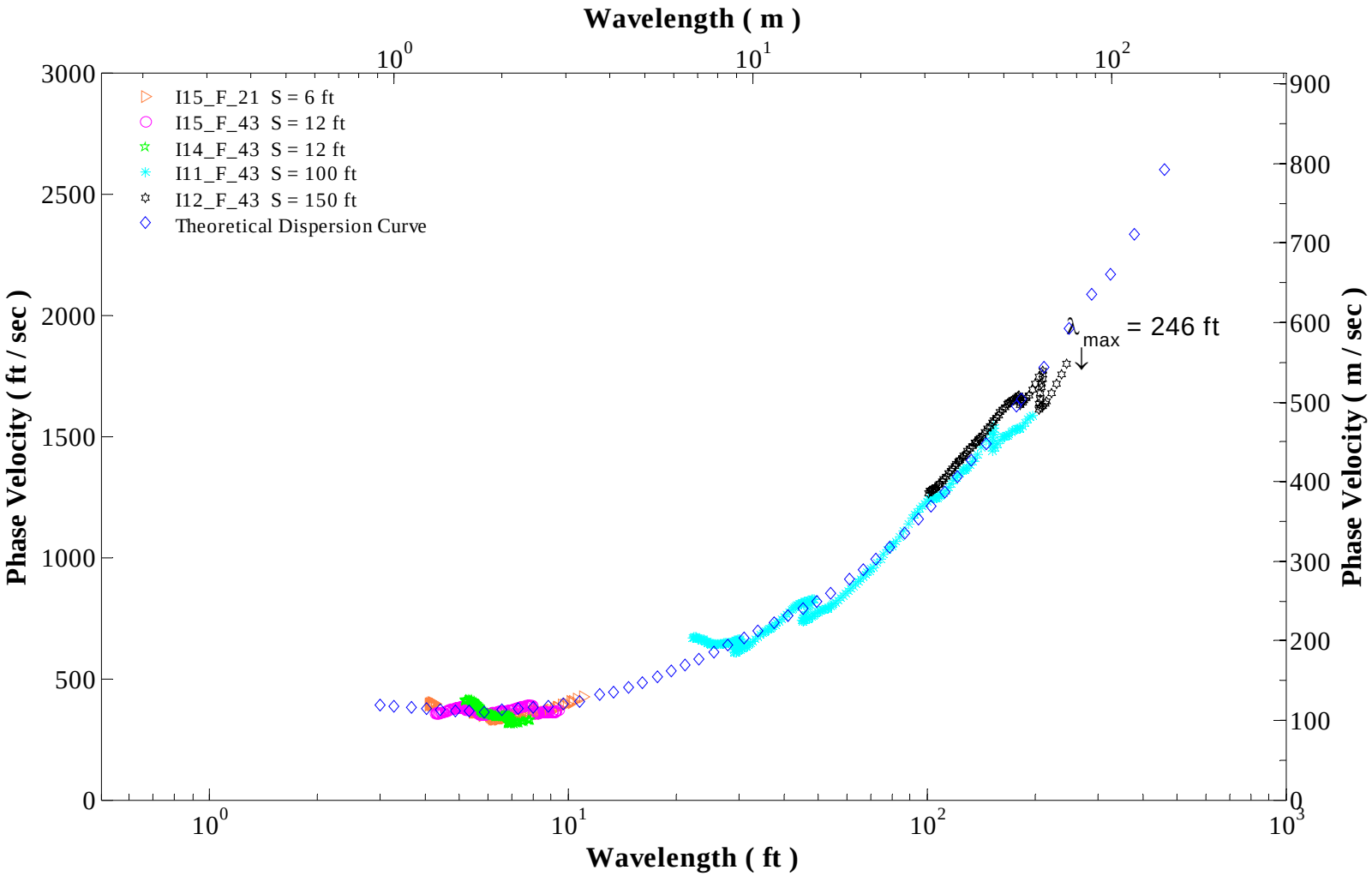
Mountain View Post Office (Array 2)



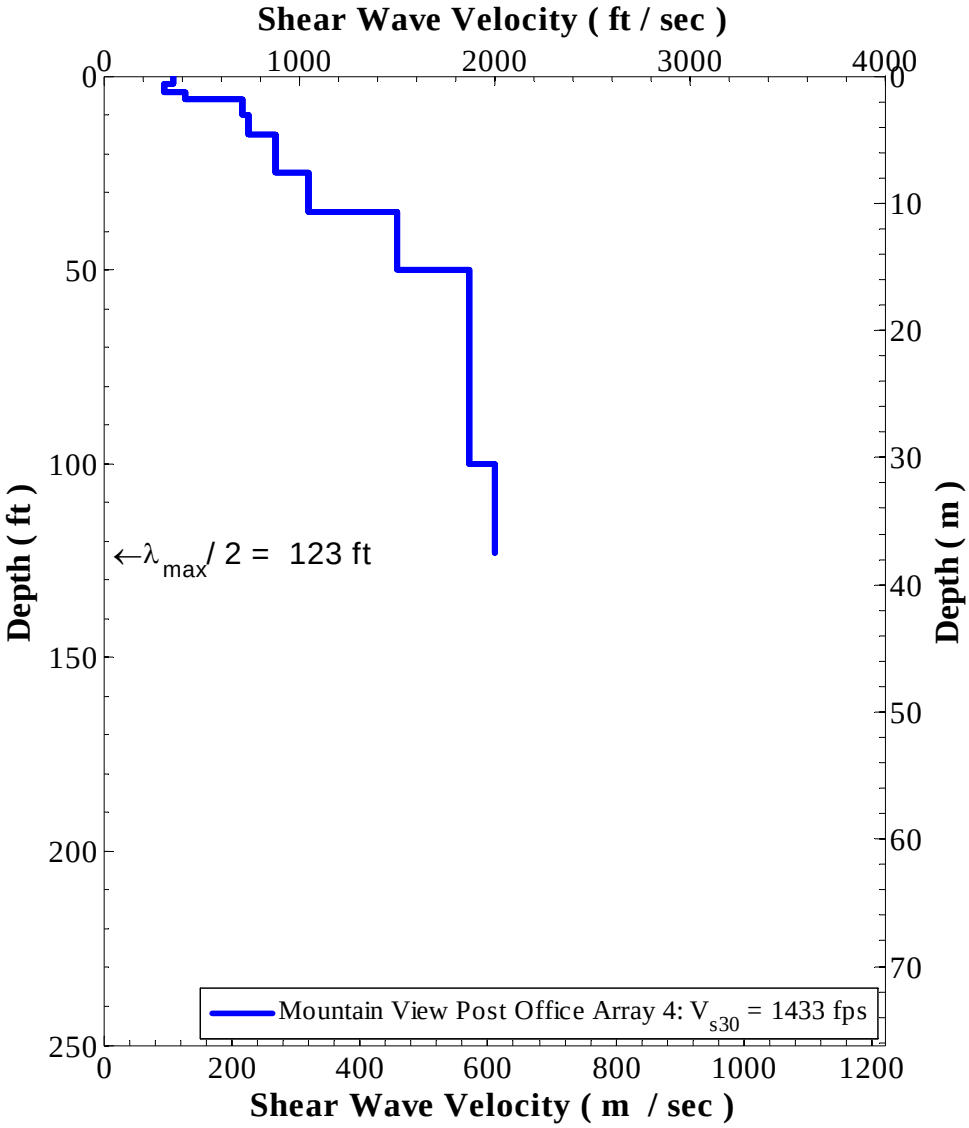
Mountain View Post Office Array 2

Profile Parameter for Mountain View Post Office Array 2

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	1.5	754.4	380	0.33	120
2	2.5	573.7	289	0.33	120
3	2	873.5	440	0.33	120
4	4	5000	700	0.49	120
5	5	5000	710	0.4897	120
6	10	5000	840	0.4862	120
7	10	5000	1050	0.4769	120
8	15	5000	1500	0.4505	120
9	50	5000	1870	0.4187	120
10	99999	5000	2000	0.4048	130



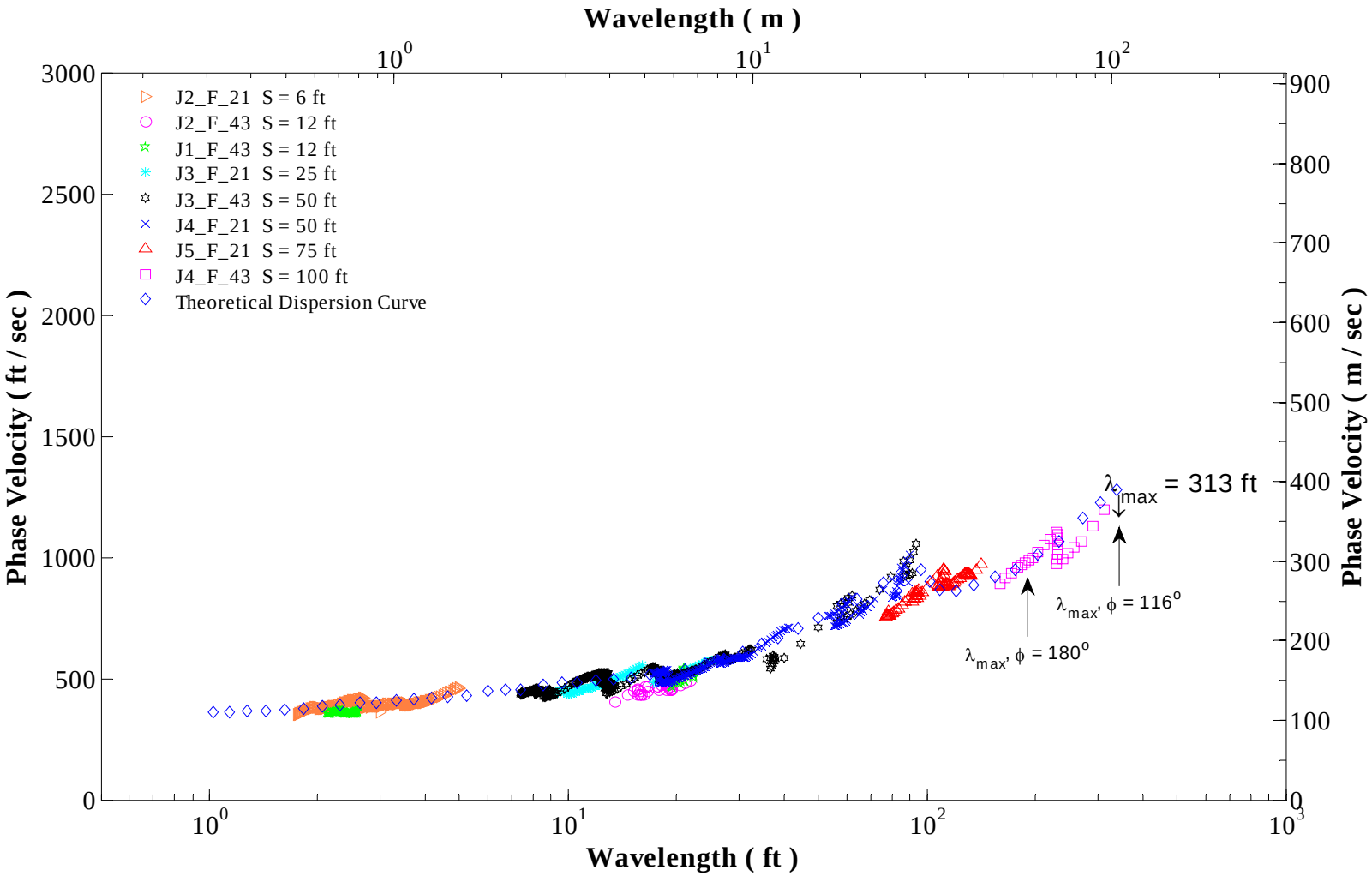
Mountain View Post Office (Array 4)



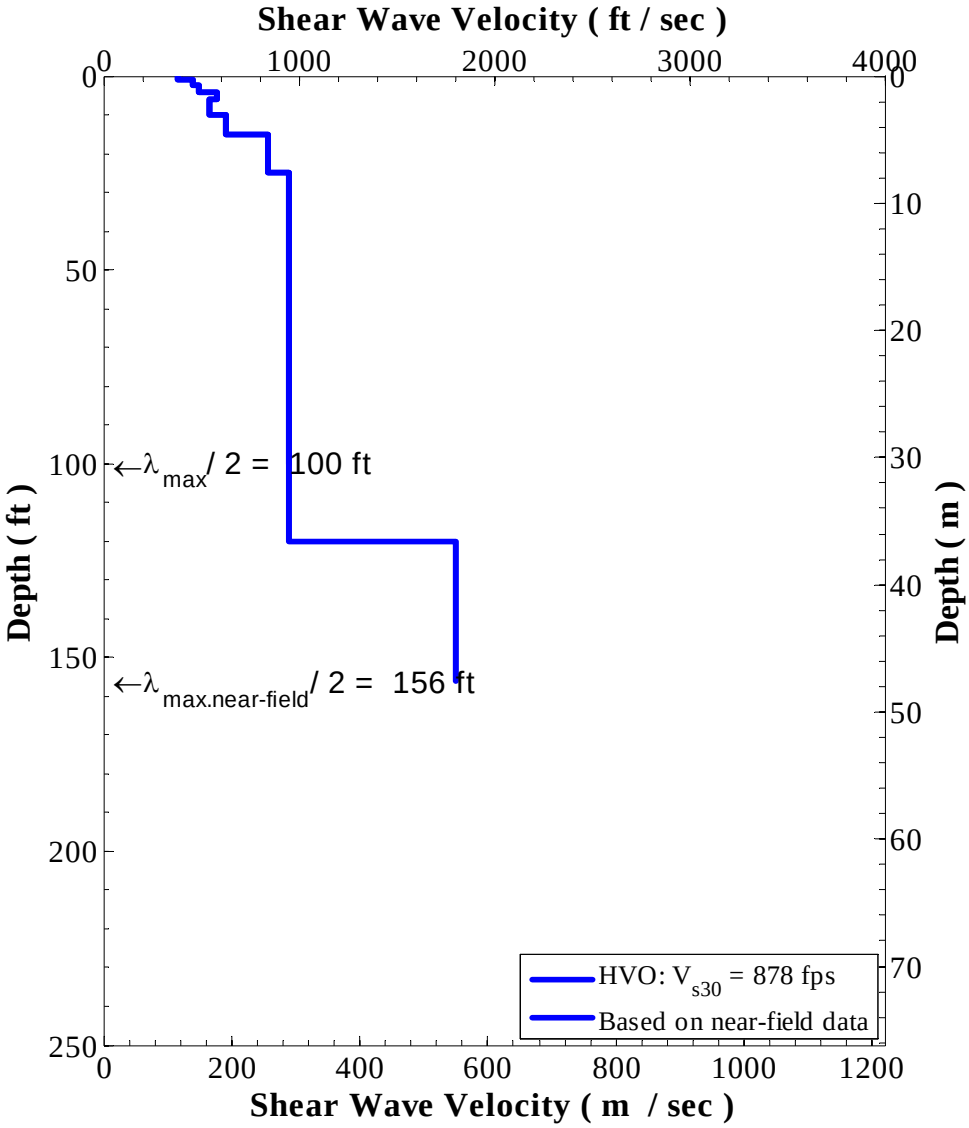
Mountain View Post Office Array 4

Profile Parameter for Mountain View Post Office Array 4

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	2	714.7	360	0.33	120
2	2	615.4	310	0.33	120
3	2	833.8	420	0.33	120
4	4	5000	710	0.4897	120
5	5	5000	740	0.4888	120
6	10	5000	880	0.484	120
7	10	5000	1050	0.4769	120
8	15	5000	1500	0.4505	120
9	50	5000	1870	0.4187	120
10	99999	5000	2000	0.4048	130


Wavelength (m)
10^0
10^1
10^2
3000
2500
2000
1500
1000
500
0
900
800
700
600
500
400
300
200
100
0
J2_F_21 S = 6 ft
J2_F_43 S = 12 ft
J1_F_43 S = 12 ft
J3_F_21 S = 25 ft
J3_F_43 S = 50 ft
J4_F_21 S = 50 ft
J5_F_21 S = 75 ft
J4_F_43 S = 100 ft
Theoretical Dispersion Curve
λ_{max} = 313 ft
λ_{max}, ϕ = 116°
λ_{max}, ϕ = 180°
Phase Velocity (ft / sec)
Phase Velocity (m / sec)
10^0
10^1
10^2
10^3
Wavelength (ft)

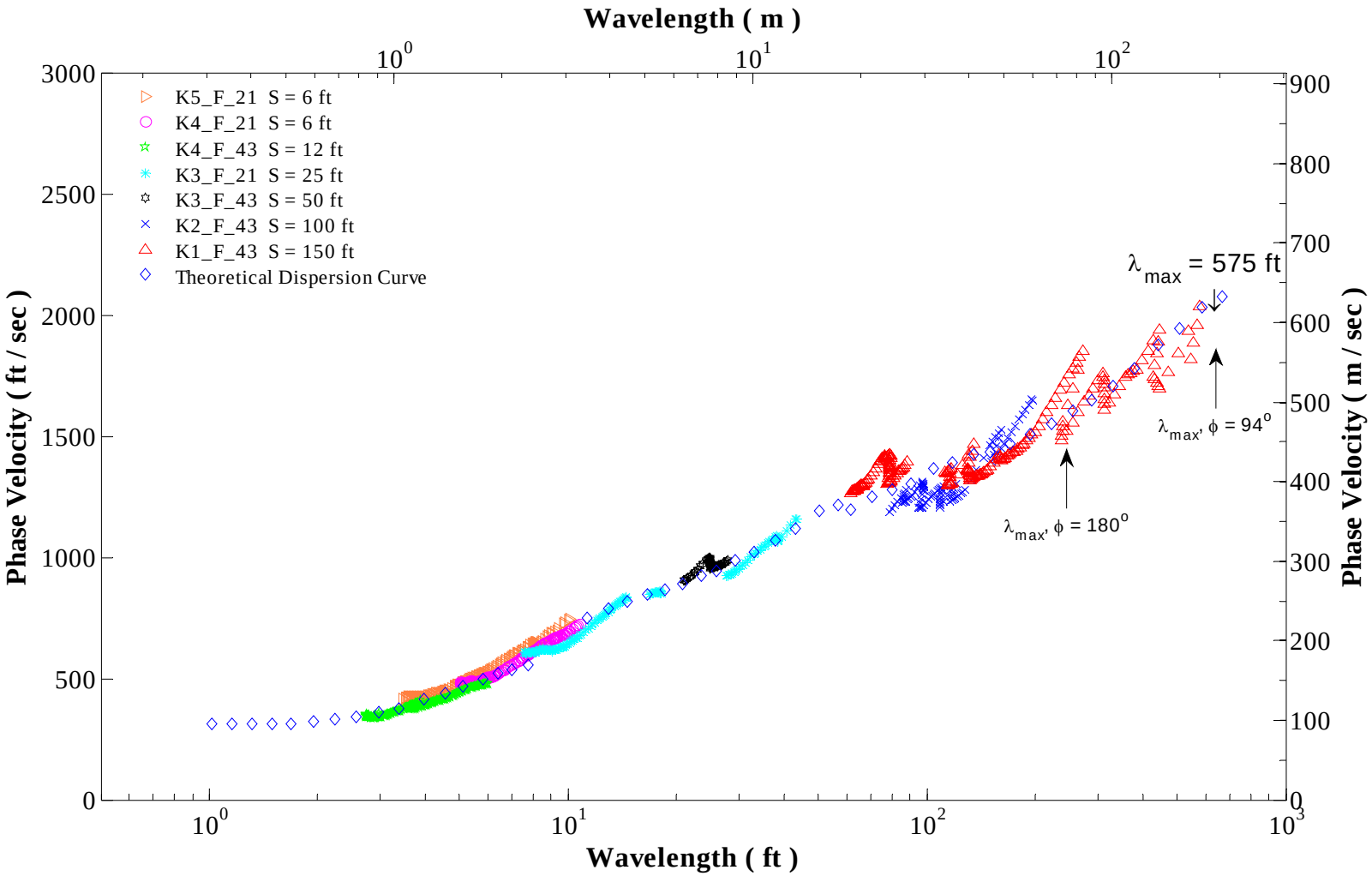
HVO


Shear Wave Velocity (ft / sec)
0
1000
2000
3000
4000
Depth (ft)
0
50
100
150
200
250
←λmax / 2 = 100 ft
←λmax.near-field / 2 = 156 ft
Depth (m)
0
10
20
30
40
50
60
70
HVO: Vs30 = 878 fps
Based on near-field data
0
200
400
600
800
1000
1200
Shear Wave Velocity (m / sec)

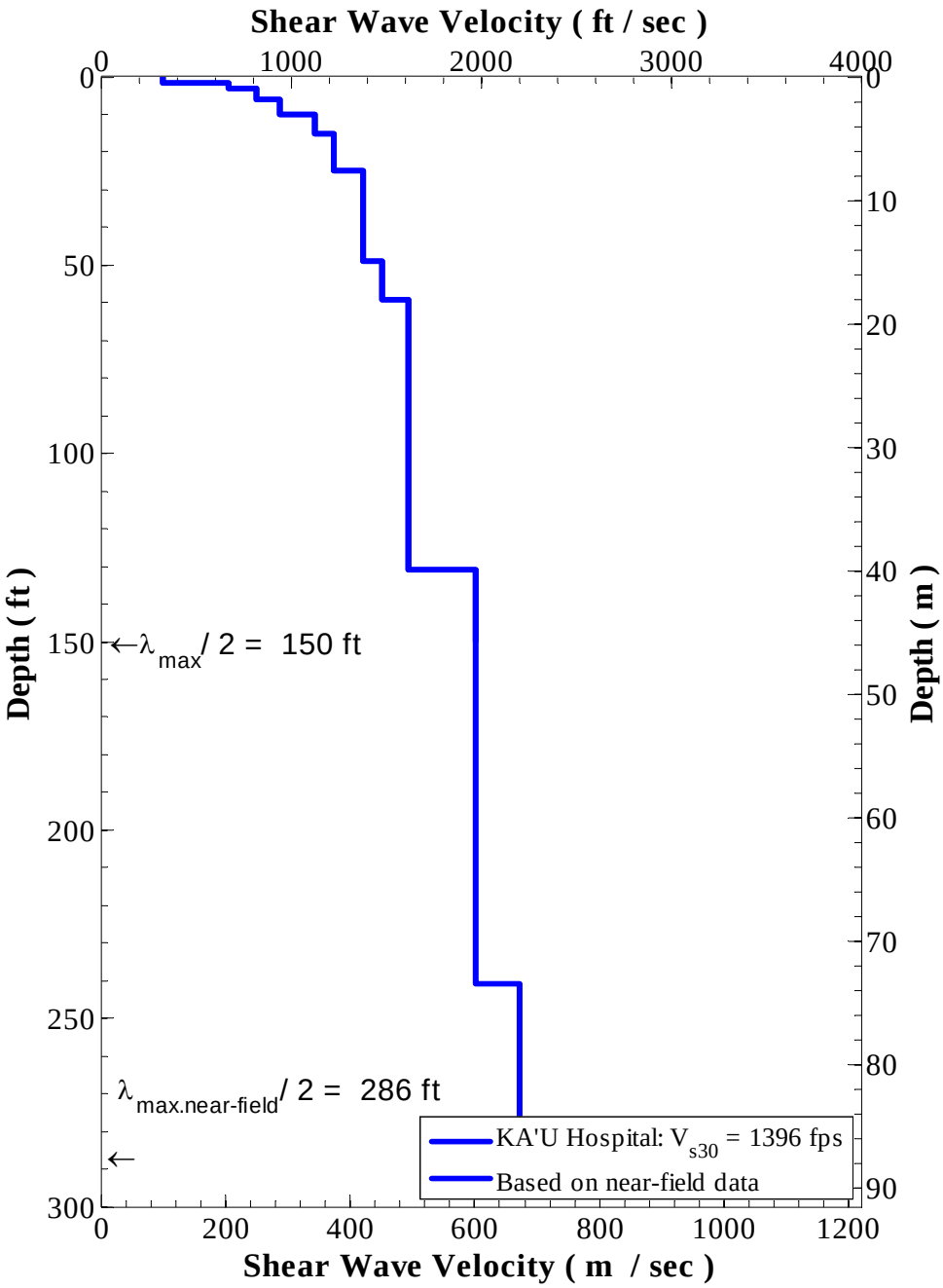
HVO

Profile Parameter for HVO

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	1	754.4	380	0.33	120
2	1.5	913.2	460	0.33	120
3	1.5	972.8	490	0.33	120
4	2	1151.4	580	0.33	120
5	4	5000	540	0.4941	120
6	5	5000	630	0.4919	120
7	10	5000	840	0.4855	120
8	95	5000	950	0.4813	120
9	99999	5000	1800	0.4256	120



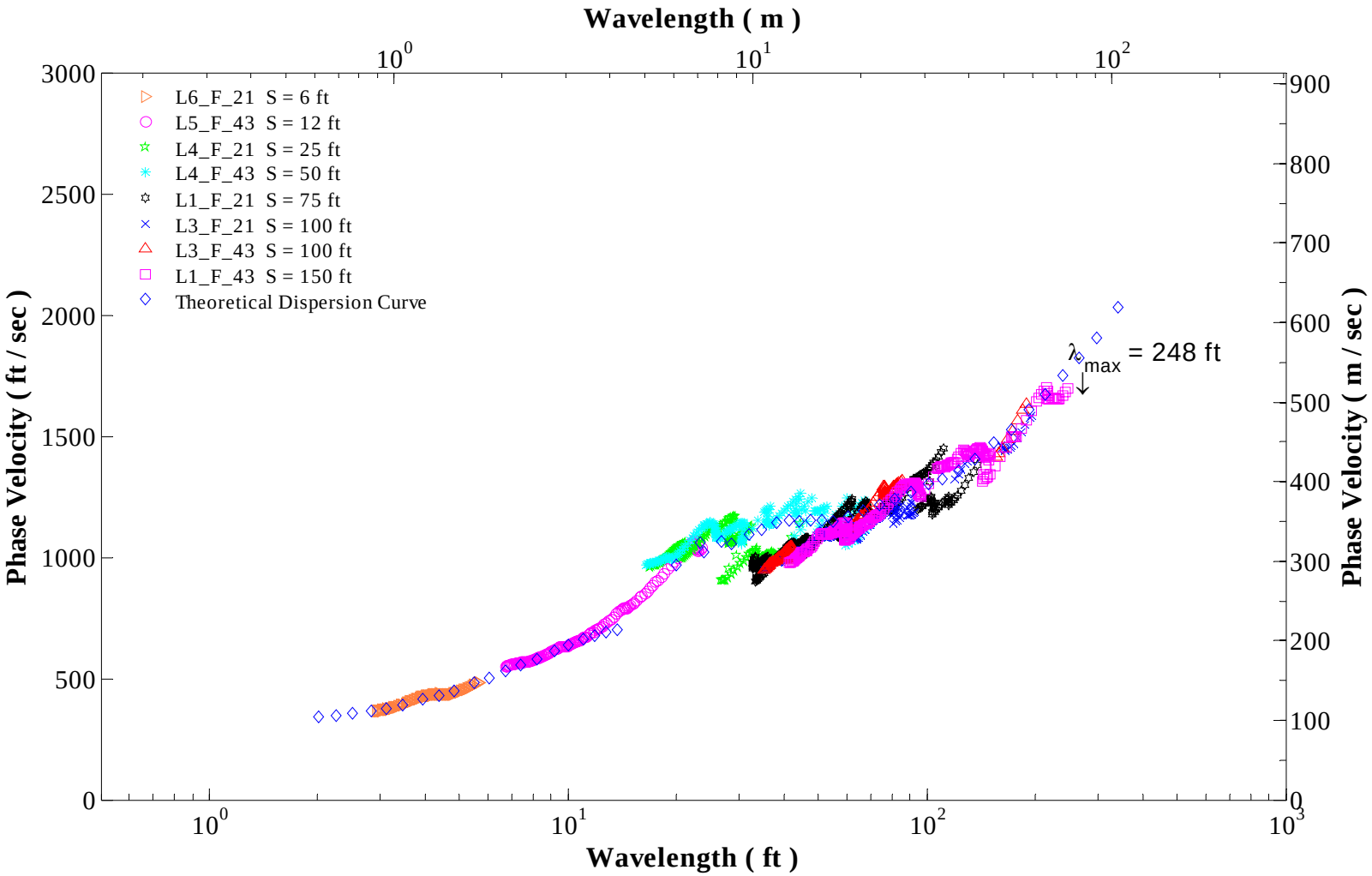
KA'U Hospital



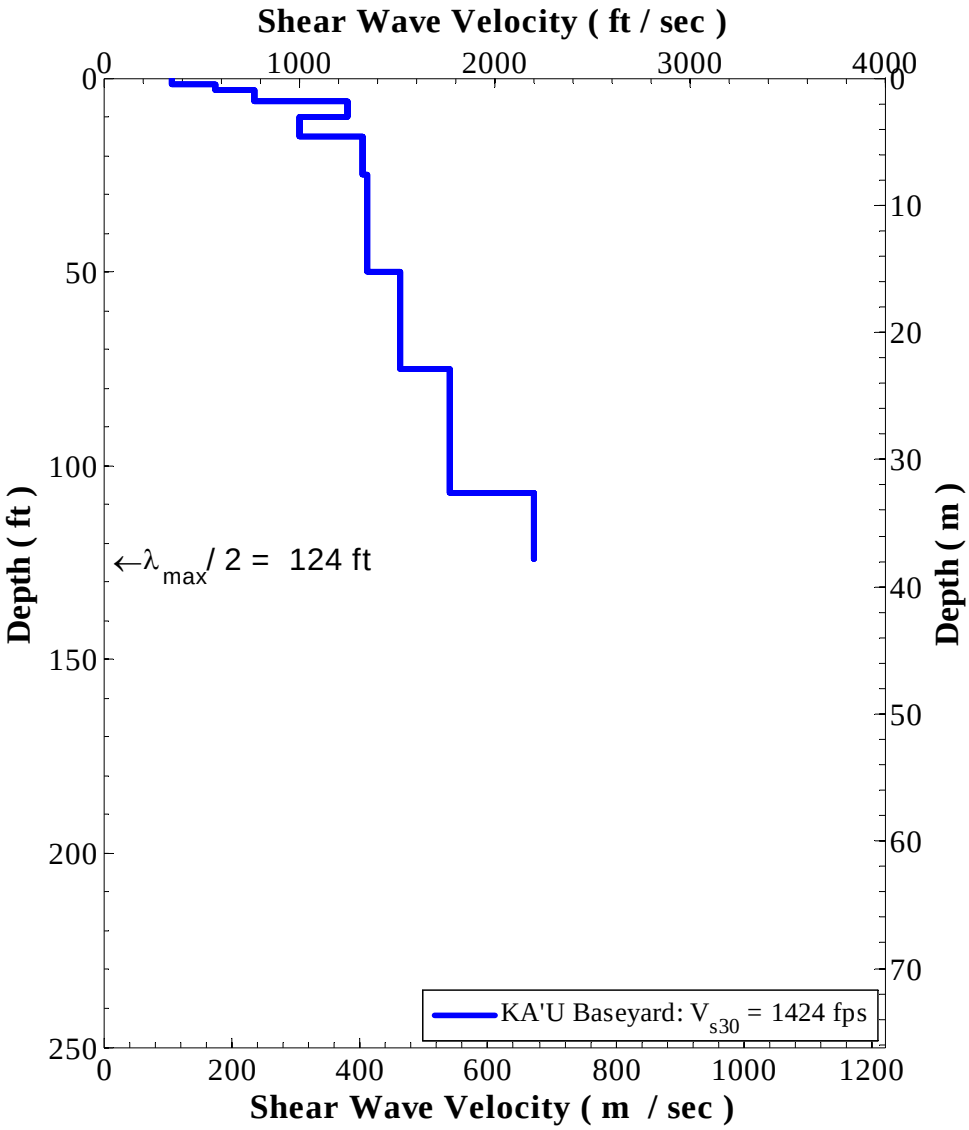
KA'U Hospital

Profile Parameter for KA'U Hospital

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	1.5	655.1	330	0.33	120
2	1.5	1338.1	674	0.33	120
3	3	1627.9	820	0.33	120
4	4	5000	940	0.4817	120
5	5	5000	1130	0.4731	120
6	10	5000	1230	0.4619	120
7	24	5000	1380	0.452	120
8	10	5000	1480	0.484	120
9	72	5000	1620	0.4414	120
10	110	5000	1970	0.4081	120
12	99999	5000	2200	0.38	130



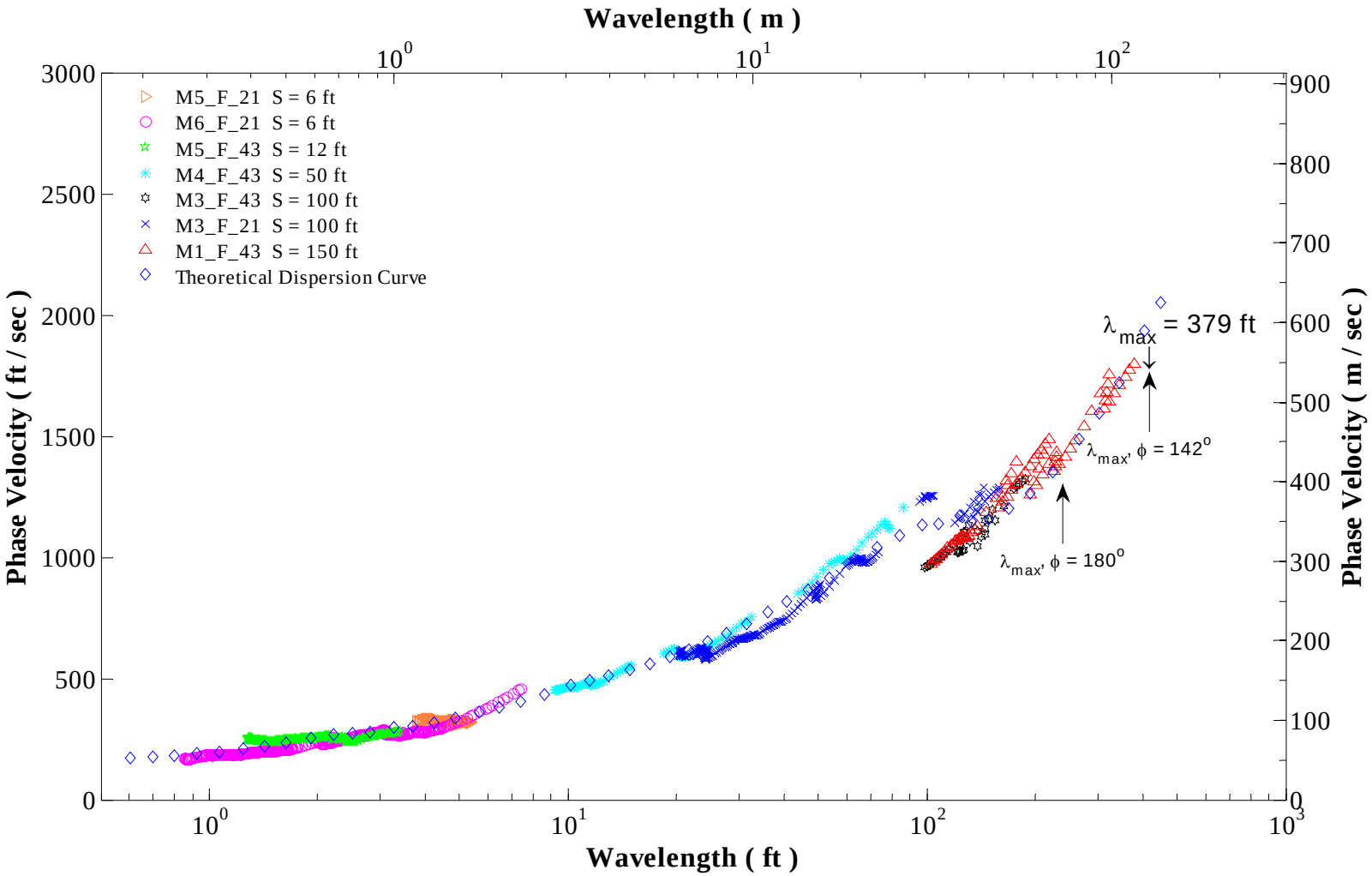
KA'U Baseyard



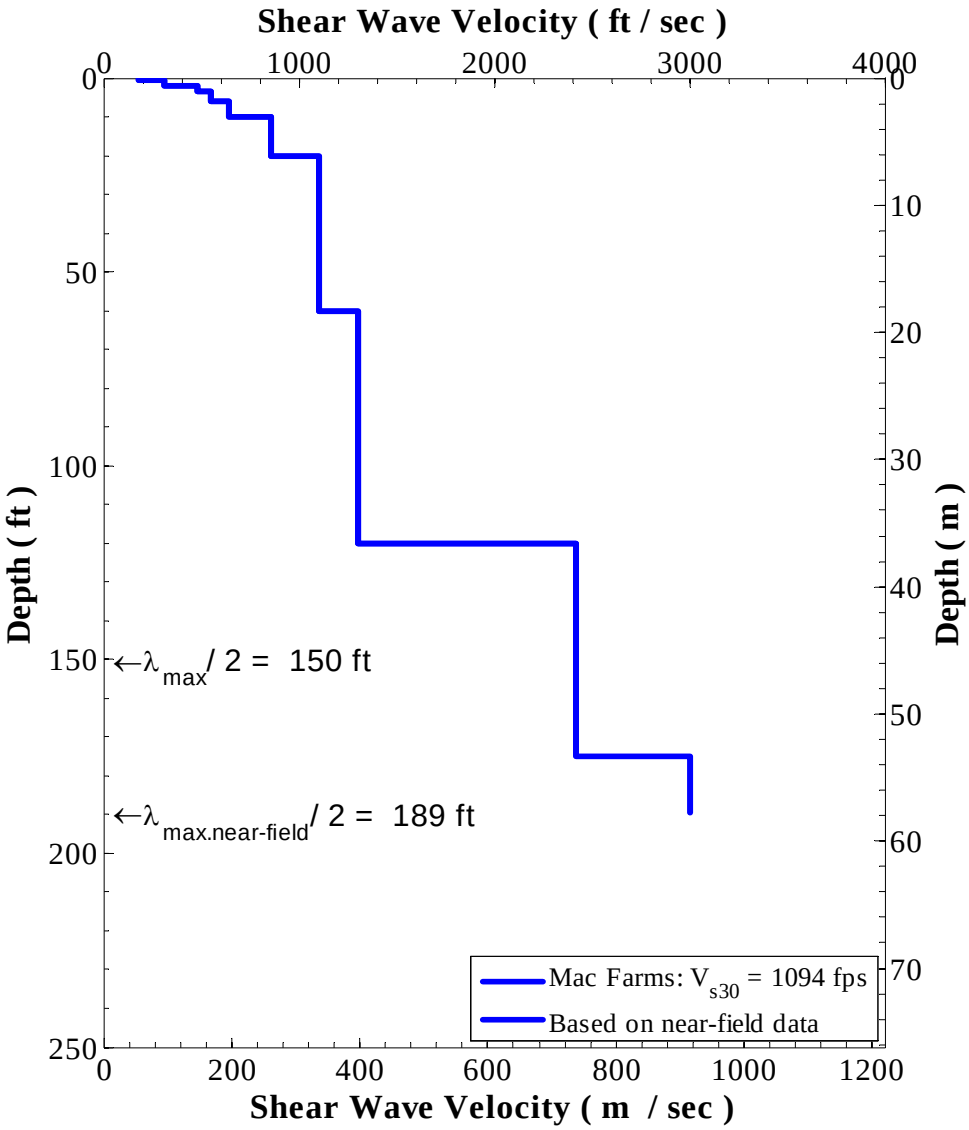
KA'U Baseyard

Profile Parameter for KA'U Baseyard

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	1.5	694.8	350	0.33	120
2	1.5	1139.5	574	0.33	120
3	3	1528.6	770	0.33	120
4	4	5000	1250	0.4667	120
5	5	5000	1000	0.4792	120
6	10	5000	1330	0.4619	120
7	25	5000	1350	0.4607	120
8	25	5000	1520	0.4491	120
9	32	5000	1770	0.4187	120
10	99999	5000	2200	0.38	130


Wavelength (m)
10^0
10^1
10^2
3000
2500
2000
1500
1000
500
0
900
800
700
600
500
400
300
200
100
0
M5_F_21 S = 6 ft
M6_F_21 S = 6 ft
M5_F_43 S = 12 ft
M4_F_43 S = 50 ft
M3_F_43 S = 100 ft
M3_F_21 S = 100 ft
M1_F_43 S = 150 ft
Theoretical Dispersion Curve
λ_{max} = 379 ft
λ_{max}, ϕ = 142°
λ_{max}, ϕ = 180°
Phase Velocity (ft / sec)
Phase Velocity (m / sec)
10^0
10^1
10^2
10^3
Wavelength (ft)

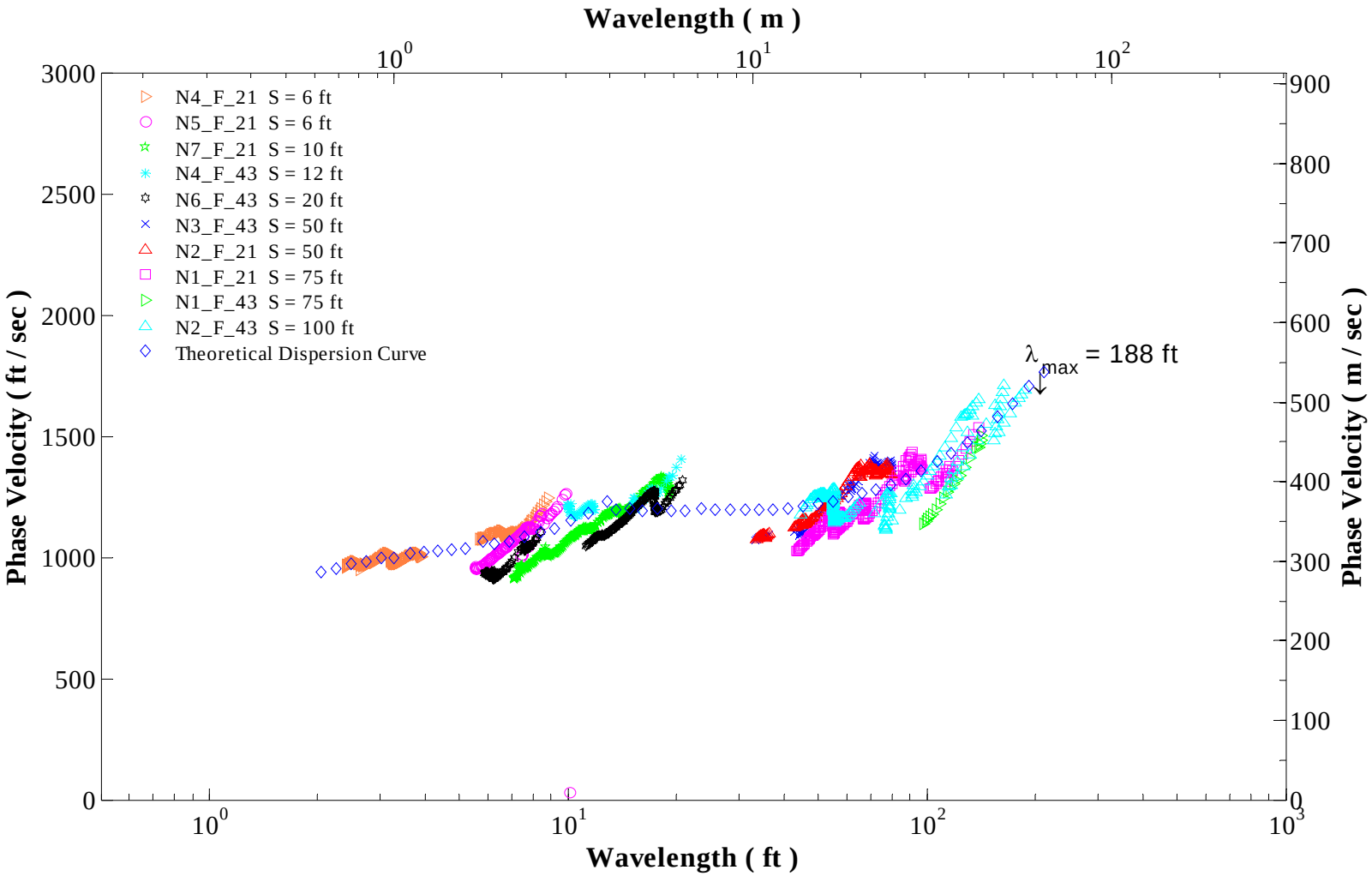
Mac Farms


Shear Wave Velocity (ft / sec)
0
1000
2000
3000
4000
Depth (ft)
0
50
100
150
200
250
Depth (m)
0
10
20
30
40
50
60
70
←λmax / 2 = 150 ft
←λmax.near-field / 2 = 189 ft
Mac Farms: Vs30 = 1094 fps
Based on near-field data
0
200
400
600
800
1000
1200
Shear Wave Velocity (m / sec)

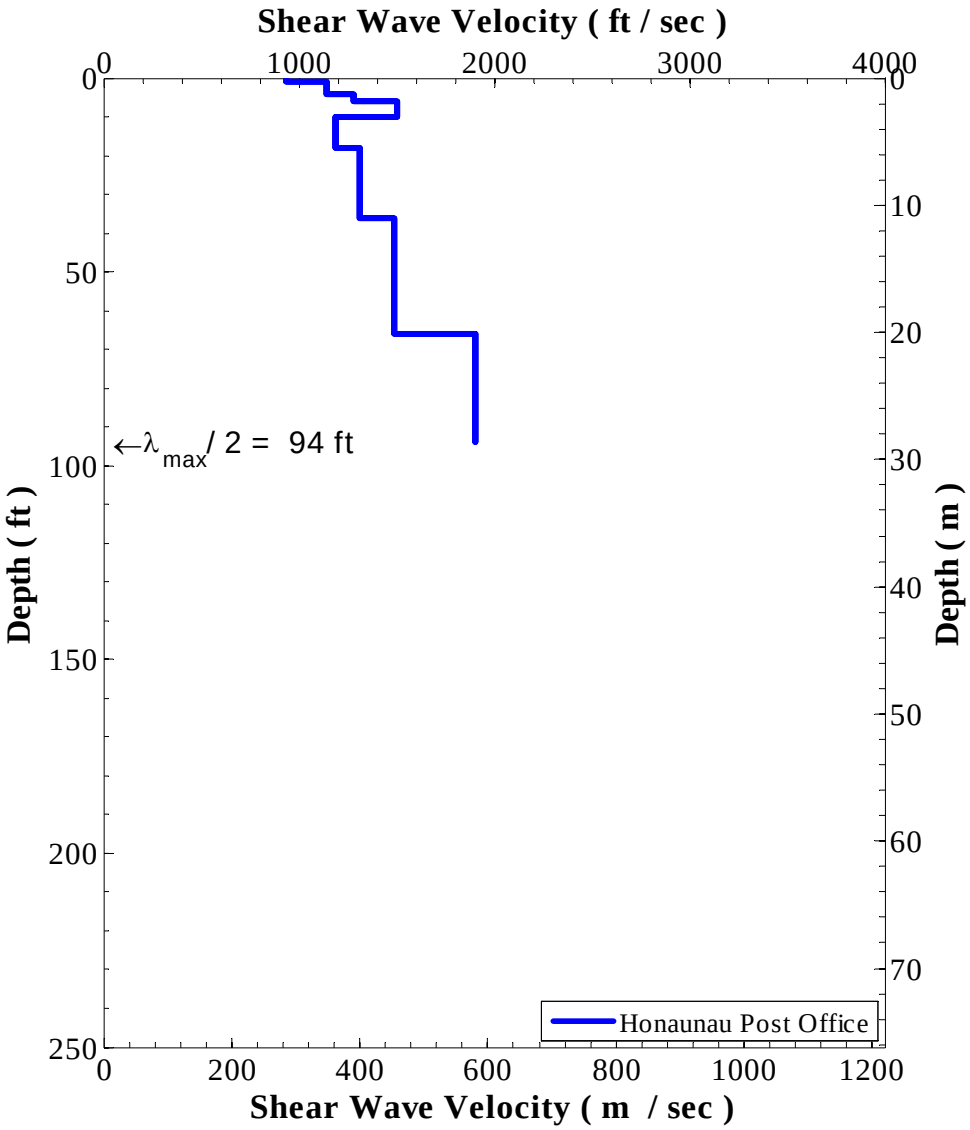
Mac Farms

Profile Parameter for Mac Farms

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	0.5	357.3	180	0.33	120
2	1.5	615.4	310	0.33	120
3	1.6	952.9	480	0.33	120
4	2.4	1091.9	550	0.33	120
5	4	5000	640	0.4917	120
6	10	5000	860	0.4848	120
7	20	5000	1300	0.4637	120
8	10	5000	620	0.4765	120
9	65	5000	1520	0.4491	120
10	99999	5000	2400	0.3503	130



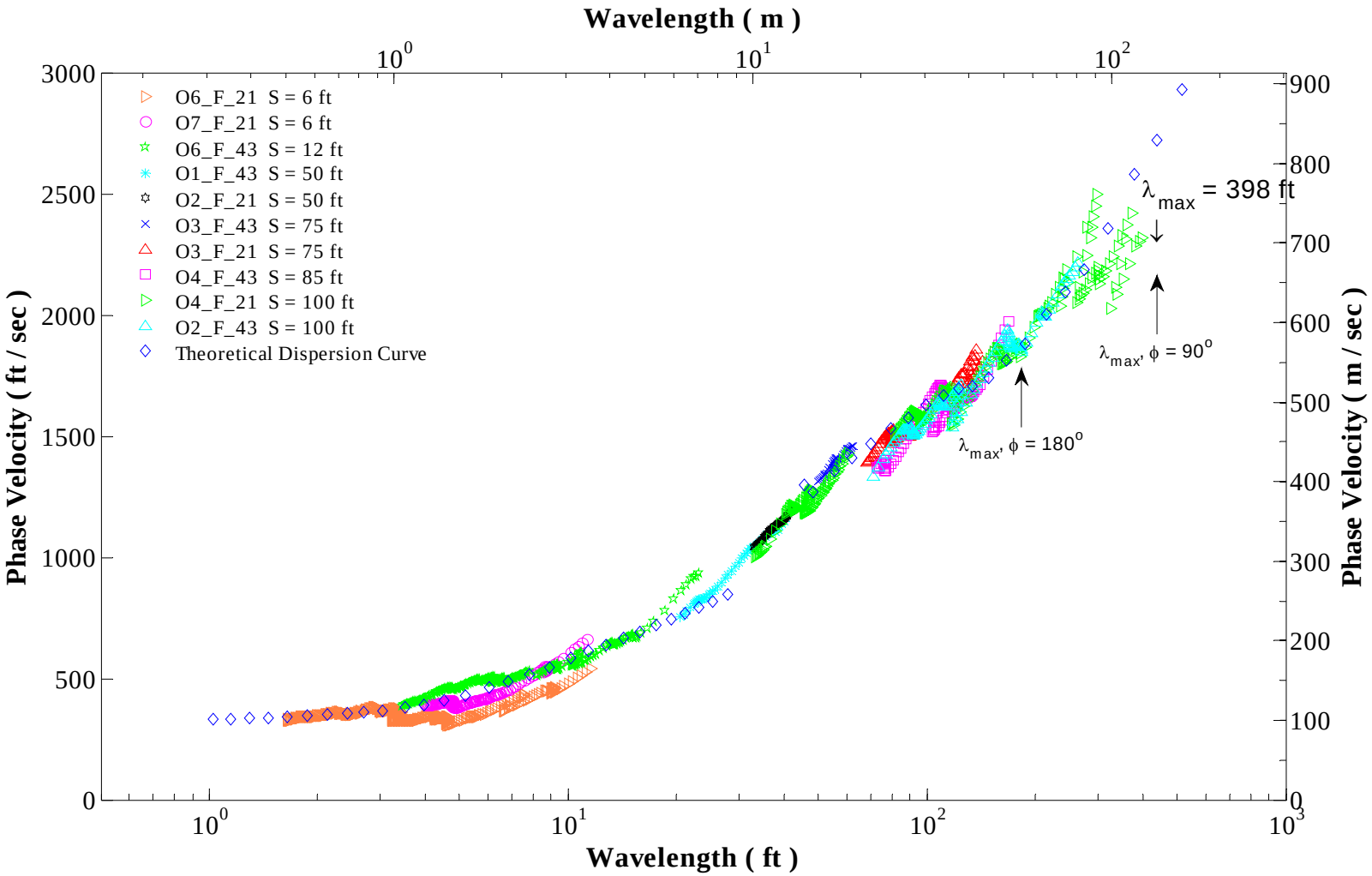
Honaunau Post Office



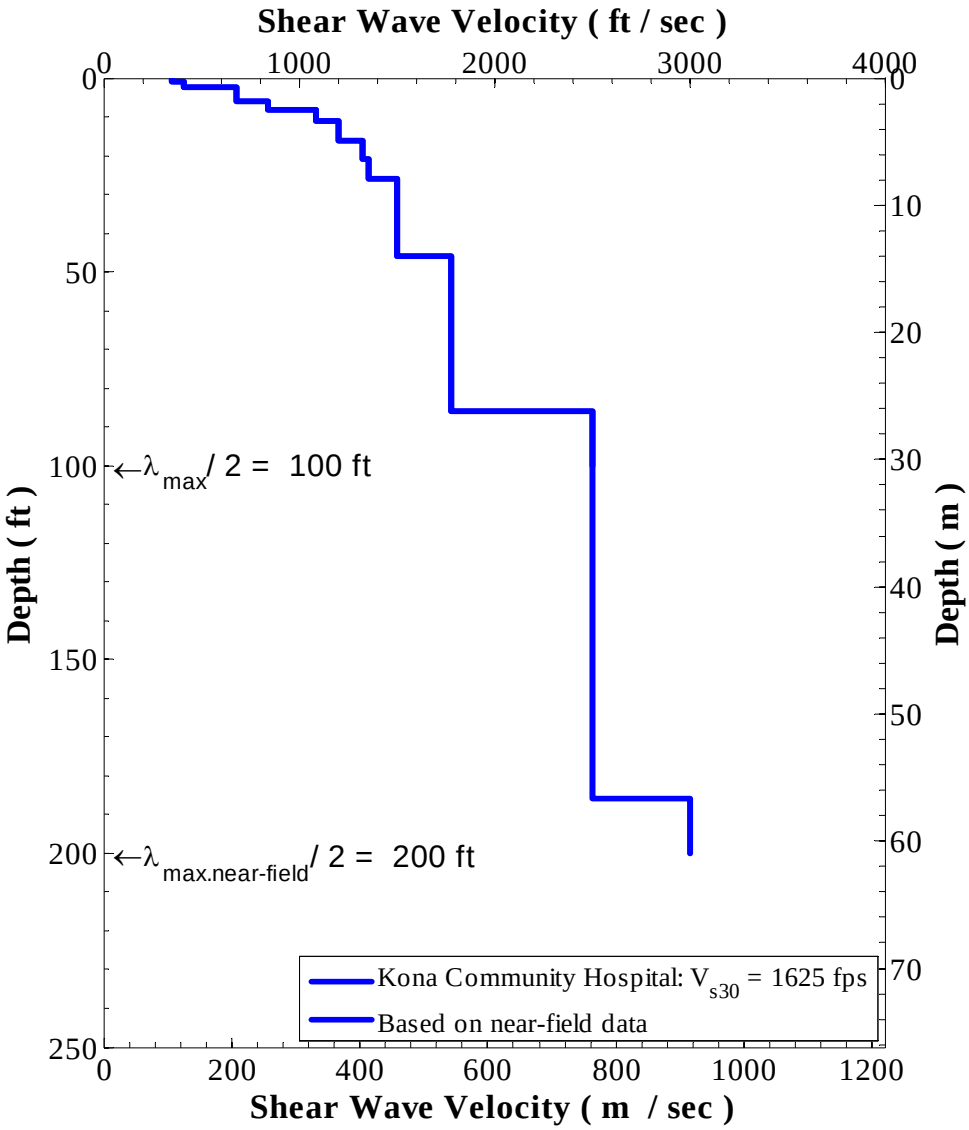
Honaunau Post Office

Profile Parameter for Honaunau Post Office

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	1	1848.3	931	0.33	120
2	3	2263.2	1140	0.33	120
3	2	2541.1	1280	0.33	120
4	4	5000	1500	0.4505	120
5	8	5000	1188	0.4701	120
6	18	5000	1310	0.4631	120
7	30	5000	1490	0.4513	120
8	40	5000	1900	0.4156	120
9	99999	5000	2200	0.38	130



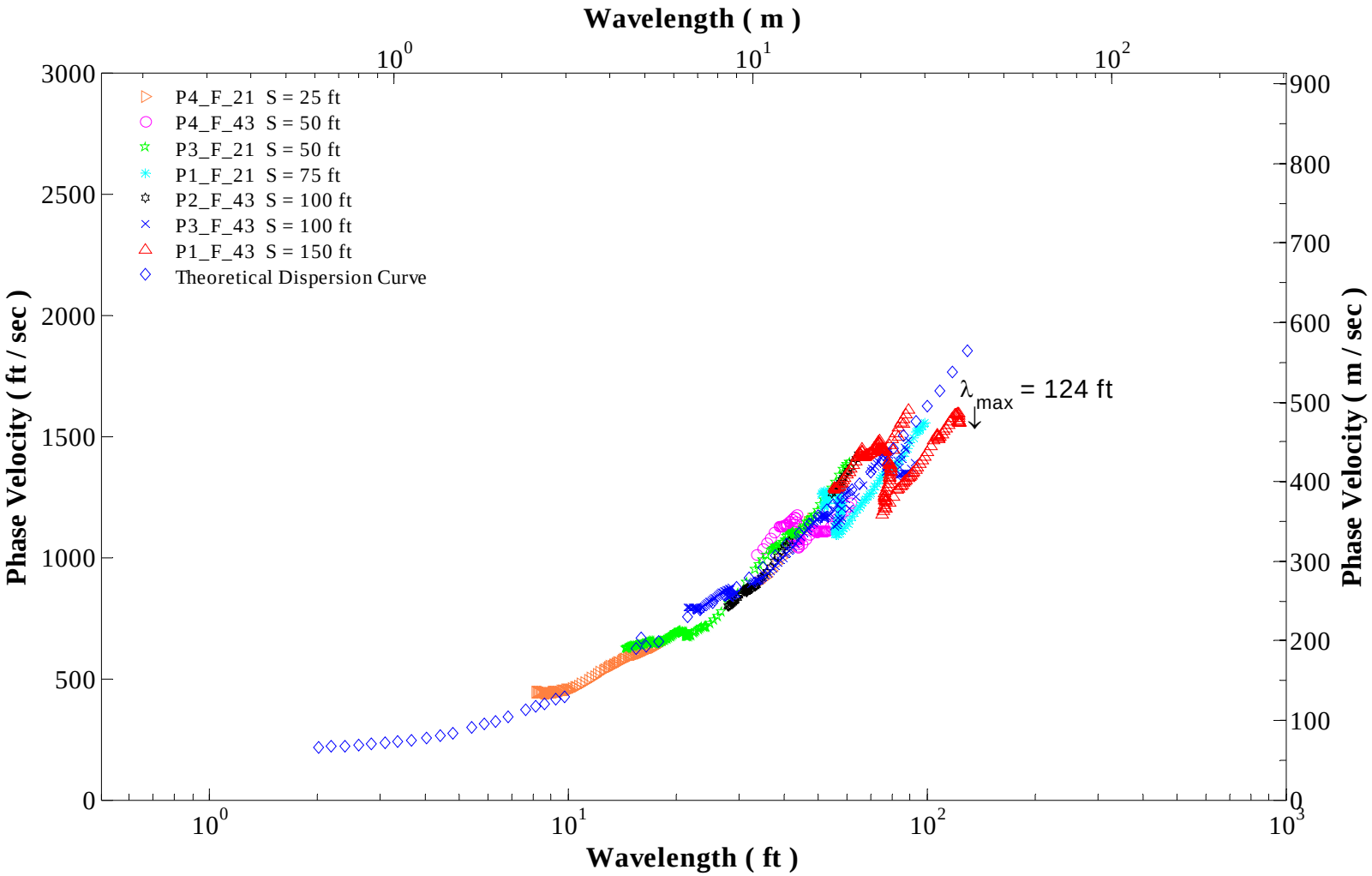
Kona Community Hospital



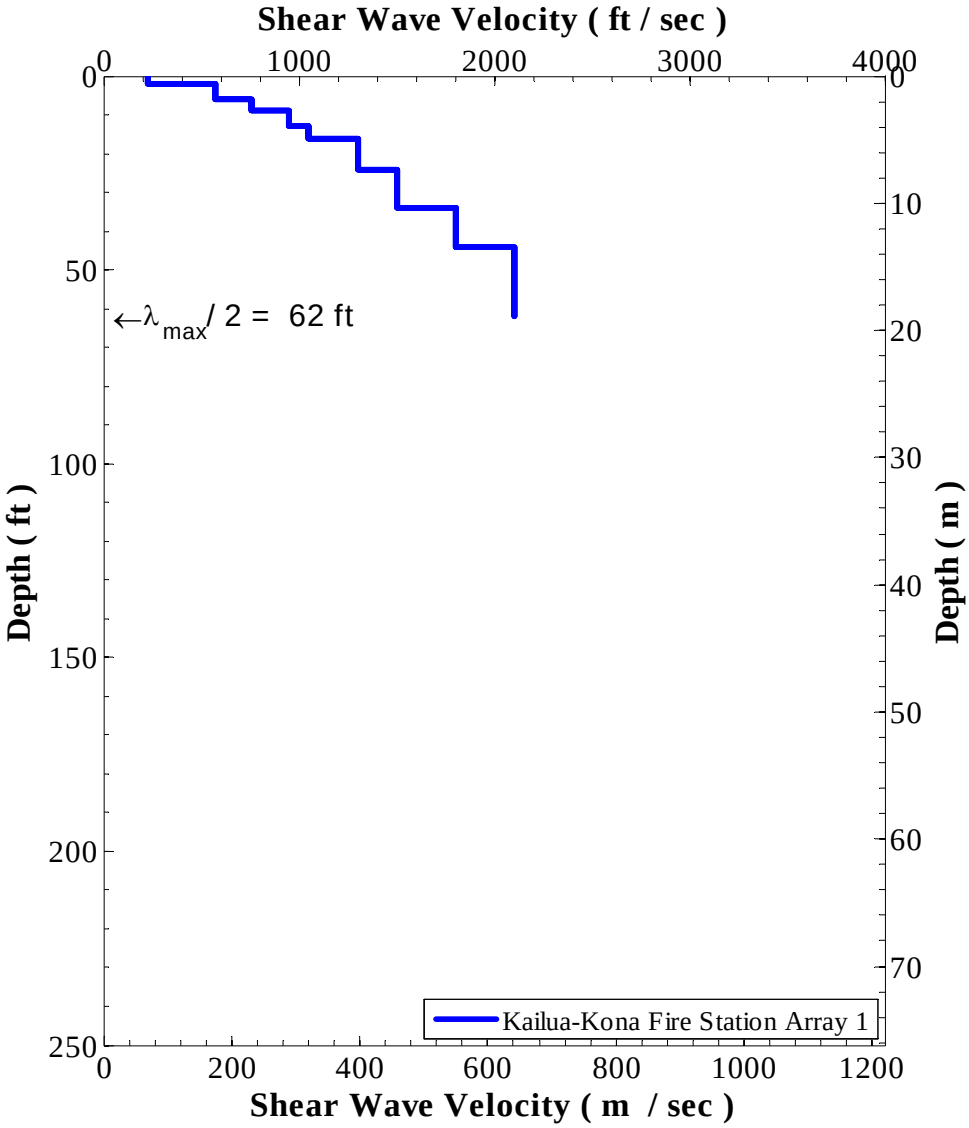
Kona Community Hospital

Profile Parameter for Kona Community Hospital

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	1	694.8	350	0.33	120
2	1.5	813.9	410	0.33	120
3	3.5	1350	680	0.33	120
4	2	5000	840	0.4855	120
5	3	5000	1085	0.4753	120
6	5	5000	1205	0.4692	120
7	5	5000	1330	0.4619	120
8	5	5000	1360	0.4601	120
9	20	5000	1500	0.4505	130
10	40	5000	1780	0.4274	130
11	100	5000	2500	0.3333	130
12	99999	5000	3000	0.4048	130



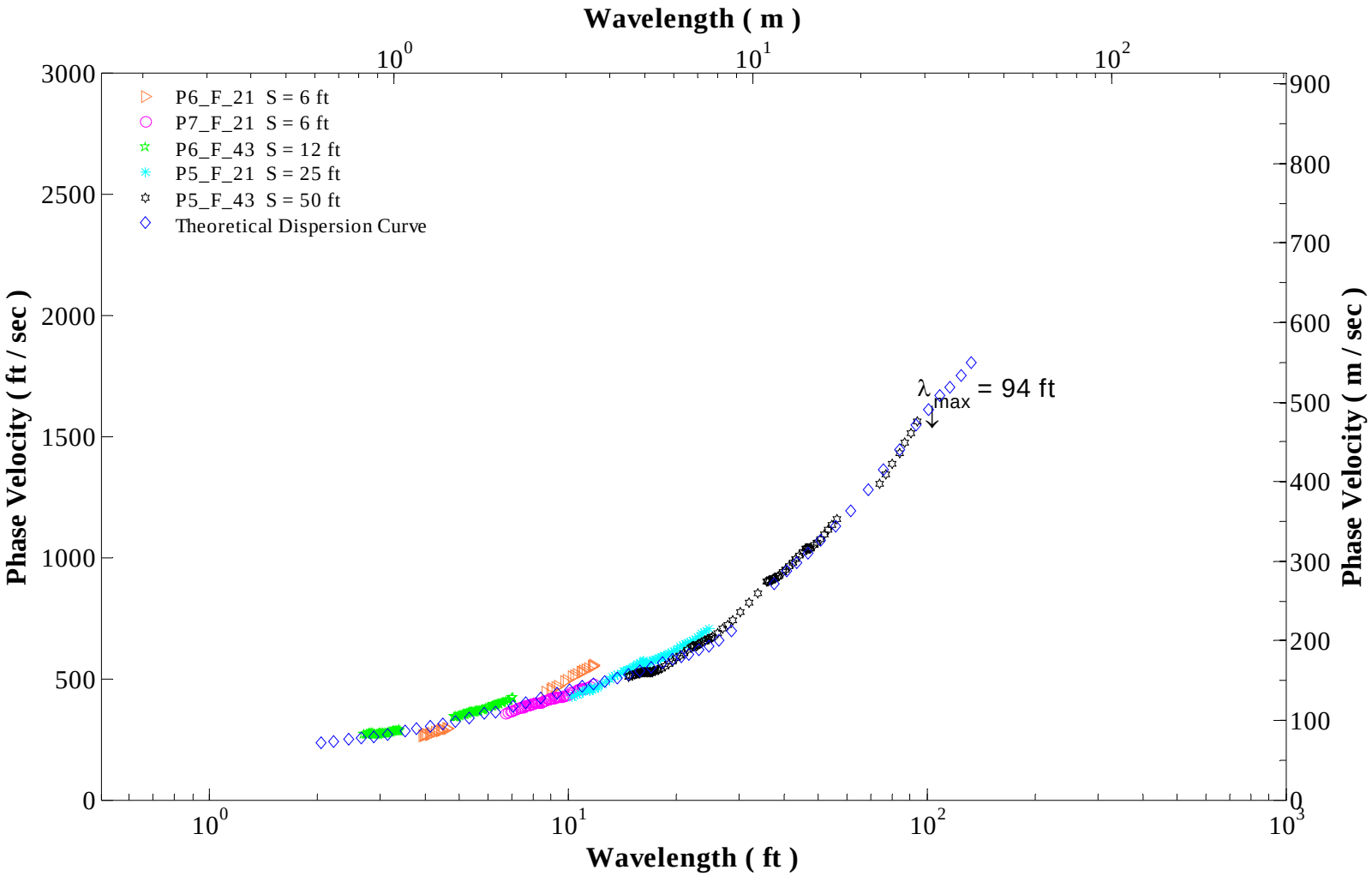
Kailua-Kona Fire Station Array 1



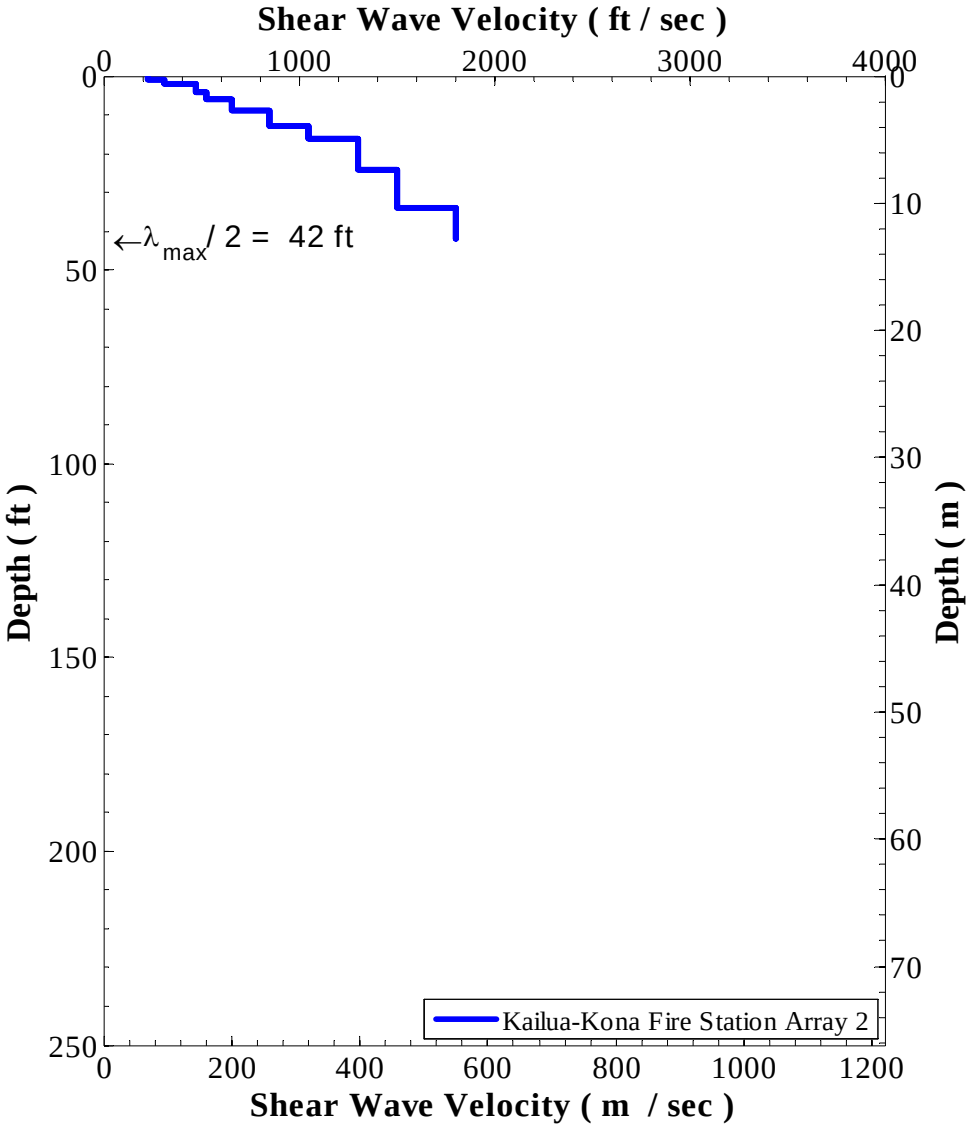
Kailua-Kona Fire Station Array 1

Profile Parameter for Kailua-Kona Fire Station Array 1

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	2	436.8	230	0.33	120
2	4	1052.2	570	0.33	120
3	3	5000	760	0.4911	120
4	4	5000	950	0.4851	120
5	3	5000	1050	0.4769	120
6	8	5000	1300	0.4637	120
7	10	5000	1500	0.4505	120
8	10	5000	1800	0.4256	120
9	99999	5000	2100	0.3929	130



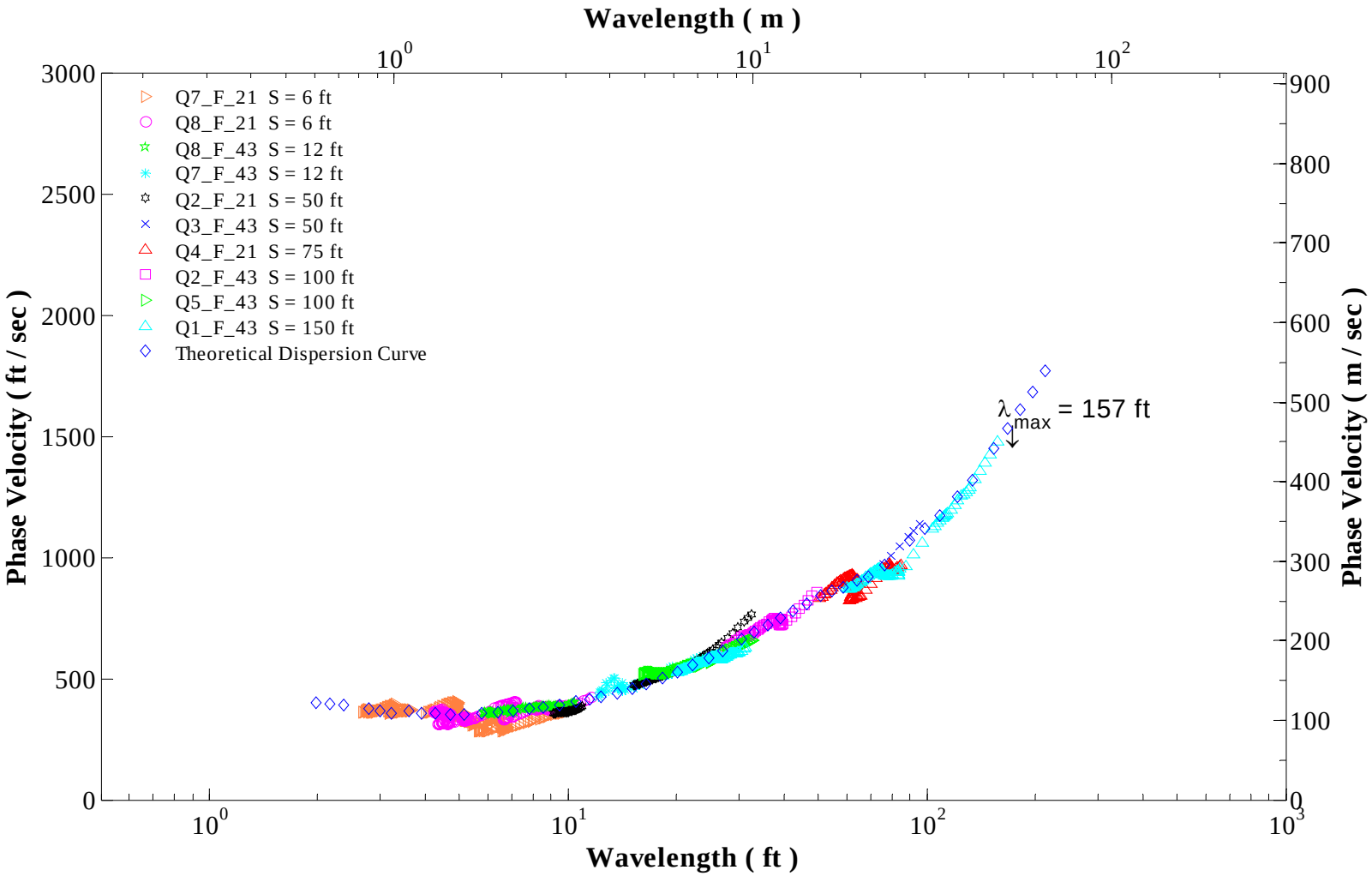
Kailua-Kona Fire Station Array 2



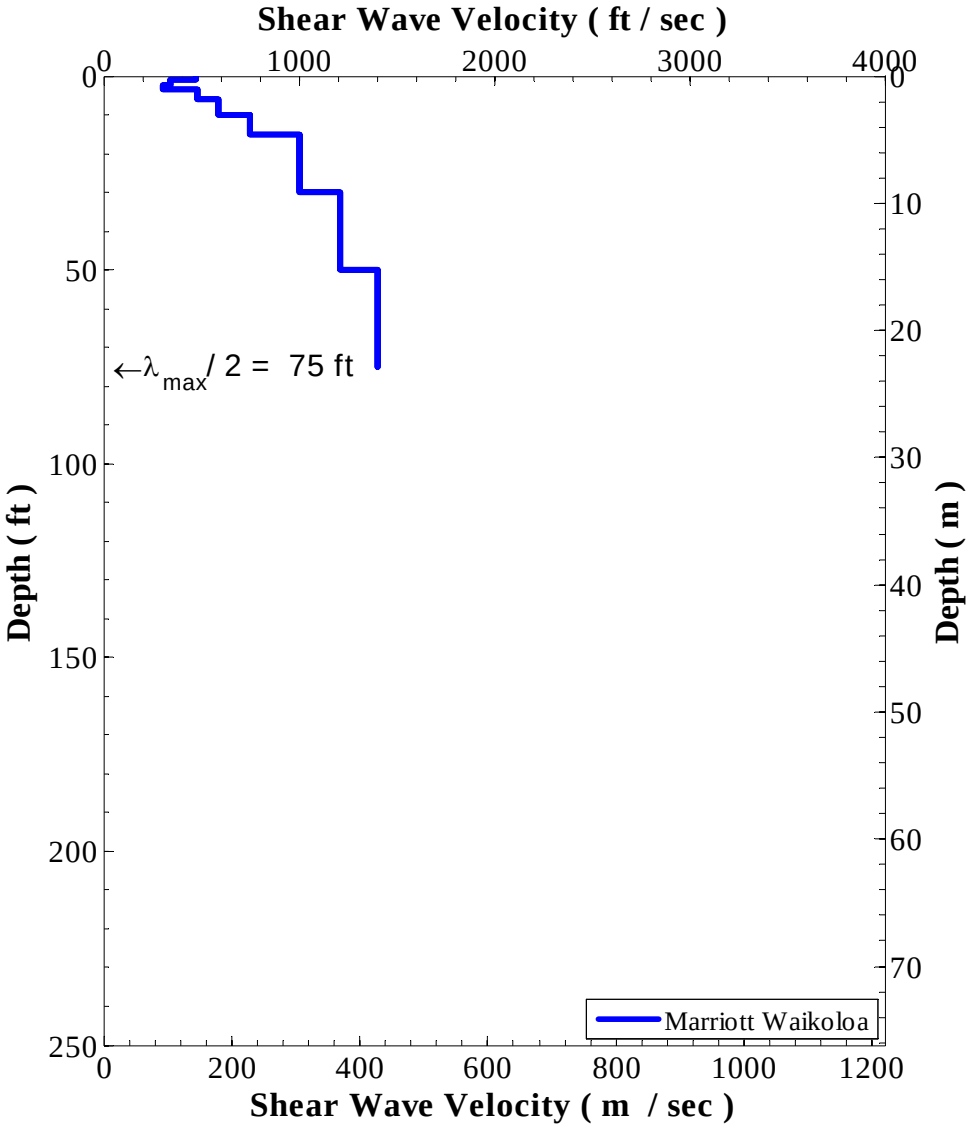
Kailua-Kona Fire Station Array 2

Profile Parameter for Kailua-Kona Fire Station Array 2

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	1	436.8	230	0.33	120
2	1	615.4	310	0.33	120
3	2	933.1	470	0.33	120
4	2	1052.2	530	0.33	120
5	3	5000	660	0.4911	120
6	4	5000	850	0.4851	120
7	3	5000	1050	0.4769	120
8	8	5000	1300	0.4637	120
9	10	5000	1500	0.4505	120
10	10	5000	1800	0.4256	120
11	99999	5000	2100	0.3929	130



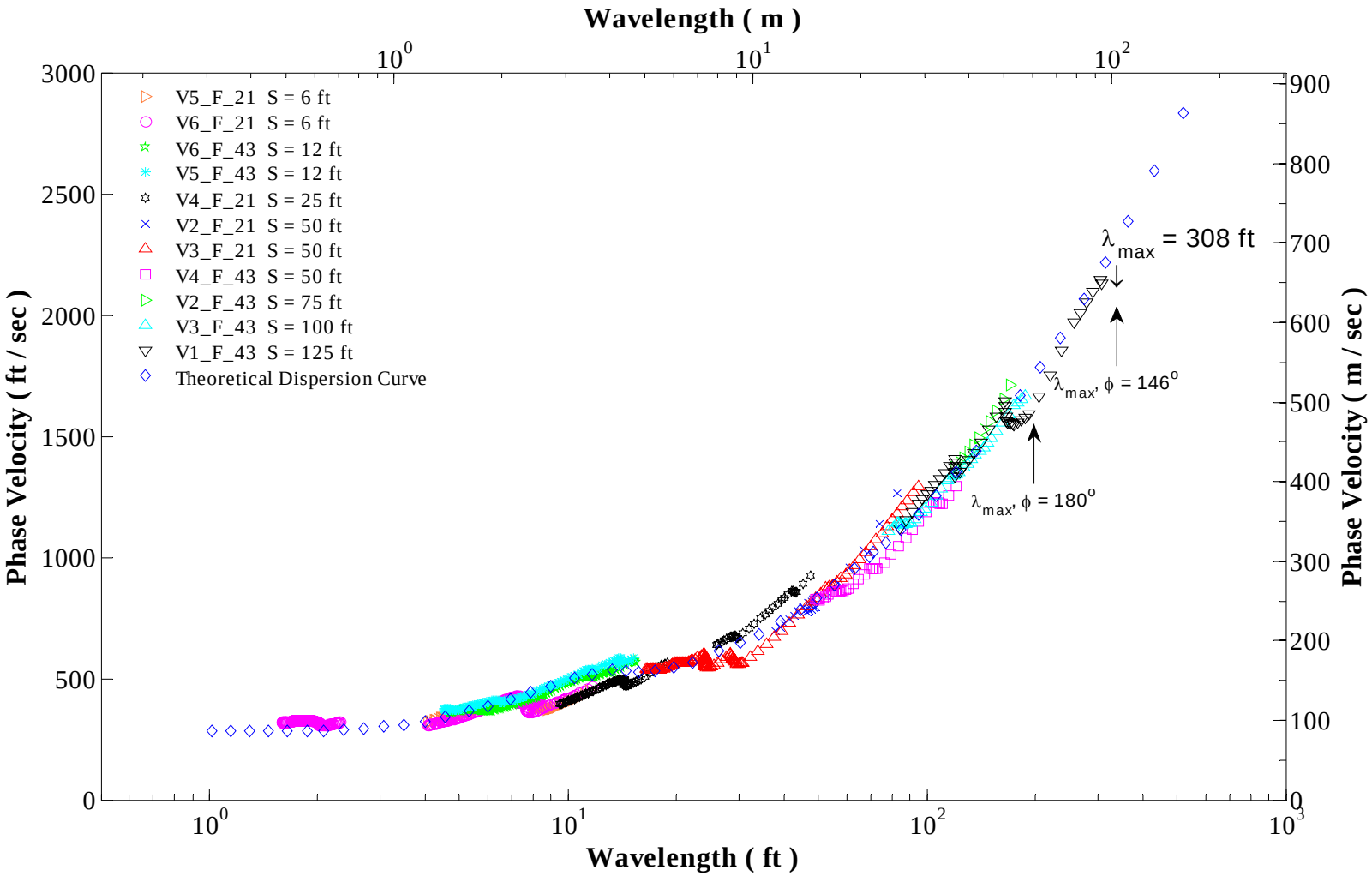
Marriott Waikoloa



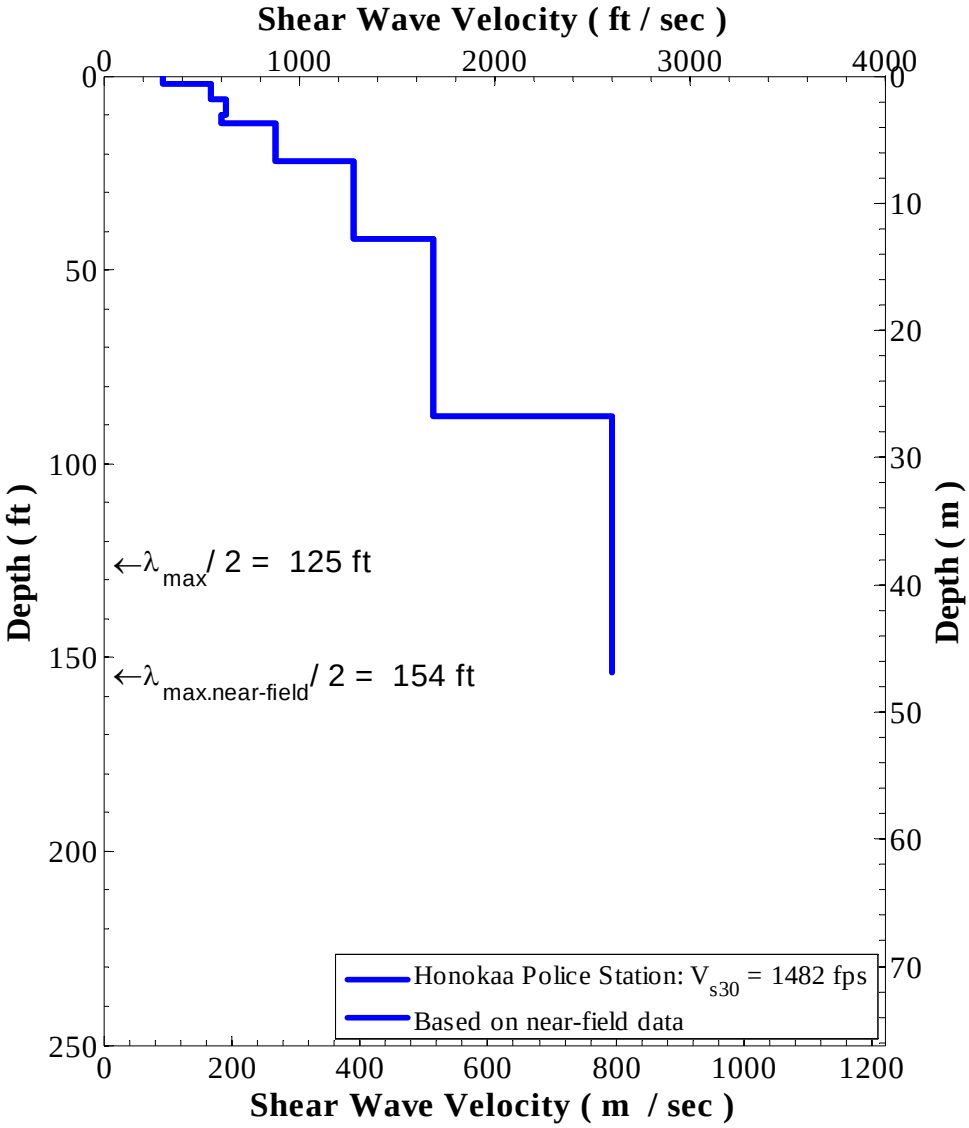
Marriott Waikoloa

Profile Parameter for Marriott Waikoloa

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	1	933.1	470	0.33	120
2	1.5	675	340	0.33	120
3	1	595.6	300	0.33	120
4	2.5	952.9	480	0.33	120
5	4	1023.4	590	0.33	120
6	5	5000	750	0.4885	120
7	15	5000	1000	0.4792	120
8	20	5000	1210	0.4689	120
9	25	5000	1400	0.4575	120
10	99999	6947.4	3000	0.3854	130



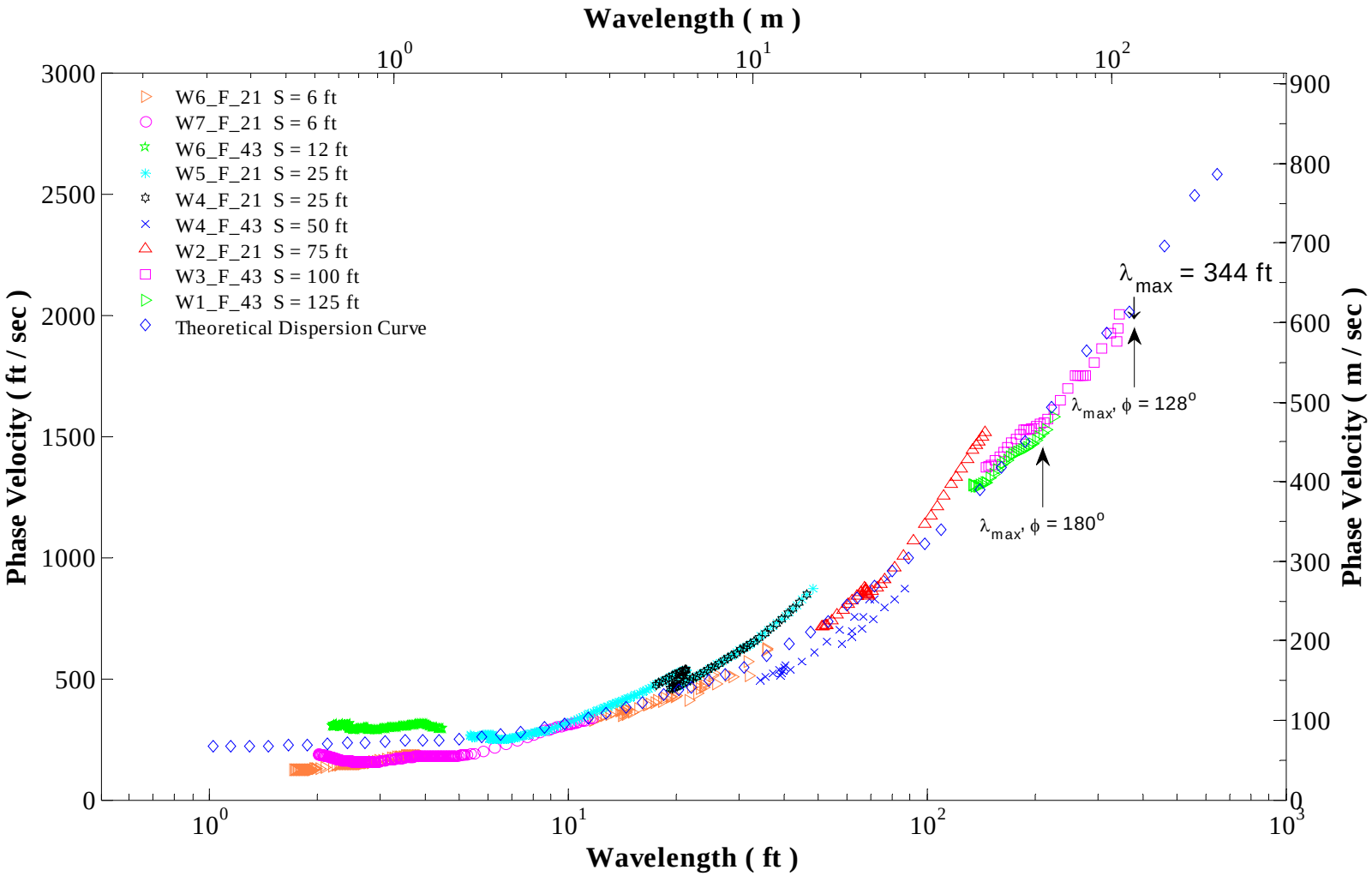
Honokaa Police Station



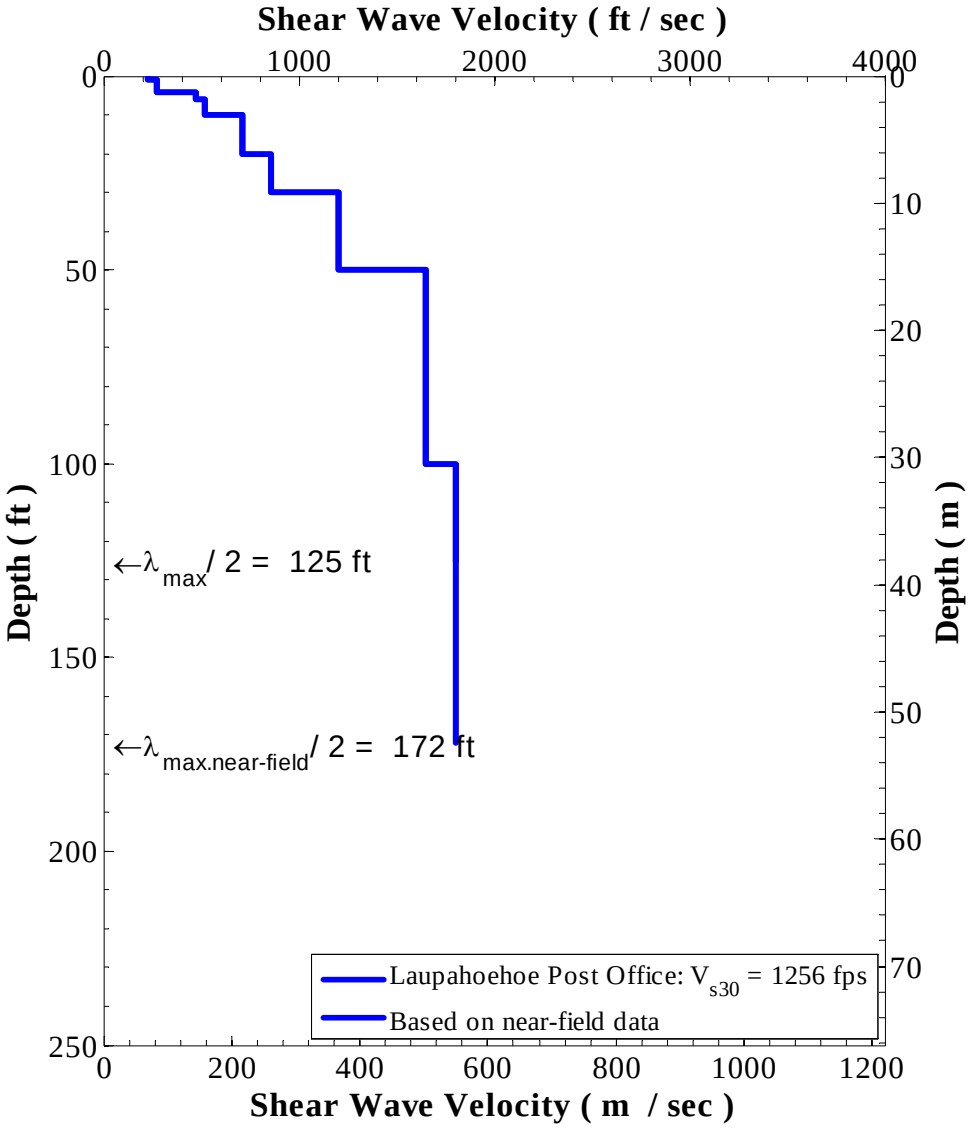
Honokaa Police Station

Profile Parameter for Honokaa Police Station

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	2	714.7	300	0.33	120
2	4	913.2	550	0.33	120
3	4	5000	630	0.4919	120
4	2	5000	600	0.4927	120
5	10	5000	880	0.484	120
6	20	5000	1280	0.4649	120
7	45.8	5000	1690	0.4355	120
8	80	5000	2600	0.3147	130
9	99999	5000	2800	0.2716	130



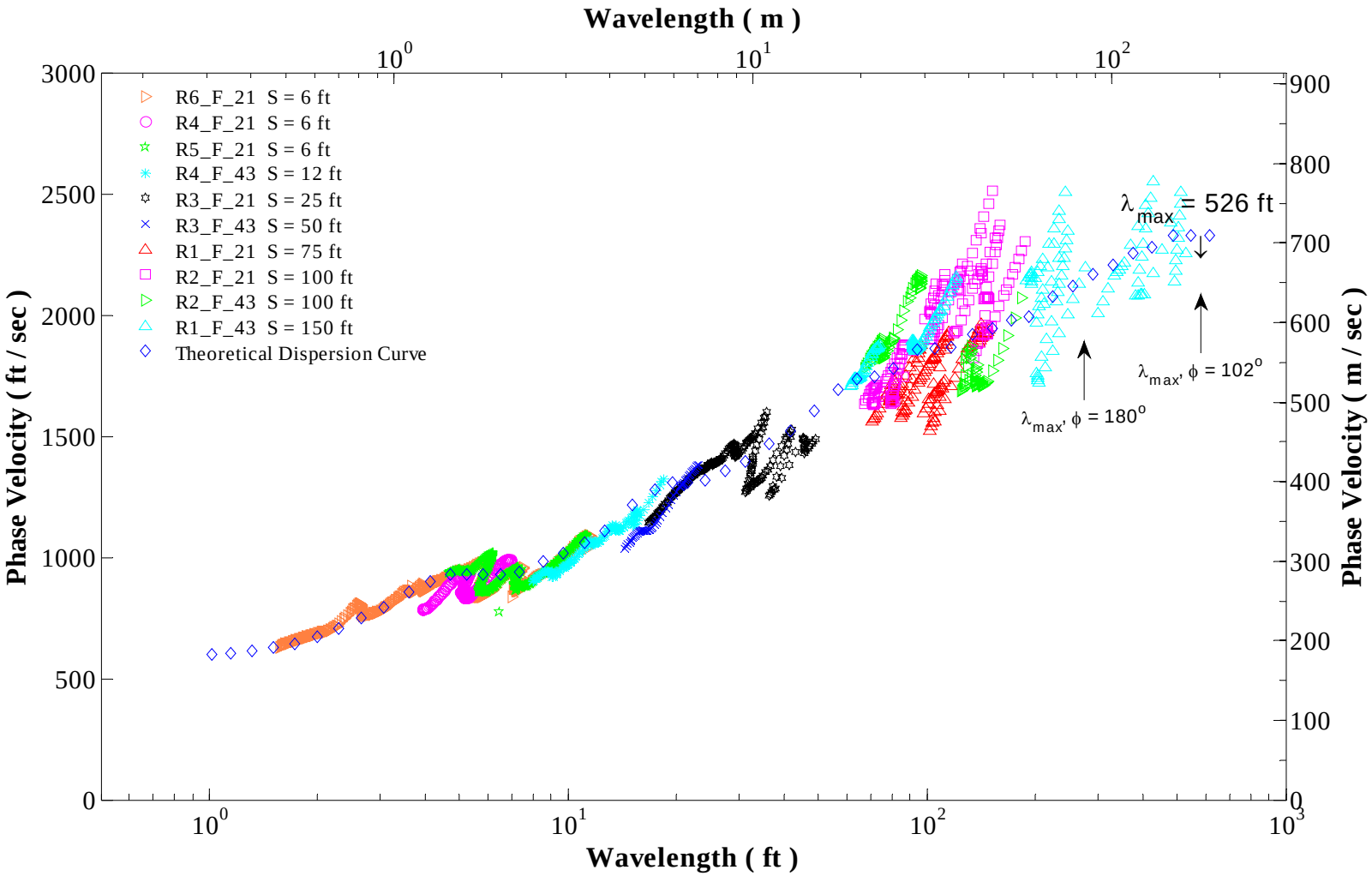
Laupahoehoe Post Office



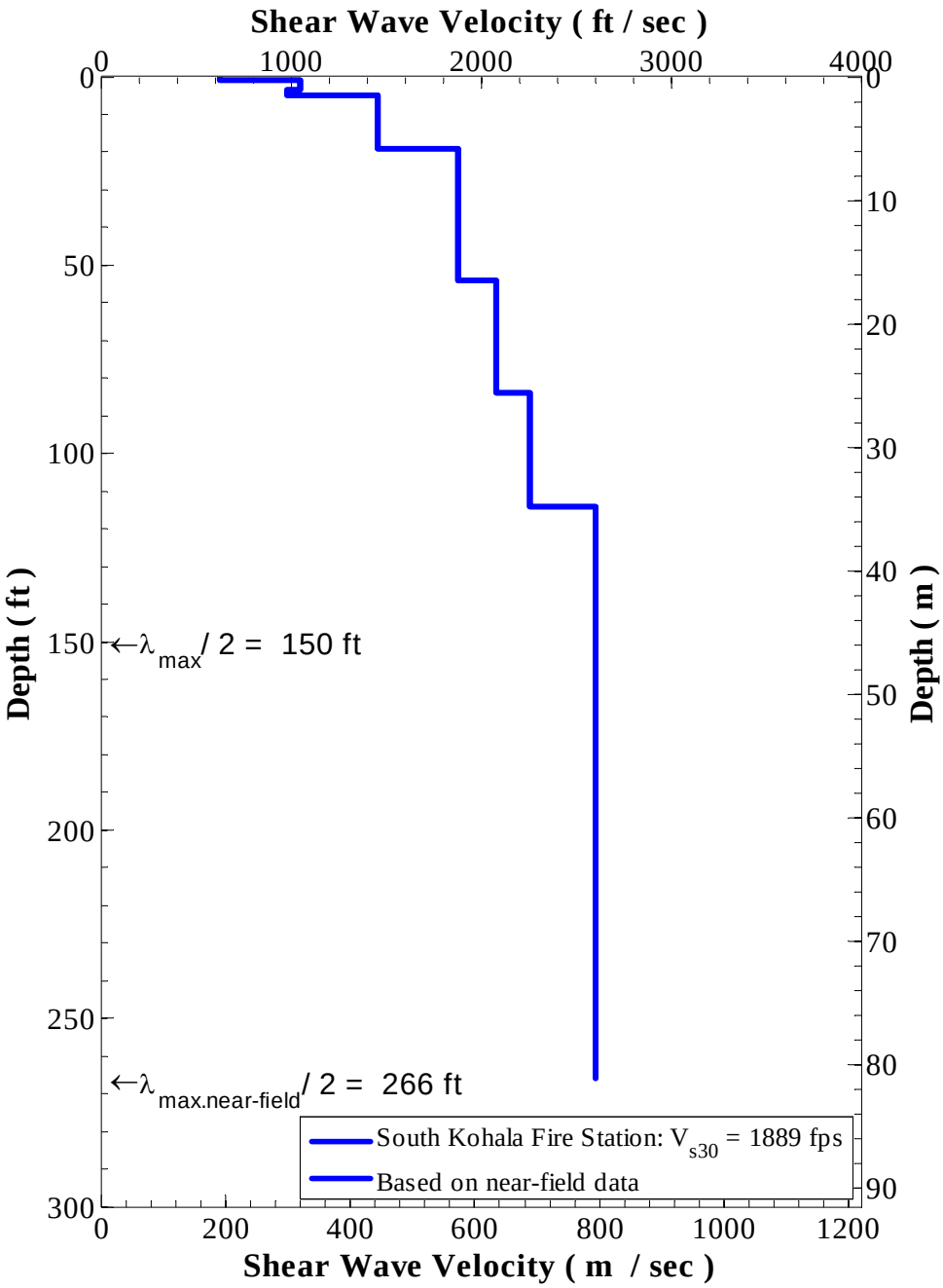
Laupahoehoe Post Office

Profile Parameter for Laupahoehoe Post Office

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	1	456.6	230	0.33	120
2	3	536	270	0.33	120
3	2	933.1	470	0.33	120
4	4	5000	520	0.4945	120
5	10	5000	710	0.4897	120
6	10	5000	860	0.4848	120
7	20	5000	1200	0.4694	120
8	50	5000	1650	0.4389	120
9	120	5000	1800	0.38	130
10	99999	5000	1900	0.3503	130



South Kohala Fire Station



South Kohala Fire Station

Profile Parameter for South Kohala Fire Station

Number of Layer	Thickness	Vp(fps)	Vs(fps)	Poisson's Ratio	Mass Density
1	1	1250.7	630	0.33	120
2	2.5	2084.5	1050	0.33	120
3	1.5	5000	980	0.48	120
4	14	5000	1460	0.4534	120
5	35	5000	1880	0.4177	120
6	30	5000	2080	0.3954	130
7	30	5000	2260	0.3716	130
8	99999	5000	2600	0.3147	130

Appendix D3
Primer on Coralline Liquefaction

D3.1 PRIMER ON CORALLINE LIQUEFACTION

The unique behavior of coralline seismic liquefaction has been a topic of both academic and practical interest in recent years (Mejia and Yeung 1995; Morioka and Nicholson 2000). Our understanding of the behavior of coralline seismic liquefaction can be traced to our general knowledge about non-classical coral material, which is more widely researched and documented (Anderson 1981; Noorany 1989; McClelland Engineers 1980).

The customary practice to evaluate liquefaction in Hawaii has been to use traditional quartz-based simplified standard penetration test (SPT) methods (Youd et al. 2001; Youd et al. 2002). This practice is corroborated by limited performance data in Guam and by the supposition that the cementation and platy shape of sand grains prove more resistant to liquefaction than quartz sands, based on shear wave velocity measurements in weakly cemented corals (Mejia and Yeung 1995; Nicholson 2006) and laboratory findings of higher cyclic shear resistance for equal relative densities (Morioka and Nicholson 1999).

However, the common presence of very loose and very weakly cemented corals with very high water contents and very low relative densities has resulted in numerous instances of construction-induced liquefaction in Hawaii and excessive static and dynamic foundation settlements (Francis 2005). Consequently, correlation attempts between static and seismic cone penetration test (CPT) data in coralline soils in Hawaii remain inconclusive, and in general liquefaction behavior is considered quite variable (Campanella 2001; Kiyoi and Campanella 2003; Nicholson 2006).

One coralline formation in Maui was shown to liquefy under lower cyclic stress than a quartz sand formation with similar shear wave velocity, and the shear wave velocity was shown to degrade significantly after liquefaction, indicating possible rupture of cementation (Rollins and Ashford 2004). International researchers have identified similar findings of problematic liquefaction behavior in coralline soils, primarily for sites in Australia and the Middle East (Abbs 1983; Angemeer et al. 1973;Randolph and Erbrich 2001; Randolph 2002). During the Kawaihae event, no liquefaction is known to have occurred outside of the areas of reclaimed soil at Kawaihae Harbor (Nicholson 2007).

Additional fundamental research is needed to better define when coral deposits are less or more resistant to liquefaction than measure by CPT or by fines content (Idriss and Boulanger 2004; Seed et al. 2003). In the interim, the use of shear wave velocity readings for liquefaction evaluation of corals is not recommended unless cementation behavior is well defined. Geotechnical data used for other liquefaction evaluation methods (SPT, CPT) should also be carefully considered against the potentially very loose and sensitive in situ skeletal matrix and highly variable cementation commonly found in low-energy coral deposits.

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Appendix D4
Coralline Liquefaction Tables and Figures

Table D4-1 Summary of Liquefaction Settlement

Summary of Calculated Liquefaction Settlement

Boring ID	Boring Location	Date of Exploration	Depth	Calculated Liquefaction Settlement			
				Total	Depth of 10ft	Depth of 20ft	Depth of 40ft
(-)	(-)	(year)	(ft)	(in)		(in)	(in)
B-8	Pier 2A Pavement	1999	20	1.9	0.02	1.9	-
B-18	Pier 2B Pavement	1988	10	9.2	9.20	-	-
B-28	Pier 2A Pavement	1988	25	4.2	1.22	3.3	-
B-35	Pier 2A Pavement	1989	45	2.2	0.08	2.2	2.2
B-36	Pier 1 Pavement	1989	35	0.0	0.00	0.0	0.0
B-A	Pier 2A	1990	40	4.3	2.68	3.5	4.3
B-B	Pier 2B	1990	20	0.1	0.00	0.1	-
B-E	Pier 2B Water	1990	40	7.5	1.37	3.7	7.5
B-F	Pier 1 Pavement	1990	25	1.4	1.19	1.2	-
		Average Settlement =		3.4	1.7	2.0	3.5
		Maximum Settlement =		9.2	9.2	3.7	7.5

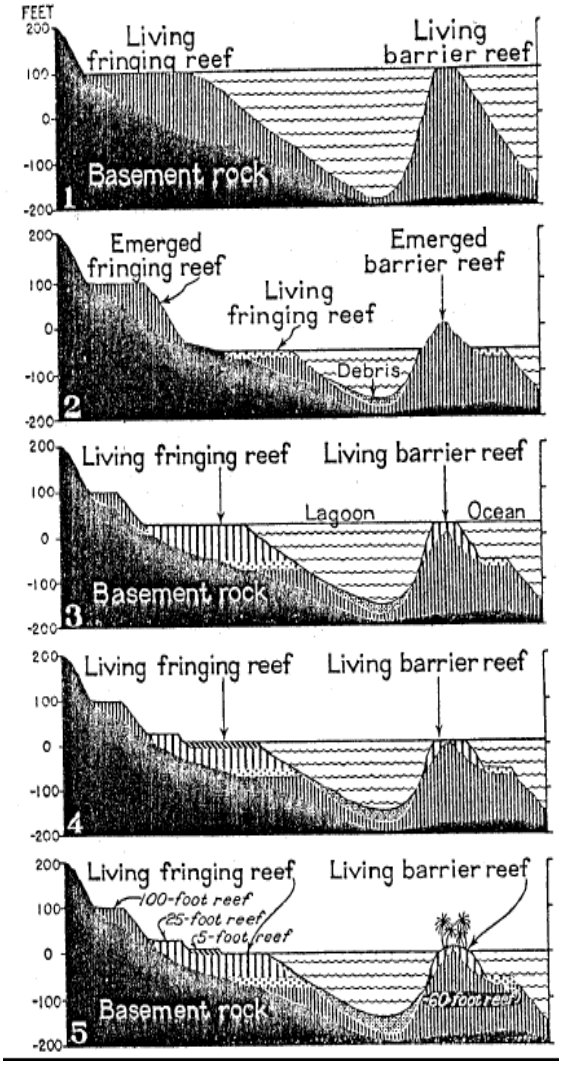
Sources: HC 5313; HC 5219; HC 5218.

Table D4-2 Lateral Spread Calculation (Kawaihae)

(Liquefaction Plots for Kawaihae Overseas Terminal [Pier 2B])

Empirical Regression to Estimate Magnitude of Liquefaction Induced Lateral Spread										Date:	2/26/2007
Ref. Bartlett & Youd, NCEL, Port Hueneme, 1992										By:	MJF
Site:	Kawaihae Harbor										
Location:	North Kona Coast, Hawaii										
Engineer:	URS										
Owner:	State of Hawaii, DOT Harbors Division										
Based upon Boring B-A to F(1990, EK Hirata), and B1 to 24 (1988, Geolabs). Liq layer from 0-								Limits:			
5	S, Slope in ground (%):							>20% may ground rupture / landslide			
4	W, Free Face Ratio - Height of FF / Distance to Base of FF (H/L) :							>20% may ground rupture / landslide			
32	R, Horizontal distance to seismic source (km):							see R limits table - >3km for M=7.5			
6.7	M, Earthquate Moment Magnitude:							or equivalent scaling			
6	T15, Cum. Thickness of Liq Layers (N1(60)<15) (m):							<15m			
0.1	DFifty15, Average mean grain size in T15 Liq Layer (mm):							0.08mm<DFifty15>1mm			
25	F15, Average % fines (<#200) in T15 Liq Layer:							<50%			
ft	*0.19*	**0.06**	Dh, Free Face Lateral Spread Distance (m)								
ft	*0.59*	**0.18**	Dh, Slope Lateral Spread Distance (m)								
Free Face Equation:											
LogDh = -16.3658 + 1.1782*M - 0.9275*LogR - 0.0133*R + 0.6572*LogW + 0.3483*LogT15 + 4.5270*Log(100-F15) - 0.9224*DFifty15											
Slope Equation:											
LogDh = -15.7870 + 1.1782*M - 0.9275*LogR - 0.0133*R + 0.4293*LogS + 0.3483*LogT15 + 4.5270*Log(100-F15) - 0.9224*DFifty15											

Source: Bartlett and Youd, NCEL, Port Hueneme, 1992.



CALCAREOUS ROCK CLASSIFICATION SYSTEM FOR HAWAII

Basic Types of Calcareous Rock

TYPE I — INSITU REEF ROCK

Composed largely of undisturbed *coralline skeletons. The skeletons are cemented together during accretion. The cementing agents are calcium compounds secreted by the marine organisms. The joined skeletons form a structural framework. Coralline and other calcareous debris commonly fill the framework interstices.

TYPE II — SECONDARY ROCK

Composed essentially of cemented fragments of coralline skeletons and/or calcareous shells. Cementation is believed to occur primarily after accretion is completed and the marine deposit begins to emerge above sea level. The common types of secondary rock are listed below.

1. Conglomerate - Cemented, non-uniform sand- and gravel-size particles of cementd coralline skeletons and/or calcareous shells. The skeletons and shells are originally deposited in near-shore waters. The cementing agent is calcium carbonate precipitated primarily from percolating ground water.

2. Shell Rock - Cemented shells and shell fragments that have accumulated in protected shallow sea water. The shells are often cemented in a clay- and silt-size matrix. Cementation develops from calcium carbonate deposited by ground water.

3. Dune Rock - Cemented dune sand. The cementing agent is generally calcium carbonate precipitated from percolating ground water. Dune rock generally has a relatively low density because the constituent sand grains are loosely packed and often poorly cemented.

TYPE III — CHALK

Poorly cemented mass of calcareous clay- and silt-size particles, believed to be precipitated from shallow sea water and associated with near-shore environments.

* The term coralline is used to indicate coral and /or other calcium compound secreting organisms.

o o o

Figure D4-1 Stages of Reef Development and Classification of Coral Material

Sources: Stearns 1960; Anderson 1981.

North Rankin Data

Low skin friction

Low modulus

Low overburden effects

CONDUCTOR LOAD TEST DATA

CONDUCTOR TESTED	DEPTH BELOW MUDLINE (m)	PEAK SKIN FRICTION (kPa)	RESIDUAL SKIN FRICTION (kPa)	DATE
NW5	63.7	14.5	8.9	SEPT 1983
CE1	112.5	7.5	5.6	OCT 1983
CE1	113.0	4.6	3.0	FEB 1984 (111 days set up)
NE2-TEST 1	111.6	7.6*	-	SEPT 1984
NE2-TEST 1A	111.6	18.6	-	SEPT 1984
NE2-TEST 2	116.5	4.1	1.4 - 2.0	SEPT 1984
SE3	112.5	15.6*	-	OCT 1984

*No failure.

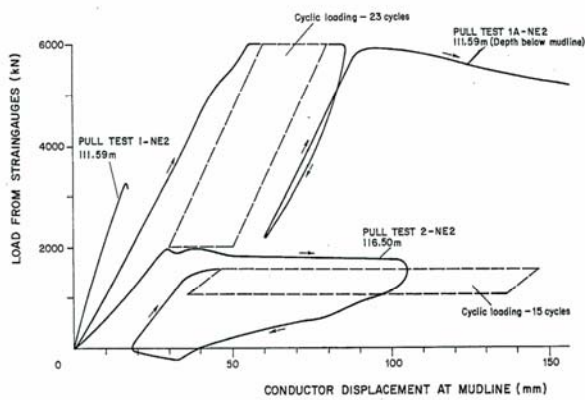


Figure 1 Conductor Pull Out Test Results - Conductor NE2

Figure D4-2 Pile Test Data in Coral Materials

Source: Abbs et al. 1988.

Cementation Study of Coral

Shearing of Equal Strength Cementations:

- Calcite bonds contract
- Portland cement bonds dilate

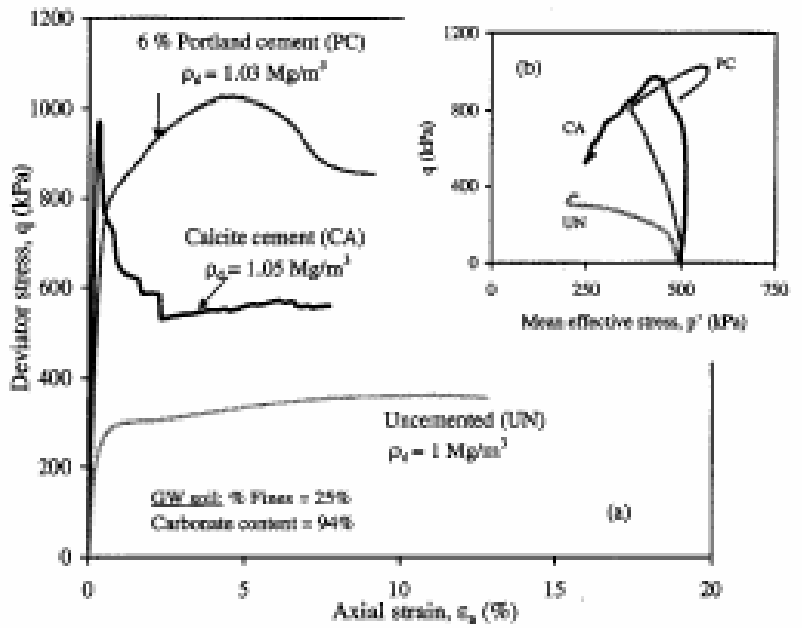


Fig. 1. Response of Goodwy soil (GW) in CIU test at σ'_{3c} = 500 kPa for different cementing agents: (a) stress-strain curves and (b) stress paths

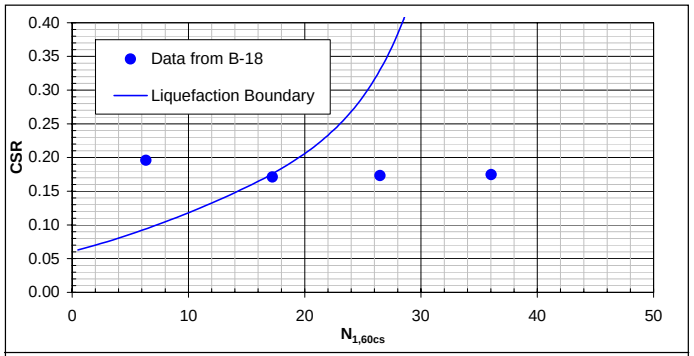
Figure D4-3 Cementation Study of Coral

Source: Ismail, Randolph et al. JGGE, June 2002.

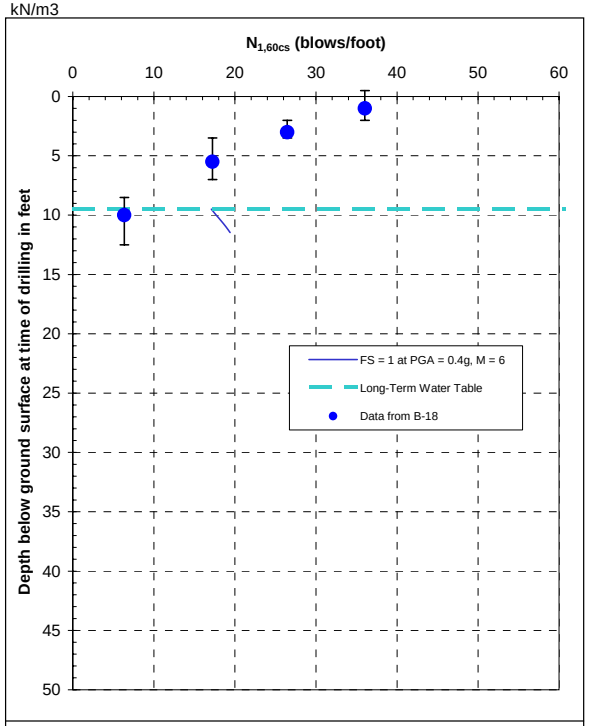
Project: New Haven: Kawaihae Harbor — Boring B-18 (1988) — System of Units pound-force, psf, ft, s — M. Luebbers 2007
Project No.: 15298973 — Unit weight of water 9.798 kN/m3 — Idriss & Boulanger 2004

URS

Borehole elevation	10.8	ft
Borehole diameter	3.25	inches
Energy transfer, %	45	%
Energy transfer ratio	0.75	
Depth to GW during investigation (ATD)	8	ft
Depth to GW for analysis (long term)	9.5	ft
amax, in g, of Earthquake 1 (EQ1)	0.4	g
Magnitude of Earthquake 1	6	
amax, in g, of Earthquake 2 (EQ2)	0.4	g
Magnitude of Earthquake 2	6	
Sampler ratio, Cs, for "Other non-SPT sampler"	1	
Residual Strength Range (0 to 1)	0.33	decimal, from 0 to 1
Height of long-term surface above the surface ATD	0	ft
Total unit weight of fill	135	pcf



Cyclic Stress Ratio vs. "Clean Sand" Corrected Blow Counts, $N_{1,60cs}$



'Clean Sand' Corrected Blow Counts versus Depth - Borehole B-18

Depth ATD ft	Elev m	Blow Count	Sampler Type (M,S)	Fines -#200	Top of Layer (opt)	Bottom of Layer (opt)	Ko	$N_{1,60}$	$N_{1,60,cs}$	Factor of Safety Eq 1	Factor of Safety Eq 2	Settlement in	PI	LL	wc %	Zone
1	-2.99	33	SPT_1.375_ID	20	0	2.5	0.5	31.6	36.0	7.94	7.94	0.000	-	-	25	Silty Sand
3	-2.38	23	SPT_1.375_ID	20	2.5	4	0.5	22.0	26.5	1.90	1.90	0.000	-	-	25	Silty Sand
5.5	-1.62	10	SPT_1.375_ID	20	4	7.5	0.5	12.8	17.2	1.03	1.03	0.026	-	-	25	Silty Sand
10	-0.24	1	SPT_1.375_ID	25	7.5	11.5	0.5	1.3	6.3	0.48	0.48	9.216	-	-	25	Silty Sand

Total Settlement = 9.24

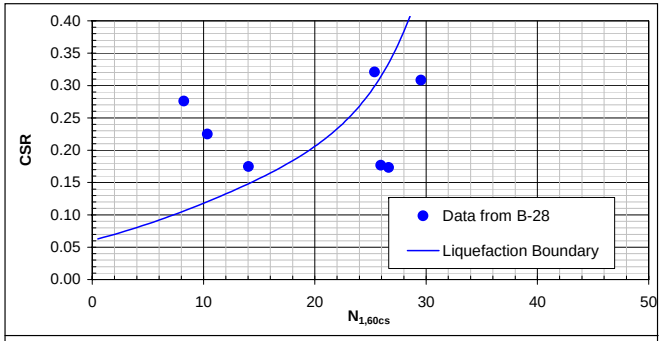
Figure D4-4 Liquefaction Settlement Calculations for B-18 (1988)

Source: Idriss and Boulanger 2004.

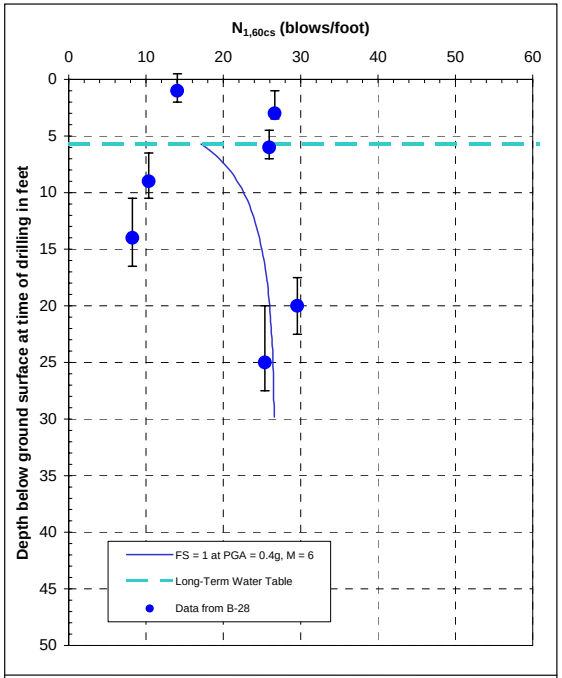
Project: Kawaihae Harbor H.C. 5218 Boring B-28 (1988) System of Units pound-force, psf, ft, s M. Luebbers 2007
Project No.: 15298973 Unit weight of water 9.798 kN/m3 Idriss & Boulanger 2004

URS

Borehole elevation	6	ft
Borehole diameter	3.25	inches
Energy transfer, %	50	%
Energy transfer ratio	0.83	
Depth to GW during investigation (ATD)	5.7	ft
Depth to GW for analysis (long term)	5.7	ft
amax, in g, of Earthquake 1 (EQ1)	0.4	g
Magnitude of Earthquake 1	6	
amax, in g, of Earthquake 2 (EQ2)	0.4	g
Magnitude of Earthquake 2	6	
Sampler ratio, Cs, for "Other non-SPT sampler"	1	
Residual Strength Range (0 to 1)	0.33	decimal, from 0 to 1
Height of long-term surface above the surface ATD	0	ft
Total unit weight of fill	135	pcf



Cyclic Stress Ratio vs. "Clean Sand" Corrected Blow Counts, $N_{1,60cs}$



'Clean Sand' Corrected Blow Counts versus Depth - Borehole B-28

Depth ATD ft	Elev m	Blow Count	Sampler Type (M,S)	Fines -#200	Top of Layer (opt)	Bottom of Layer (opt)	Ko	$N_{1,60}$	$N_{1,60,cs}$	Factor of Safety Eq 1	Factor of Safety Eq 2	Settlement inches	PI	LL	wc %	Zone
1	-1.52	9	SPT_1.375_ID	20	0	2.5	0.5	9.6	14.0	0.85	0.85	0.000	-	-	9	Silty Sand
3	-0.91	22	SPT_1.375_ID	15	2.5	5	0.5	23.4	26.6	1.93	1.93	0.000	-	-	22	Silty Sand
6	0.00	16	SPT_1.375_ID	15	5	7.5	0.5	22.7	25.9	1.77	1.77	0.002	-	-	15	Silty Sand
9	0.91	5	SPT_1.375_ID	15	7.5	11.5	0.5	7.1	10.3	0.54	0.54	1.217	-	-	24	Silty Sand
14	2.44	5	SPT_1.375_ID	10	11.5	17.5	0.5	7.1	8.2	0.38	0.38	2.056	-	-	28	Silty Sand
20	4.27	26	SPT_1.375_ID	5	17.5	22.5	0.5	29.5	29.5	1.49	1.49	0.000	-	-	21	Sand
25	5.79	23	SPT_1.375_ID	5	22.5	30	0.5	25.4	25.4	0.93	0.93	0.919	-	-	26	Sand
											Total Settlement	4.19				

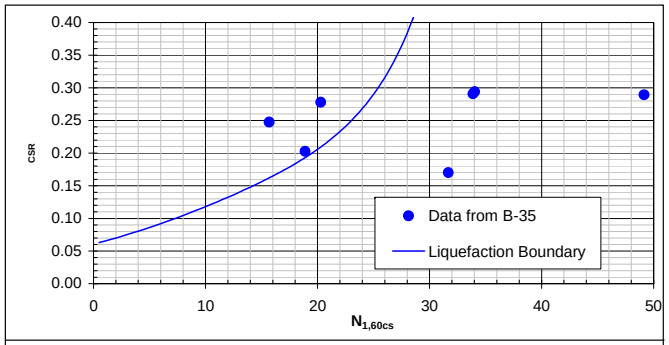
Figure D4-5 Liquefaction Settlement Calculations for B-28 (1988)

Source: Idriss and Boulanger 2004.

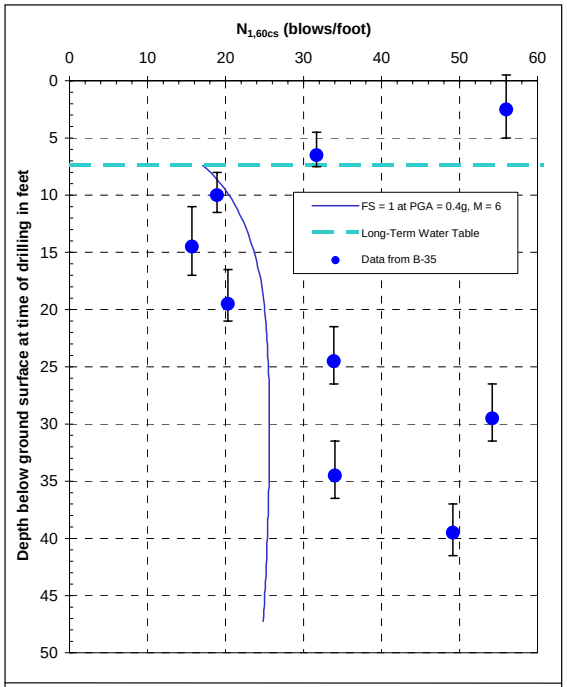
Project: Kawaihae Harbor H.C. 5218 Boring B-35 (1989) System of Units pound-force, psf, ft, s M. Luebbers 2007
Project No.: 15298973 Unit weight of water 9.798 kN/m3 Idriss & Boulanger 2004

URS

Borehole elevation	8.5	ft
Borehole diameter	3.25	inches
Energy transfer, %	50	%
Energy transfer ratio	0.83	
Depth to GW during investigation (ATD)	7.5	ft
Depth to GW for analysis (long term)	7.4	ft
amax, in g, of Earthquake 1 (EQ1)	0.4	g
Magnitude of Earthquake 1	6	
amax, in g, of Earthquake 2 (EQ2)	0.4	g
Magnitude of Earthquake 2	6	
Sampler ratio, Cs, for "Other non-SPT sampler"	1	
Residual Strength Range (0 to 1)	0.33	decimal, from 0 to 1
Height of long-term surface above the surface ATD	0	ft
Total unit weight of fill	135	pcf



Cyclic Stress Ratio vs. "Clean Sand" Corrected Blow Counts, $N_{1,60cs}$



'Clean Sand' Corrected Blow Counts versus Depth - Borehole B-35

Depth ATD ft	Elev m	Blow Count	Sampler Type (M,S)	Fines -#200	Top of Layer (opt)	Bottom of Layer (opt)	Ko	$N_{1,60}$	$N_{1,60,cs}$	Factor of Safety Eq 1	Factor of Safety Eq 2	Settlement inches	PI	LL	wc %	Zone
2.5	-1.83	56	SPT_1.375_ID	5	0	5.5	0.5	56.0	56.0	>10	>10	0.000	-	-	29	Gravel
6.5	-0.61	23	SPT_1.375_ID	10	5.5	8.5	0.5	30.5	31.7	3.60	3.60	0.003	-	-	27	Sand
10	0.46	13	SPT_1.375_ID	10	8.5	12	0.5	17.8	18.9	0.95	0.95	0.152	-	-	32	Sand
14.5	1.83	11	SPT_1.375_ID	10	12	18	0.5	14.5	15.7	0.65	0.65	1.271	-	-	36	Sand
19.5	3.35	13	SPT_1.375_ID	20	18	22.5	0.5	15.8	20.3	0.75	0.75	0.721	-	-	36	Silty Sand
24.5	4.88	28	SPT_1.375_ID	20	22.5	27.5	0.5	29.4	33.9	3.05	3.05	0.000	-	-	32	Silty Sand
29.5	6.40	53	SPT_1.375_ID	20	27.5	32.5	0.5	49.7	54.2	>10	>10	0.000	-	-	32	Silty Sand
34.5	7.92	31	SPT_1.375_ID	20	32.5	37.5	0.5	29.5	34.0	3.10	3.10	0.000	-	-	27	Silty Sand
39.5	9.45	50	SPT_1.375_ID	20	37.5	42	0.5	44.7	49.1	>10	>10	0.000	-	-	26	Silty Sand
44.5	10.97	91	SPT_1.375_ID	5	42	47.5	0.5	77.1	77.1	>10	>10	0.000	-	-	29	Sand

Total Settlement 2.15

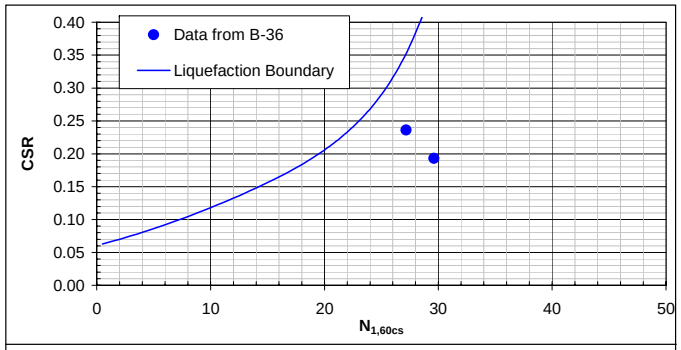
Figure D4-6 Liquefaction Settlement Calculations for B-35 (1989)

Source: Idriss and Boulanger 2004.

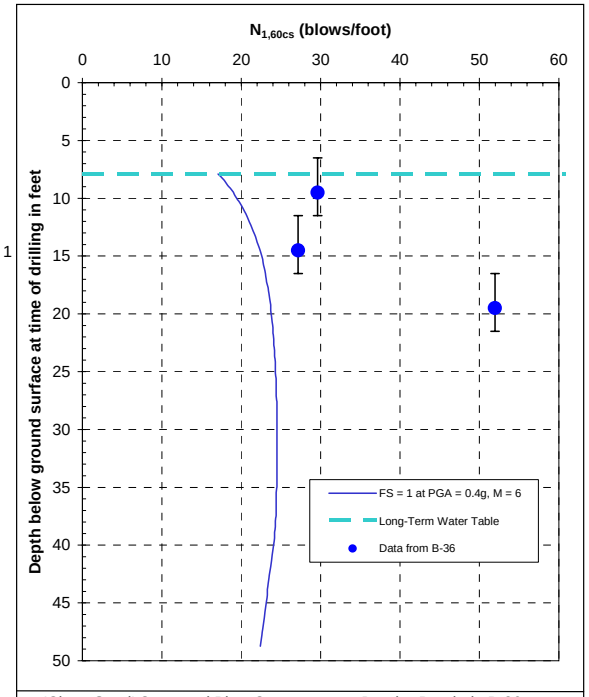
Project: Kawaihae Harbor H.C. 5218 Boring B-36 (1989) System of Units pound-force, psf, ft, s M. Luebbers 2007
Project No.: 15298973 Unit weight of water 9.798 kN/m3 Idriss & Boulanger 2004

URS

Borehole elevation	9	ft
Borehole diameter	3.25	inches
Energy transfer, %	50	%
Energy transfer ratio	0.83	
Depth to GW during investigation (ATD)	7.5	ft
Depth to GW for analysis (long term)	7.9	ft
amax, in g, of Earthquake 1 (EQ1)	0.4	g
Magnitude of Earthquake 1	6	
amax, in g, of Earthquake 2 (EQ2)	0.4	g
Magnitude of Earthquake 2	6	
Sampler ratio, Cs, for "Other non-SPT sampler"	1	
Residual Strength Range (0 to 1)	0.33	decimal, from 0 to 1
Height of long-term surface above the surface ATD	0	ft
Total unit weight of fill	135	pcf



Cyclic Stress Ratio vs. "Clean Sand" Corrected Blow Counts, $N_{1,60cs}$



'Clean Sand' Corrected Blow Counts versus Depth - Borehole B-36

Depth ATD ft	Elev m	Blow Count	Sampler Type (M,S)	Fines -#200	Top of Layer (opt)	Bottom of Layer (opt)	Ko	$N_{1,60}$	$N_{1,60,cs}$	Factor of Safety Eq 1	Factor of Safety Eq 2	Settlement inches	PI	LL	wc %	Zone
1.5	-2.29	60	SPT_1.375_ID	13	0	2.5	0.5	61.0	63.5	>10	>10	0.000	-	-	18	Gravel
5.5	-1.07	74	SPT_1.375_ID	13	2.5	7.5	0.5	73.1	75.6	>10	>10	0.000	-	-	18	Gravel
9.5	0.15	25	SPT_1.375_ID	5	7.5	12.5	0.5	29.6	29.6	2.39	2.39	0.001	-	-	27	Gravel
14.5	1.68	24	SPT_1.375_ID	5	12.5	17.5	0.5	27.2	27.2	1.49	1.49	0.000	-	-	30	Sand
19.5	3.20	51	SPT_1.375_ID	13	17.5	22.5	0.5	49.5	52.0	>10	>10	0.000	-	-	27	Sand
24.5	4.72	82	SPT_1.375_ID	13	22.5	27.5	0.5	72.6	75.1	>10	>10	0.000	-	-	31	Sand
29.5	6.25	97	SPT_1.375_ID	13	27.5	32.5	0.5	83.1	85.6	>10	>10	0.000	-	-	29	Sand
34.5	7.77	86	SPT_1.375_ID	5	32.5	38.5	0.5	73.6	73.7	>10	>10	0.000	-	-	25	Sand
											Total Settlement	0.00				

Figure D4-7 Liquefaction Settlement Calculations for B-36 (1989)

Source: Idriss and Boulanger 2004.

Project: Kawaihae Harbor
Project No.: 15298973

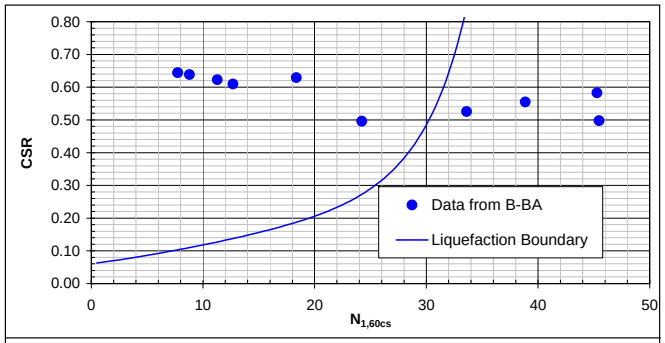
Boring BA (1990)

System of Units pound-force, psf, ft, s
Unit weight of water 9.798 kN/m3

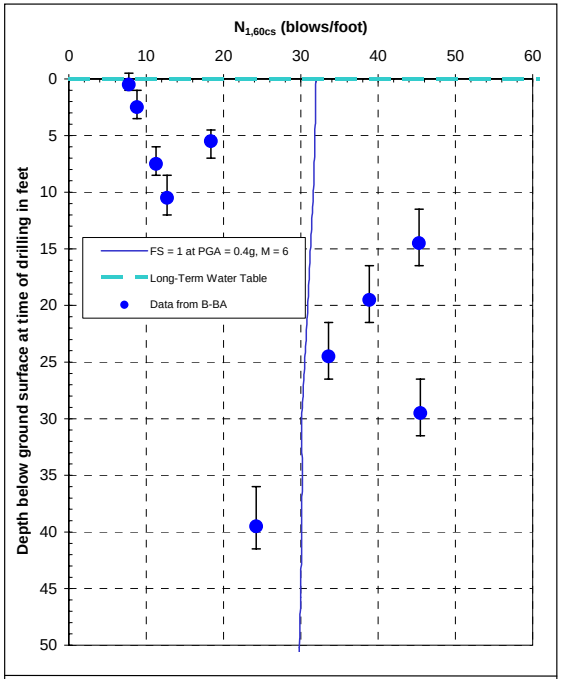
M. Luebbers 2007
Idriss & Boulanger 2004

URS

Borehole elevation	17	ft
Borehole diameter	3.25	inches
Energy transfer, %	50	%
Energy transfer ratio	0.83	
Depth to GW during investigation (ATD)	0	ft
Depth to GW for analysis (long term)	0	ft
amax, in g, of Earthquake 1 (EQ1)	0.4	g
Magnitude of Earthquake 1	6	
amax, in g, of Earthquake 2 (EQ2)	0.4	g
Magnitude of Earthquake 2	6	
Sampler ratio, Cs, for "Other non-SPT sampler"	1	
Residual Strength Range (0 to 1)	0.33	decimal, from 0 to 1
Height of long-term surface above the surface ATD	0	ft
Total unit weight of fill	135	pcf



Cyclic Stress Ratio vs. "Clean Sand" Corrected Blow Counts, $N_{1,60cs}$



'Clean Sand' Corrected Blow Counts versus Depth - Borehole B-BA

Depth ATD ft	Elev m	Blow Count	Sampler Type (M,S)	Fines -#200	Top of Layer (opt)	Bottom of Layer (opt)	Ko	$N_{1,60}$	$N_{1,60,cs}$	Factor of Safety Eq 1	Factor of Safety Eq 2	Settlement inches	PI	LL	wc %	Zone
0.5	-5.03	2	SPT_1.375_ID	55	0	1.5	0.5	2.1	7.7	0.16	0.16	0.528	-	-	38	Silty Sand
2.5	-4.42	3	SPT_1.375_ID	55	1.5	4	0.5	3.2	8.8	0.17	0.17	0.831	-	-	30	Silty Sand
5.5	-3.51	9	SPT_1.375_ID	55	4	6.5	0.5	12.8	18.4	0.30	0.30	0.504	-	-	30	Silty Sand
7.5	-2.90	4	SPT_1.375_ID	55	6.5	9	0.5	5.7	11.3	0.20	0.20	0.718	-	-	28	Silty Sand
10.5	-1.98	5	SPT_1.375_ID	55	9	12.5	0.5	7.1	12.7	0.23	0.23	0.914	-	-	34	Silty Sand
14.5	-0.76	28	SPT_1.375_ID	55	12.5	17.5	0.5	39.7	45.3	>10	>10	0.000	-	-	40	Silty Sand
19.5	0.76	24	SPT_1.375_ID	55	17.5	22.5	0.5	33.3	38.9	5.25	5.25	0.000	-	-	33	Silty Sand
24.5	2.29	21	SPT_1.375_ID	55	22.5	27.5	0.5	28.0	33.6	1.61	1.61	0.000	-	-	44	Silty Sand
29.5	3.81	35	SPT_1.375_ID	55	27.5	32.5	0.5	39.9	45.5	>10	>10	0.000	-	-	33	Silty Sand
34.5	5.33	62	SPT_1.375_ID	55	32.5	37.5	0.5	61.5	67.1	>10	>10	0.000	-	-	30	Silty Sand
39.5	6.86	15	SPT_1.375_ID	55	37.5	43	0.5	18.6	24.2	0.55	0.55	0.836	-	-	49	Silty Sand
											Total Settlement	4.33				

Figure D4-8 Liquefaction Settlement Calculations for B-A (1990)

Source: Idriss and Boulanger 2004.

Project: Kawaihae Harbor
Project No.: 15298973

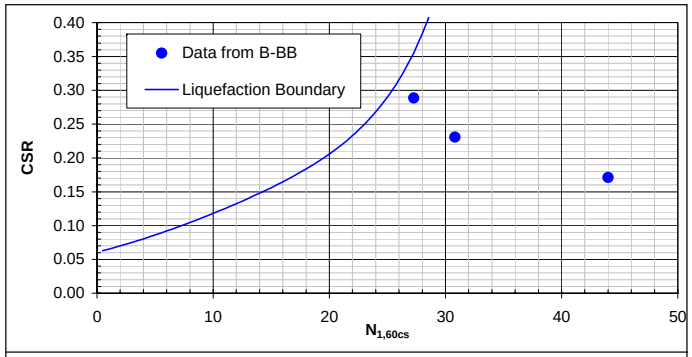
Boring BB (1990)

System of Units pound-force, psf, ft, s
Unit weight of water 9.798 kN/m3

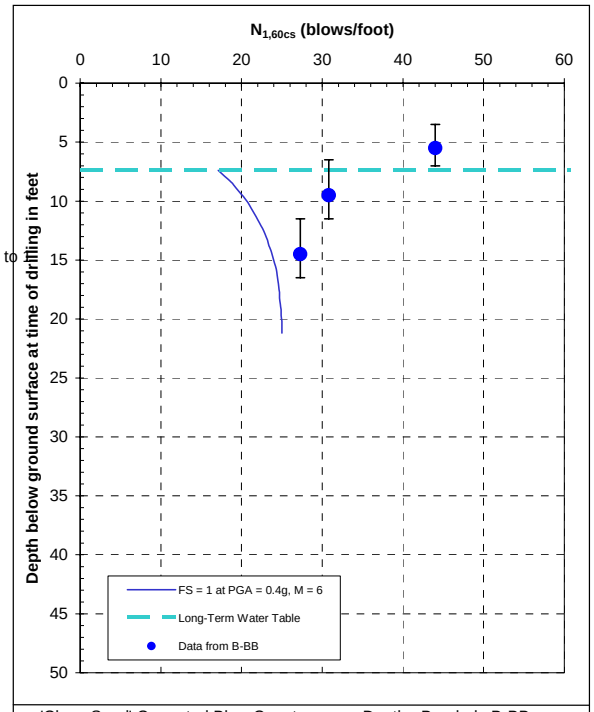
M. Luebbers 2007
Idriss & Boulanger 2004

URS

Borehole elevation	5	ft
Borehole diameter	3.25	inches
Energy transfer, %	50	%
Energy transfer ratio	0.83	
Depth to GW during investigation (ATD)	6	ft
Depth to GW for analysis (long term)	7.4	ft
amax, in g, of Earthquake 1 (EQ1)	0.4	g
Magnitude of Earthquake 1	6	
amax, in g, of Earthquake 2 (EQ2)	0.4	g
Magnitude of Earthquake 2	6	
Sampler ratio, Cs, for "Other non-SPT sampler"	1	
Residual Strength Range (0 to 1)	0.33	decimal, from 0 to
Height of long-term surface above the surface ATD	0	ft
Total unit weight of fill	135	pcf



Cyclic Stress Ratio vs. "Clean Sand" Corrected Blow Counts, $N_{1,60cs}$



'Clean Sand' Corrected Blow Counts versus Depth - Borehole B-BB

Depth ATD ft	Elev m	Blow Count	Sampler Type (M,S)	Fines -#200	Top of Layer (opt)	Bottom of Layer (opt)	Ko	$N_{1,60}$	$N_{1,60,cs}$	Factor of Safety Eq 1	Factor of Safety Eq 2	Settlement inches	PI	LL	wc %	Zone
2.5	-0.76	75	SPT_1.375_ID	20	0	4	0.5	67.1	71.5	>10	>10	0.000	-	-	9	Sand
5.5	0.15	30	SPT_1.375_ID	20	4	7.5	0.5	39.5	44.0	>10	>10	0.002	-	-	12	Sand
9.5	1.37	18	SPT_1.375_ID	55	7.5	12.5	0.5	25.2	30.8	2.34	2.34	0.000	-	-	47	Silty Sand
14.5	2.90	16	SPT_1.375_ID	55	12.5	17.5	0.5	21.7	27.3	1.23	1.23	0.103	-	-	27	Silty Sand
19.5	4.42	60	SPT_1.375_ID	25	17.5	21.5	0.5	59.3	64.4	>10	>10	0.000	-	-	25	Sand

Total Settlement = 0.11

Figure D4-9 Liquefaction Settlement Calculations for B-B (1990)

Source: Idriss and Boulanger 2004.

Project: Kawaihae Harbor
Project No.: 15298973

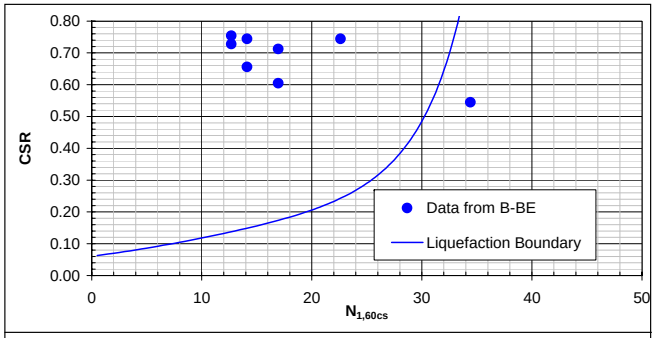
Boring BE (1990)

System of Units pound-force, psf, ft, s
Unit weight of water 9.798 kN/m3

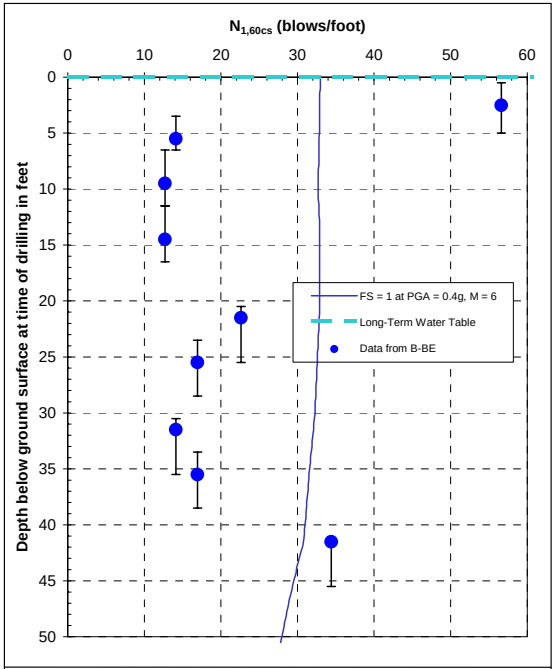
M. Luebbers 2007
Idriss & Boulanger 2004

URS

Borehole elevation	-22	ft
Borehole diameter	3.25	inches
Energy transfer, %	50	%
Energy transfer ratio	0.83	
Depth to GW during investigation (ATD)	0	ft
Depth to GW for analysis (long term)	0	ft
amax, in g, of Earthquake 1 (EQ1)	0.4	g
Magnitude of Earthquake 1	6	
amax, in g, of Earthquake 2 (EQ2)	0.4	g
Magnitude of Earthquake 2	6	
Sampler ratio, Cs, for "Other non-SPT sampler"	1	
Residual Strength Range (0 to 1)	0.33	decimal, from 0 to 1
Height of long-term surface above the surface ATD	0	ft
Total unit weight of fill	135	pcf



Cyclic Stress Ratio vs. "Clean Sand" Corrected Blow Counts, $N_{1,60cs}$



'Clean Sand' Corrected Blow Counts versus Depth - Borehole B-BE

Depth ATD ft	Elev m	Blow Count	Sampler Type (M,S)	Fines -#200	Top of Layer (opt)	Bottom of Layer (opt)	Ko	$N_{1,60}$	$N_{1,60,cs}$	Factor of Safety Eq 1	Factor of Safety Eq 2	Settlement inches	PI	LL	wc %	Zone
2.5	7.47	48	SPT_1.375_ID	55	0	4.5	0.5	51.0	56.6	>10	>10	0.000	-	-	26	Silty Sand
5.5	8.38	6	SPT_1.375_ID	55	4.5	7.5	0.5	8.5	14.1	0.20	0.20	0.714	-	-	35	Silty Sand
9.5	9.60	5	SPT_1.375_ID	55	7.5	12.5	0.5	7.1	12.7	0.19	0.19	1.306	-	-	40	Silty Sand
14.5	11.13	5	SPT_1.375_ID	55	12.5	17.5	0.5	7.1	12.7	0.18	0.18	1.306	-	-	46	Silty Sand
21.5	13.26	12	SPT_1.375_ID	55	17.5	22.5	0.5	17.0	22.6	0.33	0.33	0.829	-	-	49	Silty Sand
25.5	14.48	8	SPT_1.375_ID	55	22.5	27.5	0.5	11.3	16.9	0.24	0.24	1.069	-	-	45	Silty Sand
31.5	16.31	6	SPT_1.375_ID	55	27.5	32.5	0.5	8.5	14.1	0.23	0.23	1.189	-	-	39	Silty Sand
35.5	17.53	8	SPT_1.375_ID	55	32.5	37.5	0.5	11.3	16.9	0.29	0.29	1.069	-	-	35	Silty Sand
41.5	19.35	24	SPT_1.375_ID	55	37.5	41.5	0.5	28.8	34.4	1.81	1.81	0.000	-	-	41	Silty Sand
											Total Settlement	7.48				

Figure D4-10 Liquefaction Settlement Calculations for B-E (1990)

Source: Idriss and Boulanger 2004.

Project: Kawaihae Harbor
Project No.: 15298973

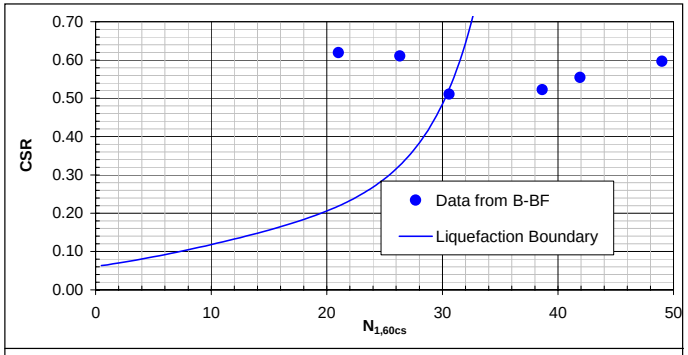
Boring BF (1990)

System of Units pound-force, psf, ft, s
Unit weight of water 9.798 kN/m3

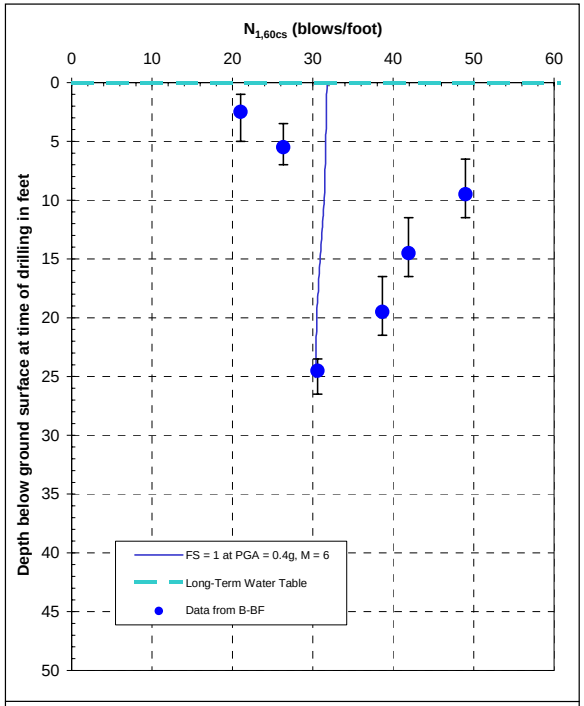
M. Luebbers 2007
Idriss & Boulanger 2004

URS

Borehole elevation	-10	ft
Borehole diameter	3.25	inches
Energy transfer, %	50	%
Energy transfer ratio	0.83	
Depth to GW during investigation (ATD)	0	ft
Depth to GW for analysis (long term)	0	ft
amax, in g, of Earthquake 1 (EQ1)	0.4	g
Magnitude of Earthquake 1	6	
amax, in g, of Earthquake 2 (EQ2)	0.4	g
Magnitude of Earthquake 2	6	
Sampler ratio, Cs, for "Other non-SPT sampler"	1	
Residual Strength Range (0 to 1)	0.33	decimal, from 0 to 1
Height of long-term surface above the surface ATD	0	ft
Total unit weight of fill	135	pcf



Cyclic Stress Ratio vs. "Clean Sand" Corrected Blow Counts, $N_{1,60cs}$



'Clean Sand' Corrected Blow Counts versus Depth - Borehole B-BF

Depth ATD ft	Elev m	Blow Count	Sampler Type (M,S)	Fines -#200	Top of Layer (opt)	Bottom of Layer (opt)	Ko	$N_{1,60}$	$N_{1,60,cs}$	Factor of Safety Eq 1	Factor of Safety Eq 2	Settlement inches	PI	LL	wc %	Zone
2.5	3.81	15	SPT_1.375_ID	25	0	4	0.5	15.9	21.0	0.35	0.35	0.717	-	-	36	Silty Sand
5.5	4.72	15	SPT_1.375_ID	25	4	7.5	0.5	21.3	26.3	0.53	0.53	0.470	-	-	30	Silty Sand
9.5	5.94	31	SPT_1.375_ID	25	7.5	12.5	0.5	43.9	49.0	>10	>10	0.000	-	-	28	Silty Sand
14.5	7.47	26	SPT_1.375_ID	25	12.5	17.5	0.5	36.8	41.9	>10	>10	0.000	-	-	27	Silty Sand
19.5	8.99	25	SPT_1.375_ID	25	17.5	22.5	0.5	33.6	38.6	5.21	5.21	0.000	-	-	31	Silty Sand
24.5	10.52	19	SPT_1.375_ID	25	22.5	25.5	0.5	25.5	30.6	1.03	1.03	0.254	-	-	30	Silty Sand

Total Settlement = 1.44

Figure D4-11 Liquefaction Settlement Calculations for B-F (1990)

Source: Idriss and Boulanger 2004.

Project: Kawaihae Harbor
Project No. 15298973

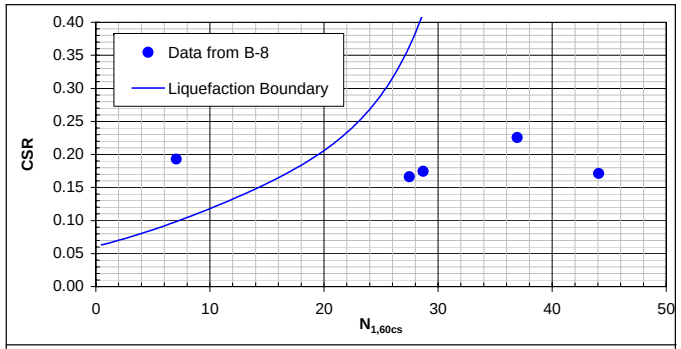
Boring B-8 (1999)

System of Units pound-force, psf, ft, s
Unit weight of water 9.798 kN/m3

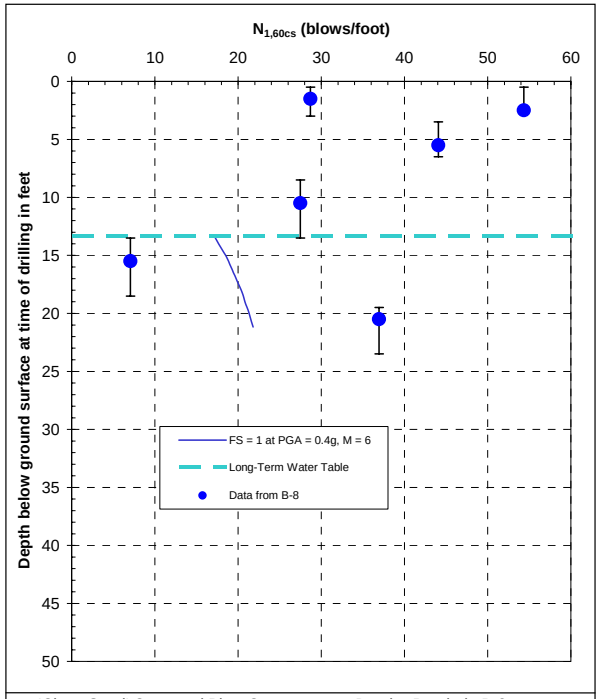
M. Luebbers 2007
Idriss & Boulanger 2004

URS

Borehole elevation	14.4	ft
Borehole diameter	3.25	inches
Energy transfer, %	50	%
Energy transfer ratio	0.83	
Depth to GW during investigation (ATD)	12	ft
Depth to GW for analysis (long term)	13.3	ft
amax, in g, of Earthquake 1 (EQ1)	0.4	g
Magnitude of Earthquake 1	6	
amax, in g, of Earthquake 2 (EQ2)	0.4	g
Magnitude of Earthquake 2	6	
Sampler ratio, Cs, for "Other non-SPT sampler"	1	
Residual Strength Range (0 to 1)	0.33	decimal, from 0 to 1
Height of long-term surface above the surface ATD	0	ft
Total unit weight of fill	135	pcf



Cyclic Stress Ratio vs. "Clean Sand" Corrected Blow Counts, $N_{1,60cs}$



'Clean Sand' Corrected Blow Counts versus Depth - Borehole B-8

Depth ATD ft	Elev m	Blow Count	Sampler Type (M,S)	Fines -#200	Top of Layer (opt)	Bottom of Layer (opt)	Ko	$N_{1,60}$	$N_{1,60,cs}$	Factor of Safety Eq 1	Factor of Safety Eq 2	Settlement inches	PI	LL	wc %	Zone
1.5	-3.93	27	SPT_1.375_ID	5	0	2.5	0.5	28.7	28.7	2.37	2.37	0.000	-	-	8	Sand w Gravel
2.5	-3.63	52	SPT_1.375_ID	5	2.5	4.5	0.5	54.3	54.3	>10	>10	0.000	-	-	9	Sand w Gravel
5.5	-2.71	35	SPT_1.375_ID	5	4.5	7.5	0.5	44.1	44.1	>10	>10	0.002	-	-	9	Sand w Gravel
10.5	-1.19	24	SPT_1.375_ID	5	7.5	12.5	0.5	27.5	27.5	2.19	2.19	0.015	-	-	13	Sand
15.5	0.34	3	SPT_1.375_ID	15	12.5	17.5	0.5	3.8	7.0	0.51	0.51	1.849	-	-	25	Sand
20.5	1.86	33	SPT_1.375_ID	15	17.5	21.5	0.5	33.7	36.9	7.64	7.64	0.000	-	-	25	Sand

Total Settlement = 1.87

Figure D4-12 Liquefaction Settlement Calculations for B-8 (1999)

Source: Idriss and Boulanger 2004.

Liquefaction of Coral Sands:

Cyclic Calibration Chamber Tests

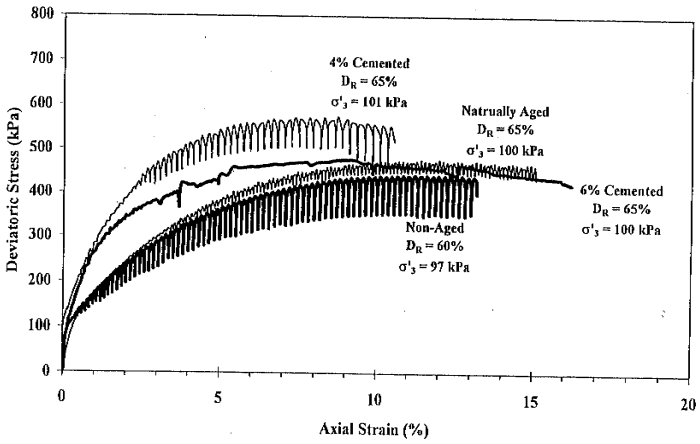


Figure 5-43. Static Strength Test Comparisons of Ewa Plains at σ_3 = 100 kPa

Figure D4-13 Liquefaction of Coral Sands

Source: Morioka and Nicholson 1999.

Coral–Silica Sand Comparison

For equal Dr, Coral is more resistant to liquefaction

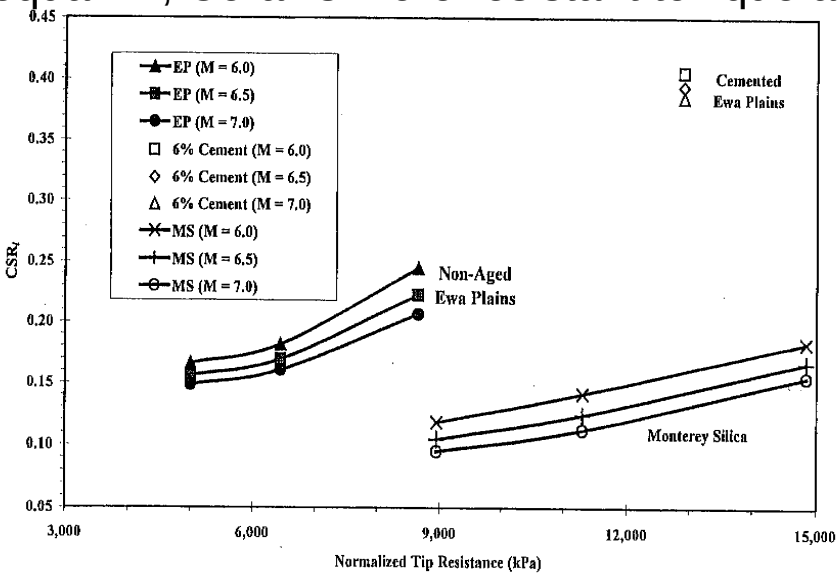
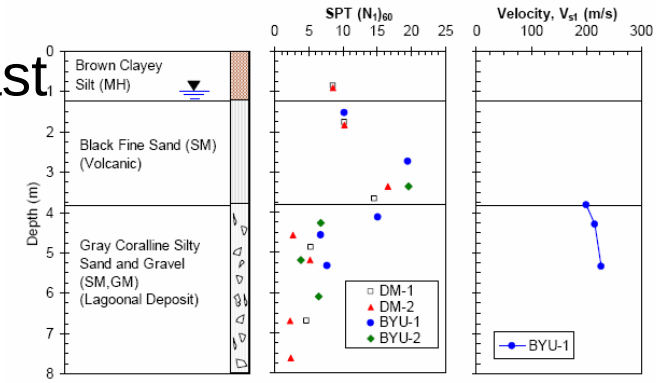


Figure 5-89. CSR versus Normalized Tip Resistance

Figure D4-14 Comparison of Coral and Silica Sands

Source: Morioka and Nicholson 1999.

Maui Blast Test



- Weakly cemented
- Liquefied at lower energy than silica

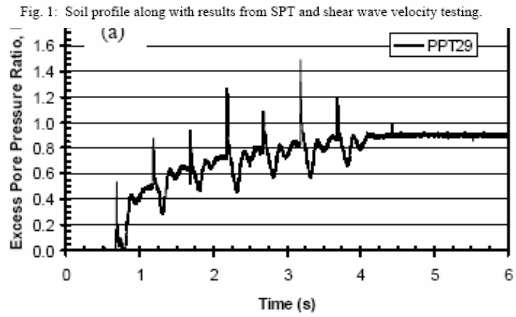


Figure D4-15 Blast Liquefaction Test at Maui Site

Source: Rollins and Ashford 2004.

Maui Blast Test cont'd.

- Vs indicated it shouldn't liquefy,
- Vs dropped 1/3 during liquef, then regained
- Apparent weak particle contact Vs correction similar to Andrus Stokoe gravels.

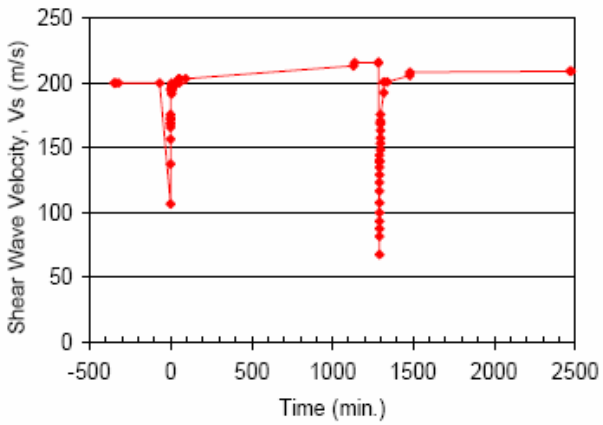


Fig. 10: Measured shear wave velocity as a function of time for both blast sequences

Figure D4-16 Shear Wave Data from Maui Blast Site

Source: Rollins and Ashford 2004.

References for Coralline Liquefaction Tables and Figures

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Anderson, B. 1981. *Engineering Geologic Classification of Coralline Deposits.* Reef Studies Institute, Contribution No 2, Ewa Beach, Hawaii.

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Ismail, Randolph, et al. JGGE, June 2002.

Morioka, B., and P. Nicholson. 1999. *Evaluation of the Static and Cyclic Properties of Calcareous Sand in a Calibration Chamber Study.* University of Hawaii Report for Hawaii Department of Transportation (DOT) and Federal Highway Administration (FHWA). Honolulu, Hawaii.

Rollins, K., and S. Ashford. 2004. *Results of Blast Liquefaction Tests in Coralline Soils.* SDEE ICEGE, San Diego, California.

Stearns, H. 1960. *The Geology of Hawaii.* University of Hawaii Press, Manoa, Hawaii.

Drawings

Drawing No.	Project Number	Location	Description
10	HC 5218	Pier 2A/2B	Drawings for landside improvements at Kawaihae Harbor, Hawaii (1990). (Includes drawings for heavy load bridge over drainage channel.)
11	HC 5219	Pier 2B	Drawings for dredging and overseas pier extension at Kawaihae Harbor by Nishimura, Katayama, Oki & Santo, Inc., Okahara & Associates, Inc. (May 1990).
14	HC 5313	Behind Pier 2A	Pave additional barge terminal area, Kawaihae Harbor, Hawaii (October 1999).
16	HC 5218	Pier 2A/2B	Pile driving logs; landside improvements at Kawaihae Harbor (June 1993). (8½- x 11-inch drawings)
17	HC 5219	Pier 2B	Ernest K. Hirata & Associates, Inc., Test Pile Driving Report, Extended Overseas Pier, Kawaihae Harbor Improvements. Summary of Test Pile Logs, Pile Driving Logs (May 1991). (8½- x 11-inch drawings)
18	HC 5219	Pier 2B	Dredging and Overseas Pier Extension at Kawaihae Harbor, Hawaii, Pile Driving Logs, Volumes 1, 2, and 3 (1991) (8½- x 11-inch drawings)

Drawing No.	Project Number	Location	Description
19	HC 5219	Pier 2B	Nishimura, Katayama, Oki & Santo, Inc., and Okahara & Associates, Inc., Structural Calculations for Dredging and Overseas Pier Extension at Kawaihae Harbor (May 1990). (8½- x 11-inch drawings)

Appendix E

Excerpts from FEMA 310

1.0 General Provisions

1.1 Scope

This Handbook provides a three-tiered process for seismic evaluation of existing buildings in any region of seismicity. Buildings are evaluated to either the Life Safety or Immediate Occupancy Performance Level.

Use of this Handbook and mitigation of deficiencies identified using this Handbook are voluntary or as required by the authority having jurisdiction. The design of mitigation measures is not addressed in this Handbook.

This Handbook does not preclude a building from being evaluated by other well-established procedures based on rational methods of analysis in accordance with principles of mechanics and approved by the authority having jurisdiction.

Commentary:

This Handbook provides a process for seismic evaluation of existing buildings. A major portion is dedicated to instructing the evaluating design professional on how to determine if a building is adequately designed and constructed to resist seismic forces. All aspects of building performance are considered and defined in terms of structural, nonstructural and foundation/geologic hazard issues.

Prior to using this Handbook, a rapid visual screening of the building may be performed to determine if an evaluation is needed using the following document:

- *Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook* (FEMA 154 and 155).

Mitigation strategies for rehabilitating buildings found to be deficient are not included in this Handbook; additional resources should be consulted for information regarding mitigation strategies.

Handbook Basis

This Handbook is based on the *NEHRP Handbook for Seismic Evaluation of Existing Buildings* (FEMA 178). This Handbook was written to:

- reflect advancements in technology,
- incorporate design professional experience,
- incorporate lessons learned during recent earthquakes,
- be nationally applicable, and
- provide evaluation techniques for varying levels of building performance.

Since the development and publication of FEMA 178, numerous significant earthquakes have occurred: the 1985 Michoacan Earthquakes that affected the Mexico City area, the 1989 Loma Prieta Earthquake in the San Francisco Bay Area, the 1994 Northridge Earthquake in the Los Angeles area, and the 1995 Hyokogen-Nanbu Earthquake in the Kobe area. While each earthquake validated the fundamental assumptions underlying the procedures presented in FEMA 178, each also offered new insights into the potential weaknesses in certain systems that should be mitigated. (It should be noted that while the publication of FEMA 178 occurred after the Mexico City and Loma Prieta Earthquakes, data and lessons learned from them were unable to be incorporated into the document prior to publication.)

Extent of Application

Model building codes typically exempt certain classes of buildings from seismic requirements pertaining to new construction. This is most often done because the building is unoccupied or it is of a style of construction that is naturally earthquake resistant. It is reasonable to expect that these classes of buildings may be exempt from the requirements of this Handbook as well.

No buildings are automatically exempt from the evaluation provisions of this Handbook; exemptions

exemptions should be defined by public policy. However, based on the exemption contained in the codes for new buildings, jurisdictions may exempt the following classes of construction:

- Detached one- and two-family dwellings located where the design short-period spectral response acceleration parameter, S_{DS}, is less than 0.4g.
- Detached one- and two-family wood frame dwellings located where the design short-period response acceleration parameter, S_{DS}, is equal to or greater than 0.4g that satisfy the light-frame construction requirements of the *1997 NEHRP Recommended Provisions for Seismic Regulations for New Buildings*; and
- Agricultural storage structures that are intended only for incidental human occupancy.

Application to Historic Buildings

Although the principles for evaluating historic structures are similar to those for other buildings, special conditions and considerations may exist of which the design professional should be aware.

Historic structures often include archaic materials, systems, and details. It may be necessary to look at handbooks and building codes from the year of construction to determine details and material properties.

Another unique aspect of historic building evaluation is the need to consider architectural elements or finishes. Testing that damages the historic character of the building generally is not acceptable.

In addition, an appropriate level of performance for historic structures needs to be chosen that is acceptable to the local jurisdiction. Some feel that historic buildings should meet the safety levels of other buildings since they are a subset of the general seismic safety needs. Others feel that historic structures, because of their value to society, should meet a higher level of performance. And in some cases a reduced level of performance has been allowed to avoid damaging historic fabric.

The following resources may be useful when evaluating historic structures:

- *Secretary of the Interior's Standards for the Treatment of Historic Properties*, and
- *National Park Service Catalog of Historical Preservation Publications.*

Alternative Methods

Alternative documents that may be used to evaluate existing buildings include:

- *Uniform Code for Building Conservation* (UCBC, 1997),
- *Los Angeles Division 91*, *Los Angeles Division 95*, and
- *Seismic Evaluation and Retrofit of Concrete Buildings.*

Some users have based the seismic evaluation of buildings on the provisions of new buildings. While this may seem appropriate, it must be done with full knowledge of the inherent assumptions. Codes for new buildings contain three basic types of requirements including strength, stiffness, and detailing. The strength and stiffness requirements are easily transferred to existing buildings; the detailing provisions are not. If the lateral-force-resisting elements of an existing building do not have the proper details of construction, the basic expectations of the other strength and stiffness provisions will not be met. Lateral-force-resisting elements that are not properly detailed should be omitted during an evaluation using a code for new buildings.

ATC-14 offered the first technique for adjusting the evaluation for the lack of proper detailing by using a three-level acceptance criteria, FEMA 178 used reduced R-factors to accomplish the same thing. FEMA 273 contains the most comprehensive procedure with its element-based approach. This Handbook follows the lead of FEMA 273 with a new style of analysis procedure tailored to the Tier 1 and Tier 2 evaluation levels.

1.2 **Basic**

Mitigation Strategies

Potential seismic deficiencies in existing buildings may be identified using this Handbook. If the evaluation is voluntary, the owner may choose to accept the risk of damage from future earthquakes rather than upgrade, or demolish the building. If the evaluation is required by a local ordinance for a hazard-reduction program, the owner may have to choose between rehabilitation, demolition, or other options.

The following documents may be useful in determining appropriate rehabilitation or mitigation strategies:

- *NEHRP Handbook of Techniques for the Seismic Rehabilitation of Existing Buildings* (FEMA 172),
- *NEHRP Benefit-Cost Model for the Seismic Rehabilitation of Buildings* (FEMA 227 and 228),
- *NEHRP Typical Costs for Seismic Rehabilitation of Existing Buildings* (FEMA 156 and 157), and
- *NEHRP Guidelines and Commentary for the Seismic Rehabilitation of Buildings* (FEMA 273 and 274).

Requirements

Prior to conducting the seismic evaluation, the evaluation requirements of Chapter 2 shall be met.

A Tier 1 evaluation shall be conducted for all buildings in accordance with the requirements of Chapter 3. Checklists, as applicable, of compliant/non-compliant statements related to structural, nonstructural and foundation conditions, shall be selected and completed in accordance with the requirements of Section 3.3 for a Tier 1 Evaluation. Potential deficiencies shall be summarized upon completion of the Tier 1 evaluation.

Structural Tier 1 checklists are not provided for unreinforced masonry bearing wall buildings with flexible diaphragms. The structural evaluation of unreinforced masonry bearing wall buildings with flexible diaphragms shall be completed using the Tier 2 Special Procedure of Section 4.2.6; a Tier 1 Evaluation for foundations and non-structural elements remains applicable for this type of building.

For those buildings identified in Section 3.4, a Full-Building Tier 2 Evaluation or a Tier 3 Evaluation shall be performed upon completion of the Tier 1 Evaluation.

For those buildings not identified in Section 3.4 as requiring a Full Building Tier 2 Evaluation or a Tier 3 Evaluation, but for which potential deficiencies were identified in Tier 1, a Deficiency-Only Tier 2 Evaluation may be performed. For a Deficiency-Only Tier 2 Evaluation, only the procedures associated with non-compliant checklist statements need be completed. Potential deficiencies shall be summarized upon completion of the Tier 2 Evaluation. Alternatively, the design professional may choose to end the investigation and report the deficiencies in accordance with Chapter 1.

A Tier 3 evaluation shall be performed in accordance with the requirements of Chapter 5 for buildings identified in Section 3.4 or when the design professional chooses to further evaluate buildings for which potential deficiencies were identified in Tier 1 or Tier 2. Potential deficiencies shall be summarized upon completion of the Tier 3 Evaluation.

After a seismic evaluation has been performed, a final report shall be prepared. As a minimum, the report shall identify: the building and its character, the tier(s) of evaluation used, and the findings.

The three-tiered process for seismic evaluation of buildings is depicted in Figure 1-1.

Commentary:

Prior to conducting the seismic evaluation based on this Handbook, the design professional should understand the evaluation process and the basic requirements specified in this section.

The evaluation process consists of the following three tiers, which are shown in Figure 1-1: Screening Phase (Tier 1), Evaluation Phase (Tier 2), and Detailed Evaluation Phase (Tier 3). As indicated in Figure 1-1, the design professional may choose to (i) report deficiencies and screening

recommend mitigation or (ii) conduct further evaluation, after any tier of the evaluation process.

The screening phase, Tier 1, consists of 3 sets of checklists that allow a rapid evaluation of the structural, nonstructural and foundation/geologic hazard elements of the building and site conditions. It shall be completed for all building evaluations conducted in accordance with this Handbook. The purpose of a Tier 1 evaluation is to screen out buildings that comply with the provisions of this Handbook or quickly identify potential deficiencies. In some cases "Quick Checks" may be required during a Tier 1 evaluation, however, the level of analysis necessary is minimal. If deficiencies are identified for a building using the checklists, the design professional may proceed to Tier 2 and conduct a more detailed evaluation of the building or conclude the evaluation and state that potential deficiencies were identified. In some cases a Tier 2 or Tier 3 evaluation may be required.

Based on the ABK research (ABK, 1984), unreinforced masonry buildings with flexible diaphragms were shown to behave in a unique manner. Special analysis procedures provided in Section 4.2.6 were developed to predict the behavior. Since this special procedure does not lend itself to the checklist format of Tier 1, no Structural Checklists are provided. The design professional must perform the Tier 2 Special Procedure as the first step of the evaluation. The Special Procedure only applies to the structural aspects of the building; Tier 1 Checklists provided for the nonstructural elements and for the foundation and geologic hazards issues still apply.

For Tier 2, a complete analysis of the building that addresses all of the deficiencies identified in Tier 1 shall be performed. Analysis in Tier 2 is limited to simplified linear analysis methods. As in Tier 1, evaluation in Tier 2 is intended to identify buildings not requiring rehabilitation. If deficiencies are identified during a Tier 2 evaluation, the design professional may choose to either conclude the evaluation and report the deficiencies or proceed to Tier 3 and conduct a detailed seismic evaluation.

Available methods and references for conducting a Tier 3 detailed evaluation are described in Chapter 5 of this Handbook. Recent research has shown that certain types of complex structures can be shown to be adequate using nonlinear analysis procedures even though other common procedures do not. While these procedures are complex and expensive to carry out, they often result in construction savings equal to many times their cost. The use of Tier 3 procedures must be limited to appropriate cases.

The final report serves to communicate the results to the owner and record the process and assumptions used to complete the evaluation. Each section should be carefully written in a manner that is understandable to its intended audience. The extent of the final report may range from a letter to a detailed document. The final report should include at least the following items:

1) Scope and Intent: a list of the tier(s) followed and level of investigation conducted;
2) Site and Building Data:
   - General building description (number of stories and dimensions),
   - Structural system description (framing, lateral load resisting system, floor and roof diaphragm construction, basement, and foundation system),
   - Nonstructural element description (nonstructural elements that could interact with the structure and affect seismic performance)
   - Building type,
   - Performance Level,
   - Region of Seismicity,
   - Soil Type,
   - Building Occupancy, and
   - Historic Significance;
3) List of Assumptions: material properties, site soil conditions;
4) Findings: list of deficiencies;
5) Recommendations: mitigation schemes or further evaluation;
6) Appendix: references, preliminary calculations.

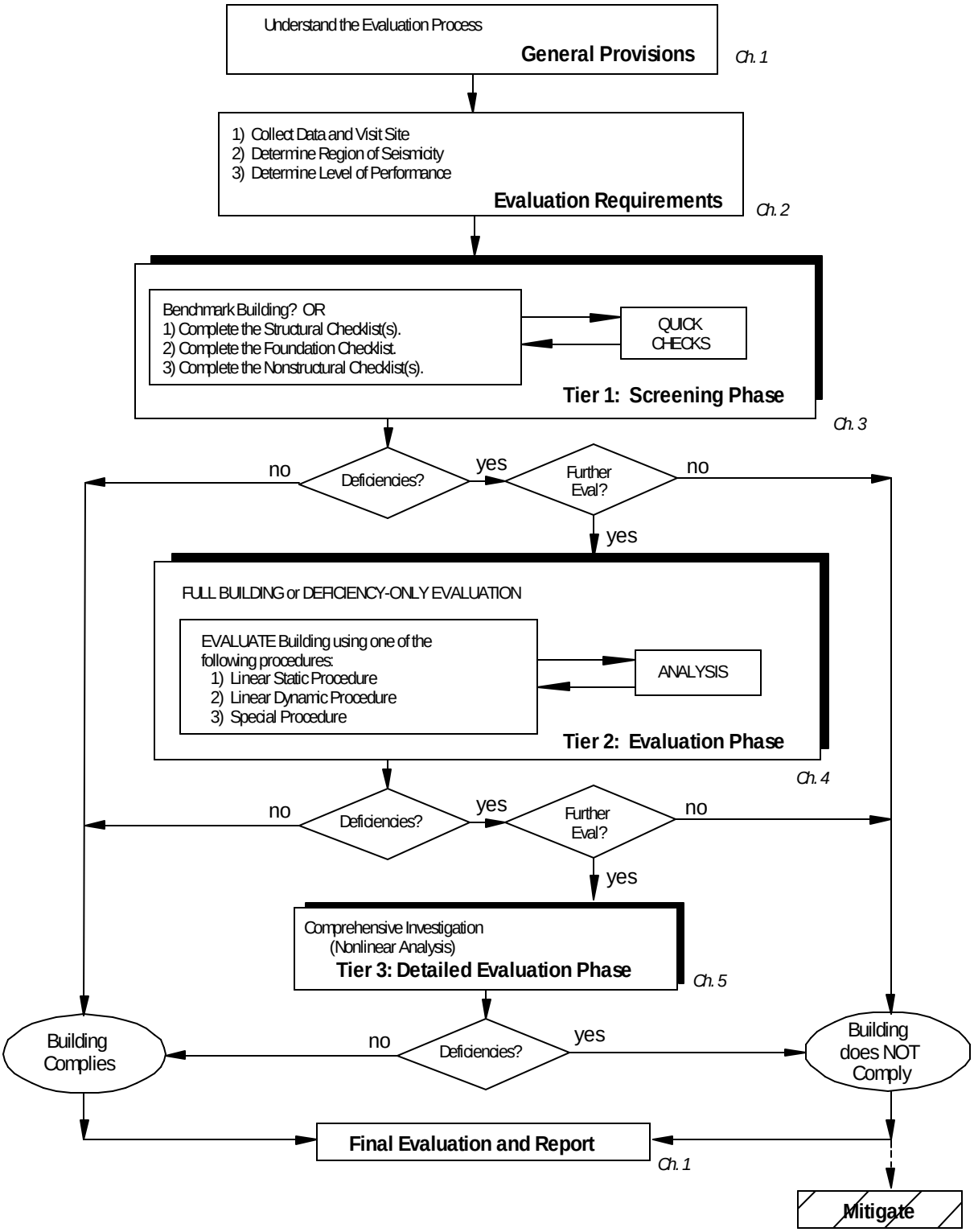

Figure 1-1. Evaluation Process

Judgment by the Design Professional

While this Handbook provides very prescriptive direction for the evaluation of existing buildings, it is not to be taken as the only direction. This Handbook provides direction for common details, deficiencies and behavior observed in past earthquakes that are found in common building types. However, every structure is unique and may contain features and details not covered by this Handbook. It is important that the design professional use judgment when applying the provisions of this Handbook. The design professional should always be looking for uncommon details and behavior about the structure not covered by this Handbook that may have the potential for damage or collapse.

1.3 Definitions

ACTION: Forces or moments that cause displacements and deformations.

ASPECT RATIO: Ratio of full height to length for shear walls; ratio of span to depth for horizontal diaphragms.

BASIC NONSTRUCTURAL CHECKLIST: Set of evaluation statements that shall be completed as part of the Tier 1 Evaluation. Each statement represents a potential nonstructural deficiency based on performance in past earthquakes.

BASIC STRUCTURAL CHECKLIST: Sets of evaluation statements that shall be completed as part of the Tier 1 Evaluation. Each statement represents a potential structural deficiency based on performance in past earthquakes.

BENCHMARK BUILDING: A building designed and constructed or evaluated to a specific performance level using an acceptable code or standard listed in Table 3-1.

BUILDING TYPE: A building classification defined in Section 2.6, that groups buildings with common lateral-force-resisting systems and performance characteristics in past earthquakes.

CAPACITY: The permissible strength or deformation for a component action.

COLLECTOR: A member that transfers lateral forces from the diaphragm of the structure to vertical elements of the lateral-force resisting system.

CROSS WALL: A wood-framed wall sheathed with lumber, structural panels, or gypsum wallboard.

DEFICIENCY-ONLY TIER 2 EVALUATION: An evaluation, beyond the Tier 1 Evaluation, that investigates only the non-compliant checklist evaluation statements.

DESIGN EARTHQUAKE: See Maximum Considered Earthquake.

DIAPHRAGM: A horizontal structural system that serves to interconnect the building and acts to transmit lateral forces to the vertical resisting elements.

DIAPHRAGM EDGE: The intersection of the horizontal diaphragm and a shear wall.

DISPLACEMENT-CONTROLLED ACTION: An action that has an associated deformation that is allowed to exceed the yield value of the element being evaluated. The extent of permissible deformation beyond yield is based on component modification factors (m-factors).

EXPECTED STRENGTH: The actual strength of a material, not the specified minimum or nominal strength. For purposes of an evaluation using this Handbook, the expected strength shall be taken equal to the nominal strength multiplied by 1.25. Alternatively, actual statistically based test data may be used.

FLEXIBLE DIAPHRAGM: A diaphragm where the maximum lateral deformation along its length is more than twice the average inter-story drift.

FORCE-CONTROLLED ACTION: An action that has an associated deformation that is not allowed to exceed the yield value of the element being evaluated. The action is not directly related to the pseudo seismic forces used in the evaluation, rather it is based on the maximum action that can be delivered to the element by the yielding structural system.

FULL-BUILDING TIER 2 EVALUATION: An evaluation beyond a Tier 1 Evaluation that involves a complete analysis of the entire lateral-force-resisting system of the building using the Tier 2 analysis procedures defined in Section 4.2. While special attention should be given to the potential deficiencies identified in the Tier 1 evaluation, all lateral force resisting elements must be evaluated. This evaluation is required when triggered by Table 3-3.

GEOLOGIC SITE HAZARDS AND FOUNDATIONS CHECKLIST: Set of evaluation statements that shall be completed as part of the Tier 1 Evaluation. Each statement represents a potential foundation or site deficiency based on the performance of buildings in past earthquakes.

IMMEDIATE OCCUPANCY PERFORMANCE LEVEL: Building performance that includes very limited damage to both structural and nonstructural components during the design earthquake. The basic vertical and lateral-force-resisting systems retain nearly all of their pre-earthquake strength and stiffness. The level of risk for life-threatening injury as a result of damage is very low. Although some minor repairs may be necessary, the building is fully habitable after a design earthquake, and the needed repairs may be completed while the building is occupied.

LATERAL FORCE RESISTING SYSTEM: The collection of frames, shear walls, bearing walls, braced frames and interconnecting horizontal diaphragms that provides earthquake resistance to a building.

LIFE SAFETY PERFORMANCE LEVEL: Building performance that includes significant damage to both structural and nonstructural components during a design earthquake, though at least some margin against either partial or total structural collapse remains. Injuries may occur, but the level of risk for life-threatening injury and entrapment is low.

LINEAR DYNAMIC PROCEDURE (LDP): A Tier 2 response spectrum based modal analysis procedure shall be used for buildings taller than 100 feet, buildings with vertical or geometric irregularities, and buildings where the distribution of the lateral forces departs from that assumed for the Linear Static Procedure.

LINEAR STATIC PROCEDURE (LSP): A Tier 2 lateral force analysis procedure where the pseudo lateral force is equal to the force required to impose the expected actual deformation of the structure in its yielded state when subjected to the design earthquake motions. It shall be used for buildings for which the Linear Dynamic or the Special Procedure is not required.

MAXIMUM CONSIDERED EARTHQUAKE: An earthquake with a 2% probability of exceedance in 50 years with deterministic-based maximum values near known fault sources.

MOMENT-RESISTING FRAME (MRF): A frame capable of resisting horizontal forces because the members (beams and columns) and joints are capable of resisting forces primarily by flexure.

PRIMARY COMPONENT: A part of the lateral-force-resisting system capable of resisting seismic forces.

PSEUDO LATERAL FORCE (V): The calculated lateral force used for the Tier 1 Quick Checks and for the Tier 2 Linear Static Procedure. The pseudo lateral force represents the force required, in a linear analysis, to impose the expected actual deformation of the structure in its yielded state when subjected to the design earthquake motions. It does not represent an actual lateral force that the building must resist in traditional code design.

QUICK CHECK: Analysis procedure used in Tier 1 Evaluations to determine if the lateral-force-resisting system has sufficient strength and/or stiffness.

REGION OF LOW SEISMICITY CHECKLIST: Set of evaluation statements that shall be completed as part of the Tier 1 Evaluation for buildings in regions of low seismicity being evaluated to the Life Safety Performance Level.

REGION OF SEISMICITY: An area with similar expected earthquake hazard. For this Handbook, all regions are categorized as low, moderate, or high, based on mapped acceleration values and site amplification factors as defined in Section 2.5.

RIGID DIAPHRAGM: A diaphragm where the maximum lateral deformation is less than half the average inter-story drift associated with the story.

SECONDARY COMPONENT: An element that is capable of resisting gravity loads, but is not able to resist seismic forces it attracts, though is not needed to achieve the designated performance level.

SITE CLASS: Groups of soil conditions that affect the site seismicity in a common manner. The soil types used are defined in Section 3.5.2.3.1; designated as A, B, C, D, E, or F.

SPECIAL PROCEDURE: Analysis procedure, used for unreinforced masonry bearing wall buildings with flexible diaphragms, that properly characterizes the diaphragm motion, strength and damping.

SPECIAL PROCEDURE TIER 2 EVALUATION: An evaluation procedure specifically written for unreinforced masonry bearing wall buildings with flexible diaphragms using the special procedure.

STIFF DIAPHRAGM: A diaphragm that is not classified as either flexible or rigid.

STORY SHEAR FORCE: Portion of the pseudo lateral force carried by each story of the building.

SUPPLEMENTAL NONSTRUCTURAL CHECKLIST: Set of nonstructural evaluation statements that shall be completed as part of the Tier 1 Evaluation for buildings in regions of moderate or high seismicity being evaluated to the Immediate Occupancy Performance Level.

SUPPLEMENTAL STRUCTURAL CHECKLIST: Set of evaluation statements that shall be completed as part of the Tier 1 Evaluation for buildings in regions of moderate seismicity being evaluated to the Immediate Occupancy Performance Level, and for buildings in regions of high seismicity.

TIER 1 EVALUATION: Completion of checklists of evaluation statements that identifies potential deficiencies in a building based on performance in past earthquakes.

TIER 2 EVALUATION: The specific evaluation of potential deficiencies to determine if they represent actual deficiencies that may require mitigation. Depending on the building type, this evaluation may be a Full-Building Tier 2 Evaluation, Deficiency-Only Tier 2 Evaluation, or a Special Procedure Tier 2 Evaluation.

TIER 3 EVALUATION: A comprehensive building evaluation implicitly or explicitly recognizing nonlinear response.

1.4 Notation

$\mathbf{a_p}$	Component amplification factor,
$\mathbf{A_{br}}$	Average cross-sectional area of the diagonal brace,
$\mathbf{A_c}$	Summation of the cross-sectional area of all columns in the story under consideration,
$\mathbf{A_n}$	Area of net mortared/grouted section (in^2),
$\mathbf{A_w}$	Summation of the horizontal cross-sectional area of all shear walls in the direction of loading,
$\mathbf{A_x}$	Amplification factor to account for accidental torsion,
C	Modification factor to relate expected maximum inelastic displacements calculated for linear elastic response,
C	Compliant,
$\mathbf{C_p}$	Horizontal force factor,
$\mathbf{C_t}$	Modification factor, based on earthquake records, used to adjust the building period to account for the characteristics of the building system,
$\mathbf{C_{vx}}$	Vertical distribution factor, based on story weights and heights, that defines a triangular loading pattern,

D	In-plane width dimension of masonry (in.) or depth of diaphragm (ft.),
DCR	Demand-capacity ratio,
$\mathbf{D_p}$	Relative displacement,
DR, $\mathbf{D_r}$	Drift ratio,
E	Modulus of Elasticity;
$\mathbf{F_a}$	Site Coefficient defined in Table 3-6,
$\mathbf{f_{br}}$	Average axial stress in diagonal bracing elements,
$\mathbf{F_i}$	Lateral force applied at floor level i,
$\mathbf{F_{px}}$	Total diaphragm force at level x,
$\mathbf{F_v}$	Site Coefficient defined in Table 3-5,
$\mathbf{F_{wx}}$	Force applied to a wall at level x (lb.),
$\mathbf{F_x}$	Total story force at level x,
$\mathbf{F_y}$	Yield Stress,
h	Story height,
$\mathbf{h_i}$,$\mathbf{h_x}$	Height (ft.) from the base to floor level i or x,
$\mathbf{h_n}$	Height (in feet) above the base to the roof level,
H	Least clear height of opening on either side of pier (in.),
I	Moment of Inertia,
IO	Immediate Occupancy Performance Level,
j	number of story level under consideration,
J	Force-delivery reduction factor,
k	Exponent related to the building period,
$\mathbf{k_b}$	Stiffness of a representative beam (I/L);
$\mathbf{k_c}$	Stiffness of a representative column (I/h);
L	Length;
$\mathbf{L_{br}}$	Average length of the diagonal brace,
LS	Life-Safety Performance Level,
m	Component modification factor,
$\mathbf{M_g}$	Moment in girder (k-ft),
n, N	number of stories above ground,
N/A	Not Applicable,
$\mathbf{N_{br}}$	Number of diagonal braces in tension and compression if the braces are designed for compression; Number of diagonal braces in tension if the braces are designed for tension only,
$\mathbf{n_c}$	Total number of columns,
$\mathbf{n_f}$	Total number of frames,
NC	Non-Compliant,
NL	No Limit,
$\mathbf{P_{CE}}$	Expected gravity compressive force applied to a wall or pier component stress,
$\mathbf{P_D}$	Superimposed dead load at the top of the pier under consideration (lb.),
$\mathbf{P_W}$	Weight of wall (lb.),
$\mathbf{Q_{CE}}$	Expected strength,
$\mathbf{Q_D}$	Actions due to effective dead load,
$\mathbf{Q_E}$	Actions due to earthquake loads,
$\mathbf{Q_G}$	Actions due to effective gravity load,
$\mathbf{Q_L}$	Actions due to effective live load,
$\mathbf{Q_S}$	Actions due to effective snow load,
$\mathbf{Q_{UD}}$	Deformation-controlled design actions,
$\mathbf{Q_{UF}}$	Force-controlled design actions,
$\mathbf{R_p}$	Component response modification factor,
s	Average span length of braced spans (ft.),
$\mathbf{S_a}$	Response spectral acceleration,
$\mathbf{S_{DS}}$	Design short-period spectral response acceleration parameter,
$\mathbf{S_{D1}}$	Design spectral response acceleration parameter at a one-second period,
$\mathbf{S_S}$	Short-period spectral response acceleration parameter,
$\mathbf{S_1}$	Spectral response acceleration parameter at a one-second period,
t	Thickness of wall (in.)

T	Fundamental period of vibration of the building,
T1	Tier 1 Evaluation,
T2	Tier 2 Evaluation,
T3	Tier 3 Evaluation,
$\mathbf{v_{avg}}$	Average shear stress,
$\mathbf{v_{me}}$	Expected masonry shear strength (psi),
$\mathbf{v_u}$	Unit shear strength for a diaphragm (lb./ft.),
$\mathbf{v_{te}}$	Average bed-joint shear strength (psi), not to exceed 100 psi,
V	Pseudo lateral force,
$\mathbf{V_a}$	Shear strength of an unreinforced masonry pier (lb.),
$\mathbf{V_c}$	Column shear force,
$\mathbf{V_{ca}}$	Total shear capacity of cross walls in the direction of analysis immediately above the diaphragm level being investigated (lb.),
$\mathbf{V_{cb}}$	Total shear capacity of cross walls in the direction of analysis immediately below the diaphragm level being investigated (lb.),
$\mathbf{V_d}$	Diaphragm shear (lb.),
$\mathbf{V_j}$	Story shear force,
$\mathbf{V_p}$	Shear force on an unreinforced masonry wall pier (lb.),
$\mathbf{V_r}$	Pier rocking shear capacity of an unreinforced masonry wall or wall pier (lb.),
$\mathbf{V_{wx}}$	Total shear force resisted by a shear wall at the level under consideration (lb.),
$\mathbf{w_i}$, $\mathbf{w_x}$	Portion of the total building weight assigned to floor level i or x,
W	Total seismic weight,
$\mathbf{W_d}$	Total dead load tributary to a diaphragm (lb.),
$\mathbf{W_j}$	Total seismic weight of all stories above level j,
$\mathbf{W_p}$	Component operating weight,
$\mathbf{W_w}$	Total dead load of an unreinforced masonry wall above the level under consideration or above an open front of a building,
$\mathbf{W_{wx}}$	Dead load of an unreinforced masonry wall assigned to level x halfway above and below the level under consideration (lb.),
x	Height in structure of highest point of attachment of component,
X,Y	Height of lower support attachment at level x or y as measured from grade,
$\mathbf{D_d}$	Diaphragm displacement,
$\mathbf{D_w}$	In-plane wall displacement,
$\mathbf{d_{avg}}$	the maximum dispalcement at any point of diaphragm at level x,
$\mathbf{d_{max}}$	the algebraic average of displacements at the extreme points of the diaphragm at level x,
$\mathbf{d_{xA}}$,$\mathbf{d_{yA}}$	Deflection at building level x or y of building A,
$\mathbf{d_{xB}}$	Deflection at building level x of building B,
r''	Volumetric ratio of horizontal confinement reinforcement in a joint.

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2.0 Evaluation Requirements

2.1 General

Prior to conducting a seismic evaluation, the evaluation requirements of this Chapter shall be met.

2.2 Level of Investigation Required

Prior to conducting a Tier 1 Evaluation, all available documents shall be collected and reviewed. A complete examination of all available documents pertaining to the design and construction of the building shall be conducted. If construction documents are available, the examination shall include verification that the building was constructed in accordance with the documents. All alterations and deviations shall be noted. The information collected shall be sufficient to define the level of performance desired in accordance with Section 2.4, the region of seismicity in accordance with Section 2.5, and the building type in accordance with Section 2.6. In addition, the level of investigation shall be sufficient to complete the Tier 1 Checklists. Destructive examination shall be conducted as required to complete the Checklists for buildings being evaluated to the Immediate Occupancy Performance Level; judgment shall be used regarding the need for destructive evaluation for buildings being evaluated to the Life-Safety Performance Level. Non-destructive examination of connections and conditions, shall be performed for all Tier 1 Evaluations. Default values may be used for material properties for a Tier 1 Evaluation.

In addition to the information required for a Tier 1 Evaluation, sufficient information shall be collected for a Tier 2 Evaluation to complete the required Tier 2 Procedures. Destructive examination shall be conducted as required to complete the Procedures for buildings being evaluated to the Immediate Occupancy Performance Level and for buildings in regions of high seismicity as defined in Table 2-1. Non-destructive examination of connections and conditions shall be performed for all Tier 2 Evaluations. While material testing is not required for a Tier 2 Evaluation, default values for material properties shall not be used. Material property data shall be obtained from building codes from the year of construction of the building being evaluated, from as-built plans, or from physical tests.

Exception: Unreinforced masonry bearing wall buildings with flexible diaphragms using the Tier 2 Special Procedure of Section 4.2.6 shall have destructive tests conducted to determine the average bed-joint shear strength, v_{te}, and the strength of the anchors.

Detailed information about the building is required for a Tier 3 Evaluation. If no documents are available, an as-built set of drawings shall be created indicating the existing lateral-force-resisting system. Non-destructive and destructive examination and testing shall be conducted for a Tier 3 Evaluation to establish:

- the expected strength of all materials that participate in the lateral-force-resisting system of the building; deterioration shall be taken into account;
- the composition and configuration of all primary components and conditions in the lateral-force-resisting system.

Commentary:

Building evaluation involves many substantial difficulties. One is the matter of uncovering the structure since plans and calculations often are not available. In many buildings the structure is concealed by architectural finishes, and the design professional will have to get into attics, crawl spaces, and plenums to investigate. Some intrusive testing may be necessary to determine material quality and allowable stresses. If reinforcing plans are available, some exposure of critical reinforcement may be necessary to verify conformance with the plans. The extent of investigation required depends on the level of

evaluation because the conservatism inherent in both the Tier 1 and Tier 2 analysis covers the lack of detailed information in most cases. The evaluating deisgn professional is encouraged to balance the investigation with the sophistication of the evaluation technique.

The design professional in responsible charge should be consulted if possible. In addition, the evaluating design professional may find it helpful to do some research on historical building systems, consult old handbooks and building codes, and perhaps consult with older engineers who have knowledge of early structural work in the community or region.

The evaluation should be based on facts, as opposed to assumptions, to the greatest extent possible.

One of the more important factors in any evaluation is the material properties and strengths. For a Tier 1 Evaluation, the following default values may be assumed: f'_c of 3000 psi for concrete, F_y of 40 ksi for reinforcing steel, F_y of 36 ksi for structural steel, f'_m of 1500 psi for masonry. For a Tier 2 Evaluation, the material strengths can be determined by existing documentation or material testing. For a Tier 3 Evaluation, material testing is required to verify the existing documentation or establish the strengths if existing documentation is not available.

Prior to evaluating a building using this handbook, the design professional should:

- Look for an existing geotechnical report on site soil conditions;
- Establish site and soil parameters;
- Assemble building design data including contract drawings, specifications, and calculations;
- Look for other data such as assessments of the building performance during past earthquakes; and
- Select and review the appropriate sets of evaluation statements included in Chapter 3.

Testing of Masonry

Different types of masonry require different tests to determine the shear capacity. The design professional should use the following as a general guide for selecting the correct test method:

- Multi-wythe masonry laid with headers should use the in-place shear push test;
- For modern masonry, the design professional should consider using a core tested as prescribed in ASTM C 496-90 to determine the tensile-splitting stress. The tensile-splitting stress is the same as the horizontal shear stress. The mortar joints should be at 45° to the load. This should be modified for axial stress by Mohr's procedures;
- Another method is to use a square prism extracted from the wall that is tested as prescribed in ASTM E 519-74 to determine the tensile-splitting stress. The method of relating the test to tensile-splitting in ASTM E 519-74 requires verification. The effect of axial loading on the tensile-splitting stress must be added for the expected horizontal shear stress;
- Use a prism extracted from the wall to determine f'_m. Then use f'_m in empirical formulas to determine the expected shear strength;
- Trace the source of the masonry units for the unit compressive strength. Then use the unit compressive strength with the mortar class on the available construction. documents to determine f'_m.

2.3 Site Visit

A site visit shall be conducted by the evaluating design professional to verify existing data or collect additional data, determine the general condition of the building, and verify or assess the site conditions.

Commentary:

Relevant building data that should be determined through a site visit includes:

- General building description - number of stories, year(s) of construction, and dimensions.
- Structural system description - framing, lateral-force-resisting system(s), floor and roof diaphragm construction, basement, and foundation system.
- Nonstructural element description - nonstructural elements that could interact with the structure and affect seismic performance.
- Building type(s) - Categorize the building as one or more of the Common Building Types, if possible.
- Performance Level - Note the performance level required in the evaluation.
- Region of Seismicity - Identify the seismicity of the site to be used for the evaluation.
- Soil type - Note the soil type.
- Building Occupancy - The occupancy of the building should be noted.
- Historic Significance - Identify any historic elements in the building. Any impacts or areas of the building affected by the evaluation should be noted.

A first assessment of the evaluation statements may indicate a need for more information about the building. The design professional may need to re-visit the site to do the following:

1. Verify existing data;
2. Develop other required data;
3. Verify the vertical and lateral-force-resisting systems;
4. Check the condition of the building;
5. Look for special conditions and anomalies;
6. Address the evaluation statements again while in the field; and
7. Perform material tests, as necessary.

Commentary:

FEMA 178 addressed only the Life Safety Performance Level for buildings. This Handbook addresses both the Life Safety and Immediate Occupancy Performance Levels.

The seismic analysis and design of buildings has traditionally focused on one performance level; reducing the risk to life loss in the largest expected earthquake. Building codes for new buildings and the wide variety of evaluation guidelines developed in the last 30 years have based their provisions on the historic performance of buildings and the deficiencies that caused life safety concerns to develop. Beginning with the damage to hospitals in the 1971 San Fernando earthquake, there has been a growing desire to design and construct certain "essential facilities" that that are needed immediately after an earthquake. In addition, there has been a growing recognition that new buildings should have some measure of damaged resistance built in while existing buildings need to be held only to a minimum safety standard. During this time, a new style of design guidelines began appearing that promised a variety of performance levels. At one extreme, the ABK Methodology was developed to better understand when URM buildings needed to be strengthened to achieve a minimum level of safety. At the other extreme, the California Building Code for Hospital Design and Construction set the

Construction set the criteria for buildings that need to remain operational.

The extensive and expensive, non-life threatening damage that occurred in the Northridge Earthquake brought these various performance levels to the point of formalization. Performance Based Engineering was rigorously described by the Structural Engineers Association of California in their Vision 2000 document. At the same time, the Earthquake Engineering Research Center published a research and development plan for the development of Performance Based Engineering Guidelines and Standards. The first formal application in published guidelines occurred in FEMA 273, where the range of possible performance levels and hazard levels were combined to define specific performance objectives to be used to rehabilitate buildings.

This Handbook defines and uses performance levels in a manner consistent with FEMA 273. The Life Safety and Immediate Occupancy Performance Levels are the same as defined in FEMA 273. The hazard level used is the third in a series of four levels defined in FEMA 273. The level chosen is consistent with the hazard traditionally used for seismic analysis and similar to that used in FEMA 178. For other performance levels and/or hazard levels, the design professional should perform a Tier 3 analysis.

The process for defining the appropriate level of performance is the responsibility of the design professional or the authority having jurisdiction. Considerations in choosing an appropriate level of performance should include achieving basic safety, a cost-benefit analysis, the building occupancy type, economic constraints, etc.

In general, buildings classified as essential facilities should be evaluated to the Immediate Occupancy Performance Level. The *1997 NEHRP Recommended Provisions for Seismic Regulations for New Buildings* categorizes the following buildings as essential facilities "...required for post-earthquake recovery":

- Fire or rescue and police stations,
- Hospitals or other medical facilities having surgery or emergency treatment facilities,
- Emergency preparedness centers including the equipment therein,
- Power generating stations or other utilities required as emergency back-up facilities for other facilities listed here,
- Emergency vehicle garages,
- Communication centers, and
- Buildings containing sufficient quantities of toxic or explosive substances deemed to be dangerous to the public if released.

2.4 Level of Performance

A desired level of performance shall be defined prior to conducting a seismic evaluation using this Handbook. The level of performance shall be determined by the design professional and by the authority having jurisdiction. The following two performance levels for both structural and nonstructural components are defined in Section 1.3 of this handbook: Life Safety (LS) and Immediate Occupancy (IO). For both performance levels, the seismic demand is based on Maximum Considered Earthquake (MCE) spectral response acceleration values. Buildings complying with the criteria of this Handbook shall be deemed to meet the specified performance level.

2.5 Region of Seismicity

The region of seismicity of the building shall be defined as low, moderate, or high in accordance with Table 2-1. Regions of seismicity are defined in terms of mapped response acceleration values and site amplification factors.

Commentary:

The successful performance of buildings in areas of high seismicity depends on a combination of strength, ductility (manifested in the details of construction) and the presence of a fully interconnected, balanced, and complete lateral-force-resisting system. As these fundamentals are applied in regions of lower seismicity, the need for strength and ductility reduces substantially and, in fact, strength can substitute for a lack of ductility. Very brittle lateral-force-resisting systems can be excellent performers as long as they are never pushed beyond their elastic strength.

ATC-14, the first generation version of FEMA 178 recognized this fact and defined separate provisions for regions of low and high seismicity. Based in part on work sponsored by the Nation Center for Earthquake Engineering Research (NCEER, 1987) FEMA 178 eliminated the separate provisions and elected to permit the lateral force calculations to determine when there was sufficient strength to make up for a lack of detailing and ductility.

The collective experience of the engineers using FEMA 178 is that the requirements too often require calculations for deficiencies that are never a problem because of the low lateral forces. This Handbook took this experience and has develop three separate Tier 1 procedures for the three fundamental regions of seismicity. The regions are defined in terms of the expected spectral response for the site under consideration. Thus the criteria for an area bepends both on the expected MCE accelerations and on the site adjustment factors. This will cause area in the transition zone between regions to have sub-areas that are in one region immediately adjacient to a sub-area in another region. This is an intentional result and the experience at the Marina District in the Loma Prieta Earthquake is ample evidence of its credibility.

Table 2-1. Regions of Seismicity Definitions

Region of Seismicity[1]	S_{DS}	S_{D1}
Low	< 0.167g	< 0.067g
Moderate	< 0.500g > 0.167g	< 0.200g > 0.067g

Commentary:

Fundamental to the Tier 1 analysis of buildings is the grouping of buildings into sets that have similar behavioral characteristics. These groups of "building types" were first defined in ATC-14 and have been used in most of the FEMA guideline documents since. During the development of FEMA 273, it was determined that a number of additional types of buildings were needed to cover all common styles of construction. These were fully developed and presented in that document. The added building types included a Northridge-style apartment building, and a number of variations on diaphragm type for the basic building systems. The new types are included as subtypes to the original fifteen, so there remains fifteen model building types.

The common building types are defined in Table 2-2. Because most structures are unique in some fashion, judgment should be used when selecting the building type, with the focus on the lateral-force-resisting system and elements.

Separate checklists for each of the Common Building Types are included in this Handbook as well as General Structural Checklists for buildings that may not be classified as one of the Common Building Types. Procedures for using the General Checklists are provided in Section 3.3.

where: $S_{DS} = \frac{2}{3} F_a S_s$
= design short-period spectral response

Table 2-2. Common Building Types

Building Type 1 : Wood Light Frames	
W1	These buildings are single or multiple family dwellings of one or more stories in height. Building loads are light and the framing spans are short. Floor and roof framing consists of closely spaced wood joists or rafters on wood studs. The first floor framing is supported directly on the foundation, or is raised up on cripple studs and post and beam supports. The foundation consists of spread footings constructed of concrete, concrete masonry block, or brick masonry in older construction. Chimneys, when present, consist of solid brick masonry, masonry veneer, or wood frame with internal metal flues. Lateral forces are resisted by wood frame diaphragms and shear walls. Floor and roof diaphragms consist of straight or diagonal wood sheathing, tongue and groove planks, or plywood. Shear walls consist of straight or diagonal wood sheathing, plank siding, plywood, stucco, gypsum board, particle board, or fiberboard. Interior partitions are sheathed with plaster or gypsum board.
W1A	These buildings are multi-story, multi-unit residences similar in construction to W1 buildings, but with open front garages at the first story. The first story consists of wood floor framing on wood stud walls and steel pipe columns, or a concrete slab on concrete or concrete masonry block walls.
Building Type 2: Wood Frames, Commercial and Industrial	
W2	These buildings are commercial or industrial buildings with a floor area of 5,000 square feet or more. Building loads are heavier than light frame construction, and framing spans are long. There are few, if any, interior walls. The floor and roof framing consists of wood or steel trusses, glulam or steel beams, and wood posts or steel columns. Lateral forces are resisted by wood diaphragms and exterior stud walls sheathed with plywood, stucco, plaster, straight or diagonal wood sheathing, or braced with rod bracing. Large openings for storefronts and garages, when present, are framed by post-and-beam framing. Lateral force resistance around openings is provided by steel rigid frames or diagonal bracing.
Building Type 3 : Steel Moment Frame s	
S1	These buildings consist of a frame assembly of steel beams and steel columns. Floor and roof framing consists of cast-in-place concrete slabs or metal deck with concrete fill supported on steel beams, open web joists or steel trusses. Lateral forces are resisted by steel moment frames that develop their stiffness through rigid or semi-rigid beam-column connections. When all connections are moment resisting connections, the entire frame participates in lateral force resistance. When only selected connections are moment resisting connections, resistance is provided along discrete frame lines. Columns are oriented so that each principal direction of the building has columns resisting forces in strong axis bending. Diaphragms consist of concrete or metal deck with concrete fill and are stiff relative to the frames. When the exterior of the structure is concealed, walls consist of metal panel curtain walls, glazing, brick masonry, or precast concrete panels. When the interior of the structure is finished, frames are concealed by ceilings, partition walls and architectural column furring. Foundations consist of concrete spread footings or deep pile foundations.
S1A	These buildings are similar to S1 buildings, except that diaphragms consist of wood framing or untopped metal deck, and are flexible relative to the frames.

Table 2-2. Common Building Types (cont'd)

Building Type 4 : Steel Braced Frame s	
S2	These buildings consist of a frame assembly of steel beams and steel columns. Floor and roof framing consists of cast-in-place concrete slabs or metal deck with concrete fill supported on steel beams, open web joists or steel trusses. Lateral forces are resisted by tension and compression forces in diagonal steel members. When diagonal brace connections are concentric to beam column joints, all member stresses are primarily axial. When diagonal brace connections are eccentric to the joints, members are subjected to bending and axial stresses. Diaphragms consist of concrete or metal deck with concrete fill and are stiff relative to the frames. When the exterior of the structure is concealed, walls consist of metal panel curtain walls, glazing, brick masonry, or precast concrete panels. When the interior of the structure is finished, frames are concealed by ceilings, partition walls and architectural furring. Foundations consist of concrete spread footings or deep pile foundations.
S2A	These buildings are similar to S2 buildings, except that diaphragms consist of wood framing or untopped metal deck, and are flexible relative to the frames.
Building Type 5: Steel Light Frame s	
S3	These buildings are pre-engineered and prefabricated with transverse rigid steel frames. They are one-story in height. The roof and walls consist of lightweight metal, fiberglass or cementitious panels. The frames are designed for maximum efficiency and the beams and columns consist of tapered, built-up sections with thin plates. The frames are built in segments and assembled in the field with bolted or welded joints. Lateral forces in the transverse direction are resisted by the rigid frames. Lateral forces in the longitudinal direction are resisted by wall panel shear elements or rod bracing. Diaphragm forces are resisted by untopped metal deck, roof panel shear elements, or a system of tension-only rod bracing.
Building Type 6: Steel Frames with Concrete Shear Walls	
S4	These buildings consist of a frame assembly of steel beams and steel columns. The floors and roof consist of cast-in-place concrete slabs or metal deck with or without concrete fill. Framing consists of steel beams, open web joists or steel trusses. Lateral forces are resisted by cast-in-place concrete shear walls. These walls are bearing walls when the steel frame does not provide a complete vertical support system. In older construction the steel frame is designed for vertical loads only. In modern dual systems, the steel moment frames are designed to work together with the concrete shear walls in proportion to their relative rigidity. In the case of a dual system, the walls shall be evaluated under this building type and the frames shall be evaluated under S1 or S1A, Steel Moment Frames. Diaphragms consist of concrete or metal deck with or without concrete fill. The steel frame may provide a secondary lateral-force-resisting system depending on the stiffness of the frame and the moment capacity of the beam-column connections.

acceleration parameter;

Table 2-2. Common Building Types (cont'd)

Building Type 7 : Steel Frames with Infill Masonry Shear Walls	
S5	This is an older type of building construction that consists of a frame assembly of steel beams and steel columns. The floors and roof consist of cast-in-place concrete slabs or metal deck with concrete fill. Framing consists of steel beams, open web joists or steel trusses. Walls consist of infill panels constructed of solid clay brick, concrete block, or hollow clay tile masonry. Infill walls may completely encase the frame members, and present a smooth masonry exterior with no indication of the frame. The seismic performance of this type of construction depends on the interaction between the frame and infill panels. The combined behavior is more like a shear wall structure than a frame structure Solidly infilled masonry panels form diagonal compression struts between the intersections of the frame members. If the walls are offset from the frame and do not fully engage the frame members, the diagonal compression struts will not develop. The strength of the infill panel is limited by the shear capacity of the masonry bed joint or the compression capacity of the strut. The post-cracking strength is determined by an analysis of a moment frame that is partially restrained by the cracked infill. The diaphragms consist of concrete floors and are stiff relative to the walls.
S5A	These buildings are similar to S5 buildings, except that diaphragms consist of wood sheathing or untopped metal deck, or have large aspect ratios and are flexible relative to the walls.
Building Type 8: Concrete Moment Frame s	
C1	These buildings consist of a frame assembly of cast-in-place concrete beams and columns. Floor and roof framing consists of cast-in-place concrete slabs, concrete beams, one-way joists, two-way waffle joists, or flat slabs. Lateral forces are resisted by concrete moment frames that develop their stiffness through monolithic beam-column connections. In older construction, or in regions of low seismicity, the moment frames may consist of the column strips of two-way flat slab systems. Modern frames in regions of high seismicity have joint reinforcing, closely spaced ties, and special detailing to provide ductile performance. This detailing is not present in older construction. Foundations consist of concrete spread footings or deep pile foundations.
Building Type 9 : Concrete Shear Wall Buildings	
C2	These buildings have floor and roof framing that consists of cast-in-place concrete slabs, concrete beams, one-way joists, two-way waffle joists, or flat slabs. Floors are supported on concrete columns or bearing walls. Lateral forces are resisted by cast-in-place concrete shear walls. In older construction, shear walls are lightly reinforced, but often extend throughout the building. In more recent construction, shear walls occur in isolated locations and are more heavily reinforced with boundary elements and closely spaced ties to provide ductile performance. The diaphragms consist of concrete slabs and are stiff relative to the walls. Foundations consist of concrete spread footings or deep pile foundations.
C2A	These buildings are similar to C2 buildings, except that diaphragms consist of wood sheathing, or have large aspect ratios, and are flexible relative to the walls.

Table 2-2. Common Building Types (cont'd)

Building Type 10: Concrete Frames with Infill Masonry Shear Walls	
C3	This is an older type of building construction that consists of a frame assembly of cast-in-place concrete beams and columns. The floors and roof consist of cast-in-place concrete slabs. Walls consist of infill panels constructed of solid clay brick, concrete block, or hollow clay tile masonry. The seismic performance of this type of construction depends on the interaction between the frame and infill panels. The combined behavior is more like a shear wall structure than a frame structure Solidly infilled masonry panels form diagonal compression struts between the intersections of the frame members. If the walls are offset from the frame and do not fully engage the frame members, the diagonal compression struts will not develop. The strength of the infill panel is limited by the shear capacity of the masonry bed joint or the compression capacity of the strut. The post-cracking strength is determined by an analysis of a moment frame that is partially restrained by the cracked infill. The shear strength of the concrete columns, after cracking of the infill, may limit the semiductile behavior of the system. The diaphragms consist of concrete floors and are stiff relative to the walls.
C3A	These buildings are similar to C3 buildings, except that diaphragms consists of wood sheathing, or have large aspect ratios, and are flexible relative to the walls.
Building Type 11 : Precast/Tilt-up Concrete Shear Wall Buildings	
PC1	These buildings are one or more stories in height and have precast concrete perimeter wall panels that are cast on site and tilted into place. Floor and roof framing consists of wood joists, glulam beams, steel beams or open web joists. Framing is supported on interior steel columns and perimeter concrete bearing walls. The floors and roof consist of wood sheathing or untopped metal deck. Lateral forces are resisted by the precast concrete perimeter wall panels. Wall panels may be solid, or have large window and door openings which cause the panels to behave more as frames than as shear walls. In older construction, wood framing is attached to the walls with wood ledgers. Foundations consist of concrete spread footings or deep pile foundations.
PC1A	These buildings are similar to PC1 buildings, except that diaphragms consist of precast elements, cast-in-place concrete, or metal deck with concrete fill, and are stiff relative to the walls.
Building Type 12 : Precast Concrete Frame s	
PC2	These buildings consist of a frame assembly of precast concrete girders and columns with the presence of shear walls. Floor and roof framing consists of precast concrete planks, tees or double-tees supported on precast concrete girders and columns. Lateral forces are resisted by precast or cast-in-place concrete shear walls. Diaphragms consist of precast elements interconnected with welded inserts, cast-in-place closure strips, or reinforced concrete topping slabs.
PC2A	These buildings are similar to PC2 buildings, except that concrete shear walls are not present. Lateral forces are resisted by precast concrete moment frames that develop their stiffness through beam-column joints rigidly connected by welded inserts or cast-in-place concrete closures. Diaphragms consist of precast elements interconnected with welded inserts, cast-in-place closure strips, or reinforced concrete topping slabs. This type of construction is not permitted in regions of high seismicity for new construction.

Table 2-2. Common Building Types (cont'd)

Building Type 13: Reinforced Masonry Bearing Wall Buildings with Flexible Diaphragms	
RM1	These buildings have bearing walls that consist of reinforced brick or concrete block masonry. Wood floor and roof framing consists of wood joists, glulam beams and wood posts or small steel columns. Steel floor and roof framing consists of steel beams or open web joists, steel girders and steel columns. Lateral forces are resisted by the reinforced brick or concrete block masonry shear walls. Diaphragms consist of straight or diagonal wood sheathing, plywood, or untopped metal deck, and are flexible relative to the walls. Foundations consist of brick or concrete spread footings.
Building Type 14: Reinforced Masonry Bearing Wall Buildings with Stiff Diaphragms	
RM2	These buildings are similar to RM1 buildings, except the diaphragms consist of metal deck with concrete fill, precast concrete planks, tees, or double-tees, with or without a cast-in-place concrete topping slab, and are stiff relative to the walls. The floor and roof framing is supported on interior steel or concrete frames or interior reinforced masonry walls.
Building Type 15 : Unreinforced Masonry Bearing Wall Buildings	
URM	These buildings have perimeter bearing walls that consist of unreinforced clay brick masonry. Interior bearing walls, when present, also consist of unreinforced clay brick masonry. In older construction floor and roof framing consists of straight or diagonal lumber sheathing supported by wood joists, on posts and timbers. In more recent construction floors consist of structural panel or plywood sheathing rather than lumber sheathing. The diaphragms are flexible relative to the walls. When they exist, ties between the walls and diaphragms consist of bent steel plates or government anchors embedded in the mortar joints and attached to framing. Foundations consist of brick or concrete spread footings.
URMA	These buildings are similar to URM buildings, except that the diaphragms are stiff relative to the unreinforced masonry walls and interior framing. In older construction or large, multistory buildings, diaphragms consist of cast-in-place concrete. In regions of low seismicity, more recent construction consists of metal deck and concrete fill supported on steel framing.

$S_{D1} = \frac{2}{3} F_v S_1$

= design spectral response acceleration parameter at a one second period;

F_v, F_a = site coefficients defined in Tables 3-5 and 3-6, respectively;

S_s = short-period spectral response acceleration parameter (Sec. 3.5.2.3.1);

S_1 = spectral response acceleration parameter at a one second period (Sec. 3.5.2.3.1).

2.6 Building Type

The building being evaluated shall be classified as one or more of the building types listed in Table 2-2 based on the lateral force-resisting system(s) and the diaphragm type. Two separate building types shall be used for buildings with different lateral-force-resisting systems in each of the two orthogonal directions.

3.0 Screening Phase (Tier 1)

3.1 General

A Tier 1 Evaluation shall be conducted for all buildings after the evaluation requirements of Chapter 2 have been completed. Tier 1 of the evaluation process is shown schematically in Figure 3-1.

Initially, the design professional shall determine whether the building meets the benchmark building criteria of Section 3.2. If the building meets the benchmark building criteria, it shall be deemed to meet the structural requirements of this Handbook for the specified level of performance; a Tier 1 Evaluation for foundations and nonstructural elements remains applicable.

If the building is not a benchmark building, the design professional shall select and complete the appropriate checklists in accordance with Section 3.3.

Structural checklists are not used for unreinforced masonry bearing wall buildings with flexible diaphragms. The structural evaluation of this type of building shall be completed using the Tier 2 Special Procedure of Section 4.2.6; a Tier 1 Evaluation for foundations and nonstructural elements remains applicable for this type of building.

A list of deficiencies identified by evaluation statements for which the building was found to be non-compliant shall be compiled upon completion of the Tier 1 Checklists.

Further evaluation requirements shall be determined in accordance with Section 3.4 once the checklists have been completed.

Commentary:

The purpose of the screening phase of the evaluation process is to identify quickly buildings that comply with the provisions of this handbook. A Tier 1 Evaluation also familiarizes the design professional professional with the building, its potential deficiencies and its potential behavior.

A Tier 1 Evaluation is required for all buildings so that potential deficiencies may be quickly identified. Further evaluation using a Tier 2 or Tier 3 Evaluation will then focus, as a minimum, on the potential deficiencies identified in Tier 1.

3.2 Benchmark Buildings

A structural seismic evaluation using this Handbook need not be performed for buildings designed and constructed or evaluated in accordance with the benchmark documents listed in Table 3-1; an evaluation for foundations and nonstructural elements remains applicable. Table 3-1 identifies documents whose seismic design, construction or evaluation provisions are acceptable for certain building types so that further evaluation is not required. If the seismicity of a region has changed since the benchmark dates listed in Table 3-1, a building must have been designed and constructed or evaluated in accordance with the current seismicity of the region to be compliant with this section. The design professional shall document in the final report the evidence used to determine that the building is designed and constructed or evaluated in accordance with the documents listed in Table 3-1 and current seismicity of the region.

The applicable level of performance is indicated in Table 3-1 for each document as a superscript.

Commentary:

While benchmark buildings need not proceed with further evaluation, it should be noted that they are not simply exempt from the criteria of this Handbook. The design professional must clearly demonstrate the building is compliant with the

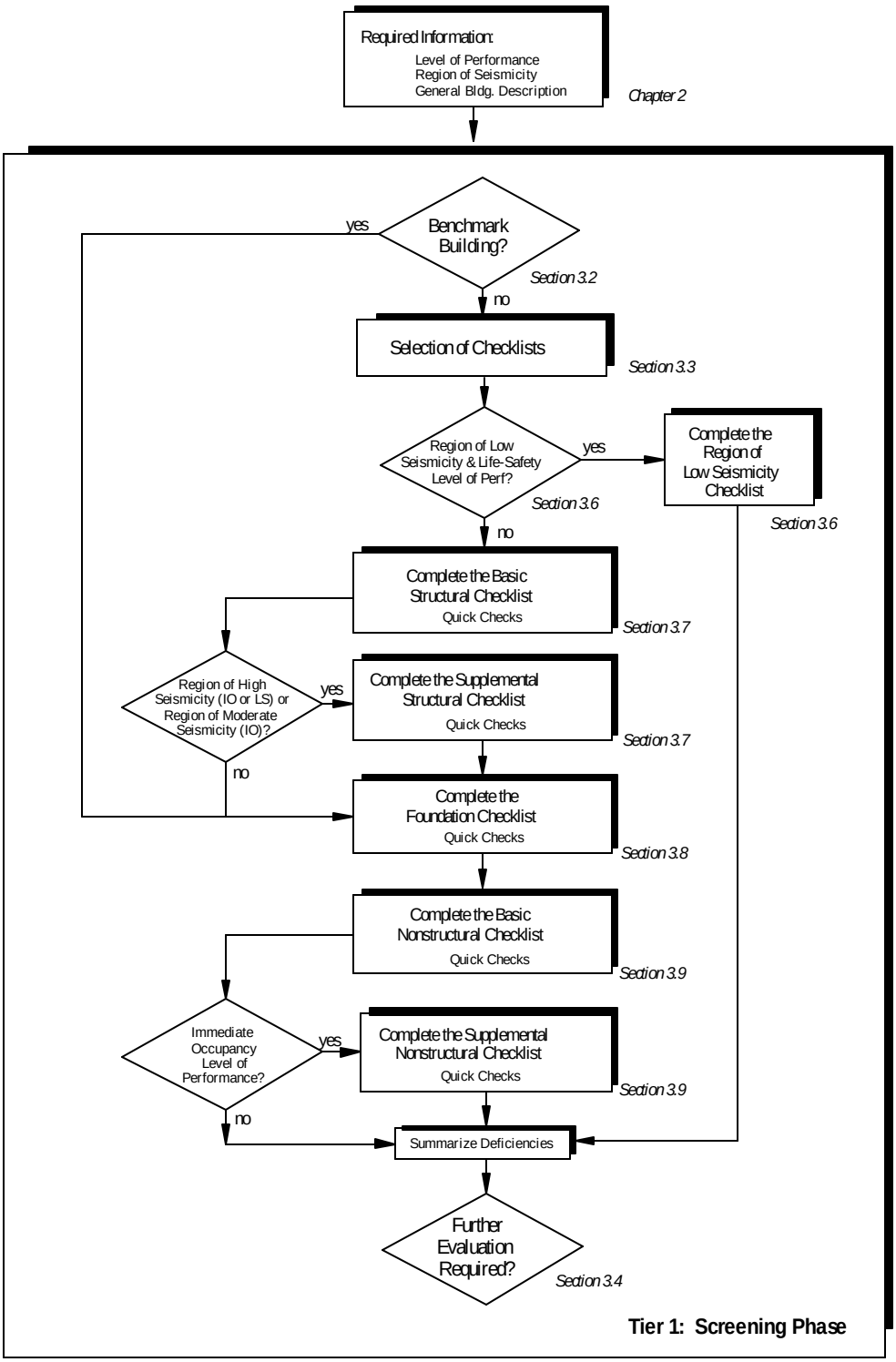


Figure 3-1. Tier 1 Evaluation Process

Table 3-1. Benchmark Buildings

	Model Building Seismic Design Provisions				FEMA 178[ls]	CBC[io]
Building Type [1]	BOCA[ls]	SBCC[ls]	UBC[ls]	NEHRP[ls]		
Wood Frame, Wood Shear Panels (Type W1 & W2)[2]	1992	1993	1976	1985	*	1973
Wood Frame, Wood Shear Panels (Type W1A)	1992	1993	1976	1985	*	1973
Steel Moment Resisting Frame (Type S1 & S1A)	**	**	1994[4]	**	*	1995
Steel Braced Frame (Type S2 & S2A)	1992	1993	1988	1991	1992	1973
Light Metal Frame (Type S3)	*	*	*	*	1992	1973
Steel Frame w/ Concrete Shear Walls (Type S4)	1992	1993	1976	1985	1992	1973
Reinforced Concrete Moment Resisting Frame (Type C1)[3]	1992	1993	1976	1985	*	1973
Reinforced Concrete Shear Walls (Type C2 & C2A)	1992	1993	1976	1985	*	1973
Steel Frame with URM Infill (Type S5, S5A)	*	*	*	*	*	*
Concrete Frame with URM Infill (Type C3 & C3A)	*	*	*	*	*	*
Tilt-up Concrete (Type PC1 & PC1A)	*	*	1997	*	*	*
Precast Concrete Frame (Type PC2 & PC2A)	*	*	*	*	1992	1973
Reinforced Masonry (Type RM1)	*	*	1997	*	*	*
Reinforced Masonry (Type RM2)	1992	1993	1976	1985	*	*
Unreinforced Masonry (Type URM)[5]	*	*	1991[6]	*	1992	*
Unreinforced Masonry (Type URMA)	*	*	*	*	*	*

[1]Building Type refers to one of the Common Building Types defined in Table 2-2.
[2]Buildings on hillside sites shall not be considered Benchmark Buildings.
[3]Flat Slab Buildings shall not be considered Benchmark Buildings.
[4]Steel Moment-Resisting Frames shall comply with the 1994 UBC Emergency Provisions.
[5]URM buildings evaluated using the ABK Methodology (ABK, 1984) may be considered benchmark buildings.
[6]Refers to the UCBC Section of the UBC.

[ls]Only buildings designed and constructed or evaluated in accordance with these documents and being evaluated to the Life-Safety Performance Level may be considered Benchmark Buildings.
[io]Buildings designed and constructed or evaluated in accordance with these documents and being evaluated to either the Life-Safety or Immediate Occupancy Performance Level may be considered Benchmark Buildings.

*No benchmark year; buildings shall be evaluated using this handbook.
**Local provisions shall be compared with the UBC.

BOCA - Building Officials and Code Administrators, *National Building Code*.
SBCC - Southern Building Code Congress, *Standard Building Code*
UBC - International Conference of Building Officials, *Uniform Building Code*
NEHRP - Federal Emergency Management Agency, *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings*
CBC - California Building Standards Commission, *California Building Code*

benchmark document. Knowledge that a code was in effect at the time of construction is not sufficient. A statement on the drawings simply stating that it was designed to the benchmark document will not suffice. Sometimes, details in the existing building will not correspond to the construction documents. Sometimes, the building is not properly detailed to meet the benchmark document. This may occur due to renovations or poor construction management. Only through a site visit, an examination of existing documentation, and other requirements of Chapter 2 will the design professional be able to determine whether the structure being evaluated complies with this section.

3.3 Selection and Use of Checklists

Required checklists, as a function of region of seismicity and level of performance, are listed in Table 3-2. Each of the required checklists designated in Table 3-2 shall be completed for a Tier 1 Evaluation. Each of the evaluation statements on the checklists shall be marked "compliant" (C), "noncompliant" (NC), or "not applicable" (N/A). Compliant statements identify issues that are acceptable according to the criteria of this Handbook, while non-compliant statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated.

Quick Checks for Tier 1 shall be performed in accordance with Section 3.5 when necessary to complete an evaluation statement.

The Region of Low Seismicity Checklist, located in Section 3.6, shall be completed for buildings in regions of low seismicity being evaluated to the Life Safety Performance Level. For buildings in regions of low seismicity being evaluated to the Immediate Occupancy Performance Level and buildings in regions of moderate or high seismicity, the appropriate Structural, Geologic Site Hazards, and Nonstructural Checklists shall be completed in accordance with Table 3-2.

The appropriate Structural Checklists shall be selected based on the Common Building Types defined in Table 2-2. The General Structural Checklists shall be used for buildings that cannot be classified as one of the Common Building Types defined in Table 2-2.

A building with a different lateral-force-resisting system in each principal direction shall use two sets of structural checklists, one for each direction. A building with more than one type of lateral-force-resisting system along a single axis of the building shall be classified as a mixed system. The General Structural Checklists shall be used for this type of building.

Two separate Structural Checklists are provided for each building type: a Basic Structural Checklist and a Supplemental Structural Checklist. As shown in Table 3-2, the Basic Structural Checklist shall be completed for buildings in regions of low seismicity being evaluated to the Immediate Occupancy Performance Level and buildings in regions of moderate and high seismicity. The Supplemental Structural Checklist shall be completed in addition to the Basic Structural Checklist for buildings in regions of moderate seismicity being evaluated to the Immediate Occupancy Performance Level and buildings in regions of high seismicity.

The Geologic Site Hazards and Foundations Checklist shall be completed for all buildings except those in regions of low seismicity being evaluated to the Life Safety Performance Level.

Two separate Nonstructural Checklists also are provided: a Basic and Supplemental Nonstructural Checklist. As shown in Table 3-2, the Basic Nonstructural Checklist shall be completed for all buildings except those in regions of low seismicity being evaluated to the Life Safety Performance Level. The Supplemental Nonstructural Checklists shall be completed in addition to the Basic Nonstructural Checklist for buildings in regions of moderate or high seismicity being evaluated to the Immediate Occupancy Performance Level.

Commentary:

The evaluation statements provided in the checklists form the core of the Tier 1 Evaluation Methodology. These evaluation statements are based on observed earthquake structural damage during actual

during actual earthquakes. The checklists do not necessarily identify the response of the structure to ground motion; rather, the design professional obtains a general sense of the structure's deficiencies and potential behavior during an earthquake. By quickly identifying the potential deficiencies in the structure, the design professional has an better idea of what to examine and analyze in a Tier 2 or Tier 3 Evaluation.

The General Structural Checklists are a complete listing of all evaluation statements used in Tier 1 Evaluations. They should be used for buildings with structural systems that do not match the common building types. While the general purpose of the Tier 1 Checklists is to identify potential weak-links in structures that have been observed in past significant earthquakes, the General Checklists, by virtue of their design, do not accomplish this. They only represent a listing of all possible deficiencies. The design professional must consider first the applicablility of the potential deficiency to the building system being considered. Generally, only the deficiencies that participate in the yielding elements of the building need be considered.

While the section numbers in parentheses following each evaluation statement correspond to Tier 2 Evaluation procedures, they also correspond to commentary in Chapter 4 regarding the statement's purpose. If the design professional requires additional information on particular evaluation statements, please refer to the commentary associated with the Tier 2 procedure for that evaluation statement..

3.4 Further Evaluation Requirements

Upon completion of the Tier 1 Evaluation, further evaluation shall be conducted in accordance with Table 3-3.

A Full-Building Tier 2 Evaluation shall be completed for buildings with more than the number of stories listed in Table 3-3. 'NL' designates No Limit on the number of stories.

A Full-Building Tier 2 Evaluation also is required for buildings designated in Table 3-3 by 'T2'. A Tier 3 Evaluation shall be required for buildings designated by 'T3' in Table 3-3.

For buildings not requiring a Full-Building Tier 2 Evaluation or a Tier 3 Evaluation, a Deficiency-Only Tier 2 Evaluation may be conducted if potential deficiencies are identified by the Tier 1 Evaluation. Alternatively, the design professional may choose to end the investigation and report the deficiencies in accordance with Chapter 1.

Commentary:

In most cases, the Tier 1 identification of potential deficiencies leads to further evaluation of only these deficiencies. As defined in Chapter 4, the required analysis may be localized to the specific deficiencies or it may involve a global analysis to evaluate the specific deficiency. Each checklist evaluation statement concludes with a reference to the applicable section in Chapter 4; the Tier 2 procedures as well as commentary on the statements' purpose.

The 'NL' designation for most buildings being evaluated to the Life Safety Performance Level is consistent with FEMA 178, which had no restriction on the use of the checklists. The 'SP' designation for unreinforced masonry bearing wall buildings with flexible diaphragms also is consistent with FEMA 178.

The 'T2,' 'T3,' and number of story designations in the Immediate Occupancy Performance Level category indicates that the building cannot be deemed to meet the requirements of this Handbook without a full evaluation of the building. Based on past performance of these types of buildings in earthquakes, the behavior of the structure must be examined and understood. However, the Tier 1 Checklists will provide insight and information about the structure prior to a Tier 2 or Tier 3 Evaluation.

Table 3-2. Checklists Required for a Tier 1 Evaluation

Region of Seismicity	Level of Performance[2]	Required Checklists[1]					
		Region of Low Seismicity (Sec. 3.6)	Basic Structural (Sec. 3.7)	Supplemental Structural (Sec. 3.7)	Geologic Site Hazard and Foundation (Sec. 3.8)	Basic Nonstructural (Sec. 3.9.1)	Supplemental Nonstructural (Sec. 3.9.2)
Low	LS	√					
	IO		√		√	√	
Moderate	LS		√		√	√	
	IO		√	√	√	√	√
High	LS		√	√	√	√	
	IO		√	√	√	√	√

[1]A checkmark (√) designates that the checklist that must be completed for a Tier 1 evaluation as a function of the region of seismicity and level of performance.
[2]LS = Life-Safety; IO = Immediate Occupancy; defined in Section 2.3.

Table 3-3. Further Evaluation Requirements[1]

	Number of Stories beyond which a Full-Building Tier 2 Evaluation is Required					
	Low		Moderate		High	
Model Building Type	**LS**	**IO**	**LS**	**IO**	**LS**	**IO**
Wood Frames						
Light (W1)	NL	2	NL	2	NL	2
Multistory, Multi-Unit Residential (W1A)	NL	3	NL	2	NL	2
Commercial and Industrial (W2)	NL	2	NL	2	NL	2
Steel Moment Frames						
Rigid Diaphragm (S1)	NL	3	NL	T2	NL	T2
Flexible Diaphragm (S1A)	NL	3	NL	T2	NL	T2
Steel Braced Frames						
Rigid Diaphragm (S2)	NL	3	NL	2	NL	2
Flexible Diaphragm (S2A)	NL	3	NL	2	NL	2
Steel Light Frames (S3)	NL	1	NL	1	NL	1
Steel Frame with Concrete Shear Walls (S4)	NL	4	NL	4	NL	3
Steel Frame with Infill Masonry Shear Walls						
Rigid Diaphragm (S5)	NL	2	NL	T2	NL	T2
Flexible Diaphragm (S5A)	NL	2	NL	T2	NL	T2
Concrete Moment Frames (C1)	NL	2	NL	T2	NL	T2
Concrete Shear Walls						
Rigid Diaphragm (C2)	NL	4	NL	4	NL	3
Flexible Diaphragm (C2A)	NL	4	NL	4	NL	3
Concrete Frame with Infill Masonry Shear Walls						
Rigid Diaphragm (C3)	NL	2	NL	T2	NL	T2
Flexible Diaphragm (C3A)	NL	2	NL	T2	NL	T2
Precast/Tilt-up Concrete Shear Walls						
Flexible Diaphragm (PC1)	NL	1	NL	T2	NL	T2
Rigid Diaphragm (PC1A)	NL	1	NL	T2	NL	T2
Precast Concrete Frames						
With Shear Walls (PC2)	NL	4	NL	4	NL	3
Without Shear Walls (PC2A)	NL	T2	NL	T2	NL	T2
Reinforced Masonry Bearing Walls						
Flexible Diaphragm (RM1)	NL	3	NL	T2	NL	T2
Rigid Diaphragm (RM2)	NL	3	NL	3	NL	2
Unreinforced Masonry Bearing Walls						
Flexible Diaphragm (URM)	NL	T3	SP	T3	SP	T3
Rigid Diaphragm (URMA)	NL	1	NL	T3	NL	T3
Mixed Systems	NL	2	NL	T2	NL	T2

[1]A Full-Building Tier 2 or Tier 3 Evaluation shall be completed for buildings with more than the number of stories listed herein .
SP - Special Procedure (A Tier 2 Evaluation is required using the Special Procedure defined in Section 4.2.6; the Geologic Site Hazards and Foundations Checklist and the Nonstructural Checklist shall be completed prior to performing the Special Procedure Analysis) .
NL - No Limit (No limit on the number of stories).
T2 - Tier 2 (A Full-Building Tier 2 Evaluation is required; proceed to Chapter 4).
T3 - Tier 3 (A Tier 3 Evaluation is required; proceed to Chapter 5).

3.5 Tier 1 Analysis

3.5.1 Overview

Analyses performed as part of Tier 1 of the Evaluation Process are limited to Quick Checks. Quick Checks shall be used to calculate the stiffness and strength of certain building components to determine whether the building complies with certain evaluation criterion. Quick Checks shall be performed in accordance with Section 3.5.3 when triggered by evaluation statements from the Checklists of Section 3.7. Seismic shear forces for use in the Quick Checks shall be computed in accordance with Section 3.5.2.

3.5.2 Seismic Shear Forces

3.5.2.1 Pseudo Lateral Force

The pseudo lateral force, in a given horizontal direction of a building, shall be calculated in accordance with Equations (3-1) and (3-2).

$$V = CS_a W \qquad (3\text{-}1)$$

where:

V = Pseudo lateral force;

C = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response; C shall be taken from Table 3-4;

S_a = Response spectral acceleration at the fundamental period of the building in the direction under consideration. The value of S_a shall be calculated in accordance with the procedures in Section 3.5.2.3.

W = Total dead load and anticipated live load as follows:

- In storage and warehouse occupancies, a minimum of 25% of the floor live load; The actual partition weight or minimum weight of 10 psf of floor area, whichever is greater;
- The applicable snow load;
- The total weight of permanent equipment and furnishings.

Alternatively, for buildings with shallow foundations and without basements being evaluated for the Life Safety Performance Level, Equation (3-2) may be used to compute the pseudo lateral force:

$$V = 0.75W \qquad (3\text{-}2)$$

If Equation (3-2) is used, an *m*-factor of 1.0 shall be used to compute the component forces and stresses for the Quick Checks of Section 3.5.3 and acceptance criteria of Section 4.2.4.

Table 3-4. Modification Factor, C

Building Type[1]	**Number of Stories**			
	1	2	3	≥ 4
Wood (W1, W1A, W2) Moment Frame (S1, S3, C1, PC2A)	1.3	1.1	1.0	1.0
Shear Wall (S4, S5, C2, C3, PC1A, PC2, RM2, URMA) Braced Frame (S2)	1.4	1.2	1.1	1.0
Unreinforced Masonry (URM) Flexible Diaphragms (S1A, S2A, S5A, C2A, C3A, PC1, RM1)	1.0	1.0	1.0	1.0

[1]Defined in Table 2-2.

Commentary:

The seismic evaluation procedure of this Handbook, as well as the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings* and the *Uniform Building Code*, is based on a widely-accepted philosophy that permits nonlinear response of a building when subjected to a ground motion that is representative of the design earthquake. The *NEHRP Recommended Provisions for Seismic Regulations for New Buildings*, the *Uniform Building Code* and FEMA 178 account for nonlinear seismic response in a linear static analysis procedure by including a response modification factor, R, in calculating a reduced equivalent base shear to produce a rough approximation of the internal forces during a design earthquake. In other words, the base shear is equivalent to what the bulding is expected to resist strength-wise, but the building displacement using this base shear are significantly less than the displacements the building will actually experience during a design earthquake. Thus, this approach

increases the base shear by another factor (C_d, .7R, etc.) when checking drift and ductility requirements. In summary, this procedure is based on equivalent lateral forces and pseudo displacements.

The linear static analysis procedure in this Handbook, as well as in FEMA 273, takes a different approach to account for the nonlinear seismic response. Pseudo static lateral forces are applied to the structure to obtain "actual" displacements during a design earthquake. The pseudo lateral force of Equation (3-1) represents the force required, in a linear static analysis, to impose the expected actual deformation of the structure in its yielded state when subjected to the design earthquake motions.

It does not represent an actual lateral force that the building must resist in traditional design codes or FEMA 178. In summary, this procedure is based on equivalent displacements and pseudo lateral forces. For additional commentary regarding this linear static analysis approach, please refer to the commentary for Section 4.2.2.1 and FEMA 273 and 274.

Instead of applying a ductility related response reduction factor, *R*, to the applied loads, this Handbook uses ductility related *m*-factors in the acceptability checks of each component. Thus, instead of using a single *R*-value for the entire structure, different *m*-factors are used depending on the ductility of the component being evaluated. The *m*-factors specified for each Tier of analysis shall not be used for other Tiers of analysis (i.e., Tier 3 values of *m* may not be used when a Tier 1 or Tier 2 analysis is performed).

For short and stiff buildings with low ductility located in regions of high seismicity, the required building strength in accordance with Equation (3-1) may exceed the force required to cause sliding at the foundation level. The strength of the structure, however, does not need to exceed the strength of the ground. Thus, when Equation (3-2) is applied to these buildings, the required strength of structural components need not exceed 0.75W.

3.5.2.2 Story Shear Forces

For multi-story buildings, the pseudo lateral force computed in accordance with Section 3.5.2.1 shall be distributed vertically in accordance with Equation (3-3).

$$V_j = \left(\frac{n+j}{n+1}\right)\left(\frac{W_j}{W}\right)V \qquad (3\text{-}3)$$

where:

V_j = Story shear at story level j,
n = Total number of stories above ground level,
j = Number of story level under consideration,
W_j = Total seismic weight of all stories above level j,
W = Total seismic weight per Section 3.5.2.1,
V = Pseudo lateral force from Equation (3-1) or (3-2).

For buildings with flexible diaphragms (Types S1A, S2A, S5A, C2A, C3A, PC1, RM1, URM), story shear shall be calculated separately for each line of lateral resistance. This value shall be calculated using Equation (3-3) with W_j defined as the seismic weight of all stories above level j tributary to the line of resistance under consideration.

3.5.2.3 Spectral Acceleration

Spectral acceleration for use in computing the pseudo lateral force shall be computed in accordance with this section. Spectral acceleration shall be based on mapped spectral accelerations, defined in Section 3.5.2.3.1, for the site of the building being evaluated. Alternatively, a site specific response spectrum may be developed according to Section 3.5.2.3.2.

3.5.2.3.1 Mapped Spectral Acceleration

The mapped spectral acceleration, S_a, shall be computed in accordance with Equation (3-4).

$$S_a = \frac{S_{D1}}{T}, \text{ but} \qquad (3\text{-}4)$$

S_a shall not exceed S_{DS} ;

where:

$$S_{D1} = \frac{2}{3}F_v S_1 \qquad (3\text{-}5)$$

$$S_{DS} = \frac{2}{3}F_a S_s \qquad (3\text{-}6)$$

T = Fundamental period of vibration of the building calculated in accordance with Section 3.5.2.4.

S_s and S_1 are short period response acceleration and spectral response acceleration at a one second period parameters, respectively, for the Maximum Considered Earthquake (MCE). S_s and S_1 shall be obtained from the *Seismic Map Package*. F_v and F_a are site coefficients and shall be determined from Tables 3-5 and 3-6, respectively, based on the site class and the values of the response acceleration parameters S_s and S_1. The site class of the building shall be defined as one of the following:

- **Class A**: Hard rock with measured shear wave velocity, $\bar{v}_s$ > 5,000 ft/sec;
- **Class B**: Rock with 2,500 ft/sec $\leq \bar{v}_s \leq$ 5,000 ft/sec.
- **Class C**: Very dense soil and soft rock with 1,200 ft/sec < $\bar{v}_s \leq$ 2,500 ft/sec or with either standard blow count $\bar{N}$ > 50 or undrained shear strength $\bar{s}_u$ > 2,000 psf.
- **Class D**: Stiff soil with 600 ft/sec < $\bar{v}_s \leq$ 1,200 ft/sec or with 15 < $\bar{N} \leq$ 50 or 1,000 psf $\leq \bar{s}_u$ < 2000 psf.
- **Class E**: Any profile with more than 10 feet of soft clay defined as soil with plasticity index PI >20, or water content w > 40 percent, and $\bar{s}_u$ < 500 psf or a soil profile with $\bar{v}_s$ < 600 ft/sec.
- **Class F**: Soils requiring a site-specific geotechnical investigation and dynamic site response analyses:
  - Soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, quick and highly-sensitive clays, collapsible weakly-cemented soils;
  - Peats and/or highly organic clays (H>10 feet of peat and/or highly organic clay; where H = thickness of soil);
  - Very high plasticity clays (H > 25 feet with PI > 75 percent);
  - Very thick soft/medium stiff clays (H > 120 feet).

For a soil profile classified as Class F, a Class E soil profile may be assumed for a Tier 1 Evaluation. If sufficient data is not available to classify a soil profile, a Class E profile shall be assumed. For one- and two-story buildings with a roof height equal to or less than 25 feet, a Class D soil profile may be assumed if site conditions are not known.

Table 3-5. Values of F_v as a Function of Site Class and Mapped Spectral Acceleration at a One Second Period, S_1

Site Class	Mapped Spectral Acceleration at One Second Period[1]				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.4	2.8	2.4	2.2
F	*	*	*	*	*

[1]Note: Use straight-line interpolation for intermediate values of S_1.

* See Class F soil profile.

Table 3-6. Values of F_a as a Function of Site Class and Mapped Short-Period Spectral Acceleration, S_s

Site Class	Mapped Spectral Acceleration at Short Periods[1]				
	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	*	*	*	*	*

[1]NOTE: Use straight-line interpolation for intermediate values of S_s.

*See Class F soil profile.

Commentary:

The short period response acceleration and spectral response acceleration at a one second period parameters, S_s and S_1, are provided in the *Seismic Map Package*. The values of S_s and S_1 represent an earthquake with a 2% probability of exceedance in 50 years with deterministic-based maximum values near known fault sources. For information on obtaining a copy of the *Seismic Map Package*, please contact the FEMA Distribution Facility at 1-800-480-2520.

3.5.2.3.2 Site-Specific Spectral Acceleration

Development of site-specific response spectra shall be based on the geologic, seismological, and soil characteristics associated with the specific site of the building being evaluated. Site-specific response spectra shall be based on input ground motions with a 2% probability of exceedance in 50 years (2500 year return interval) and developed for an equivalent viscous damping ratio of 5%. The site specific response spectra need not exceed the mean deterministic spectra for faults with known slip rates. When the 5% damped site specific spectrum has spectral amplitudes in the period range of greatest significance to the structural response that are less than 70% of the mapped spectral amplitudes, an independent third-party review of the spectrum shall be made by an individual with expertise in the evaluation of ground motion.

3.5.2.4 Period

The fundamental period of a building, in the direction under consideration, shall be calculated in accordance with Equation (3-7).

$$T = C_t h_n^{3/4} \qquad (3\text{-}7)$$

where:

- T = Fundamental period (in seconds) in the direction under consideration;
- C_t = 0.060 for wood buildings (Building Types W1, W1A, and W2);
  - = 0.035 for moment-resisting frame systems of steel (Building Types S1 and S1A);
  - = 0.030 for moment-resisting frames of reinforced concrete (Building Type C1);
  - = 0.030 for eccentrically-braced steel frames (Building Types S2 and S2A);
  - = 0.020 for all other framing systems;
- h_n = height (in feet) above the base to the roof level.

Alternatively, for steel or reinforced-concrete moment frames of 12 stories or less the fundamental period of the building may be calculated as follows:

$$T=0.10N \qquad (3\text{-}8)$$

where:

N = number of stories above the base.

3.5.3 Quick Checks for Strength and Stiffness

Quick Checks shall be used to compute the stiffness and strength of building components. Quick Checks are triggered by evaluation statements in the Checklists of Section 3.7 and are required to determine the compliance of certain building components. The seismic shear forces used in the Quick Checks shall be calculated in accordance with Section 3.5.2.

Commentary:

The quick check equations used here are essentially the same as those used in FEMA 178, modified for use with the pseudo lateral forces and the appropriate material *m*-factors.

3.5.3.1 Story Drift for Moment Frames

Equation (3-9) shall be used to calculate the drift ratios of regular, multistory, multibay moment frames with columns continuous above and below the story under consideration. The drift ratio is based on the deflection due to flexural displacement of a representative column, including the effect of end rotation due to bending of the representative girder.

$$DR = \left(\frac{k_b+k_c}{k_b \cdot k_c}\right)\left(\frac{h}{12E}\right)V_c \qquad (3\text{-}9)$$

where:

- DR = Drift Ratio = Interstory displacement divided by story height,
- k_b = I/L for the representative beam,
- k_c = I/h for the representative column,
- h = Story height (in.),
- I = Moment of inertia (in^4),

L = Center to center length of columns (in.),
E = Modulus of elasticity (ksi),
V_c = Shear in the column (kips).

The column shear forces shall be taken as a portion of the story shear forces, computed in accordance with Section 3.5.2.2. For reinforced concrete frames, an equivalent cracked section moment of inertia equal to one half of gross value shall be used.

Equation (3-9) may also be used for the first floor of the frame if columns are fixed against rotation at the bottom. However, if columns are pinned at the bottom, an equivalent story height equal to twice the actual story height shall be used in calculating the value of k_c.

For other configurations of frames, the quick check need not be performed as a Full-Building Tier 2 Evaluation including calculation of the drift ratio shall be completed based on principles of structural mechanics.

3.5.3.2 Shear Stress in Concrete Frame Columns

The average shear stress, v_{avg}, in the columns of concrete frames shall be computed in accordance with Equation (3-10).

$$v_{avg} = \frac{1}{m}\left(\frac{n_c}{n_c - n_f}\right)\left(\frac{V_j}{A_c}\right) \qquad (3\text{-}10)$$

where:

n_c = Total number of columns;
n_f = Total number of frames in the direction of loading;
A_c = Summation of the cross sectional area of all columns in the story under consideration; and
V_j = Story shear computed in accordance with Section 3.5.2.2.
m = component modification factor; m shall be taken equal to 2.0 for buildings being evaluated to the Life Safety Performance Level and 1.3 for buildings being evaluated to the Immediate Occupancy Performance Level.

Commentary:

Equation (3-10) assumes that all of the columns in the frame have similar stiffness.

3.5.3.3 Shear Stress in Shear Walls

The average shear stress in shear walls, v_{avg}, shall be calculated in accordance with Equation (3-11).

$$v_{avg} = \frac{1}{m}\left(\frac{V_j}{A_w}\right) \qquad (3\text{-}11)$$

where:

V_j = Story shear at level j computed in accordance with Section 3.5.2.2;
A_w = Summation of the horizontal cross sectional area of all shear walls in the direction of loading. Openings shall be taken into consideration when computing A_w. For masonry walls, the net area shall be used. For wood framed walls, the length shall be used rather than the area.
m = component modification factor; m shall be taken from Table 3-7.

Table 3-7. m-factors for Shear Walls

Wall Type	Level of Performance[1]	
	LS	IO
Reinforced Concrete, Precast Concrete, and Wood	4.0	2.0
Reinforced Masonry	3.0	1.5
Unreinforced Masonry	1.5	N/A

[1]Defined in Section 2.4.

3.5.3.4 Diagonal Bracing

The average axial stress in diagonal bracing elements, f_{br}, shall be calculated in accordance with Equation (3-12).

$$f_{br} = \frac{1}{m}\left(\frac{V_j}{sN_{br}}\right)\left(\frac{L_{br}}{A_{br}}\right) \qquad (3\text{-}12)$$

where:

L_{br} = Average length of the braces (ft);
N_{br} = Number of braces in tension and compression if the braces are designed for compression; if not, use the number of braces in tension, if the braces are not designed for compression;
s = Average span length of braced spans (ft);
A_{br} = Average area of a diagonal brace (in^2);

V_j = Maximum story shear at each level (kips);
m = component modification factor; m shall be taken from Table 3-8.

Table 3-8. m-factors for Diagonal Braces

Brace Type	(d/t) *	Level of Performance[1]	
		LS	IO
Tube	$\leq 90/(F_{ye})^{1/2}$	6.0	2.5
	$> 190/(F_{ye})^{1/2}$	3.0	1.5
Pipe	$\leq 1500/F_{ye}$	6.0	2.5
	$> 6000/F_{ye}$	3.0	1.5
Tension-only		3.0	1.5
All others		6.0	2.5

[1]Defined in Section 2.4.
*Interpolation permitted.
$F_{ye} = 1.25F_y$; expected yield stress as defined by Section 4.2.4.4.

3.5.3.5 Precast Connections

The precast connection in precast concrete moment frames shall be able to develop the moment in the girder, M_g, calculated in accordance with Equation (3-13).

$$M_g = \frac{V_j}{m}\left(\frac{n_c}{n_c - n_f}\right)\left(\frac{h}{2}\right) \qquad (3\text{-}13)$$

where:
n_c = Total number of columns;
n_f = Total number of frames in the direction of loading;
V_j = Story shear at the level directly below the connection under consideration;
h = Typical column story height;
m = Component modification factor taken equal to 2.0 for buildings being evaluated to the Life Safety Performance Level and 1.3 for buildings being evaluated to the Immediate Occupancy Performance Level.

3.5.3.6 Axial Stress Due to Overturning

The axial stress of columns subjected to overturning forces, p_{ot}, shall be calculated in accordance with Equation (3-14).

$$p_{ot} = \frac{1}{m}\left(\frac{2}{3}\right)\left(\frac{Vh_n}{Ln_f}\right) \qquad (3\text{-}14)$$

where:
n_f = Total number of frames in the direction of loading;
V = Pseudo lateral force;
h_n = height (in feet) above the base to the roof level.
L = Total length of the frame (in feet);
m = Component modification factor taken equal to 2.0 for buildings being evaluated to the Life Safety Performance Level and 1.3 for buildings being evaluated to the Immediate Occupancy Performance Level.

PORT FACILITY ANALYSIS - Seismic Risk Analysis

Company Name:	Baldridge _Associates	**Date:**	February 20, 2007
Building Name:	South Freight Shed	**Job Number:**	06101
Street Address:	Kawaihae Harbor	**Engineer:**	Steve Baldridge
	Kawaihae, HI, Hawaii 96743	**PE Number/State:**	7876-S

INFORMATION SOURCES

SiteVisit:		**Date:**	
Interviewed:		**Docs Reviewed:**	

BUILDING DESCRIPTION

Building Classification: S3(2B) - Steel Light Frame
Occupancy: Warehousing
Latitude/Longitude: 20.0230 -155.7401
Region: USA, Hawaii
Region Version: 1.00
Evaluation Lifetime (yrs): 30
Uniform Building Code Design Edition: 1968
Year Constructed: 1968
Year Retrofitted:
Building Height (ft): 23
Fundamental Period (s): 0.100000
Area (sf): 5,000
Replacement Cost ($): 500,000
Plan Dimensions: 100'x50'
Exterior North-South Walls: metal girts with metal siding
Exterior East-West Walls: metal girts with metal siding
Roof Deck/Framing: sheet metal roof on metal purlins
Intermediate Floors/Framing: N/A
Ground Floors: concrete slab-on-grade
Columns: steel
Foundation: shallow spread footings
Basement Levels: N/A
Parking Structure: N/A

LATERAL FORCE RESISTING SYSTEM

Floors/Roof: Sheet metal roof on metal purlins; slab-on-grade floor
Walls/Braces: Steel moment-resisting frame (transverse direction); Tension-only braced frames (longitudinal direction)

BUSINESS INTERRUPTION

Max. Loss With No BI: 20
Min. Loss At Abandonment: 40
BI Months At Abandonment: 3
BI Revenue Loss Rate($/Month): 10,000

GEOTECHNICAL DESCRIPTION

Provider:
Date:
UBC Soil Class: DE
Liquefaction Resilience: Low
Liquefaction Susceptibility: Unknown [Assuming - Very Low]
Depth to Water Table (ft): 5
Landslide Susceptibility: Low

Topography:
Soil Conditions:

COMMENTS

Comments:

PORT FACILITY ANALYSIS

Company Name:	Baldridge _Associates	**Date:**	February 20, 2007
Building Name:	South Freight Shed	**Job Number:**	06101
Street Address:	Kawaihae Harbor	**Engineer:**	Steve Baldridge
	Kawaihae, HI, Hawaii 96743	**PE Number/State:**	7876-S

MODIFIED FEMA-310 WORKSHEET

S3(2B)Steel Light Frame

Category	Range	Typical	Modifier
GENERAL BUILDING FEATURES			
Complete load path	T, F	T	T
Interior mezzanines adequately braced	N/A, T, F	T	N/A
No vertical discontinuities	T, F	T	T
Only minor torsion	T, F	T	T
One story	T, F	T	T
LATERAL FORCE RESISTING SYSTEM			
Diagonals pass axial stress check	T, F, 0-20	15	F
No pre-Northridge moment connections	N/A, T, F, 0-20	15	15
Beam penetrations properly sized and located	N/A, T, F, 0-5	2	N/A
Compact members	T, F, 0-15	8	F
Out-of-plane bracing present	T, F, 0-5	5	T
Bottom beam flange bracing	T, F, 0-10	10	F
CONNECTIONS			
Adequate column anchorage	T, F, 0-10	0	T
Wall panels to foundation connection	T, F, 0-10	0	0
Roof panels adequately attached	T, F, 0-10	0	0
Wall panel attachments adequate for seismic forces	T, F, 0-10	0	0
Lateral load path at pile caps	N/A, T, F, 0-10	0	N/A
FLOOR DIAPHRAGMS			
Adequate diaphragm transfer to steel frame	T, F, 0-10	5	0
Reinforcing at re-entrant corner	N/A, T, F, 0-10	0	N/A
Adequate reinforcing at openings	N/A, T, F, 0-5	0	N/A
Other diaphragms meet requirements	N/A, T, F, 0-5	2	N/A
Collectors	T, F, 0-5	2	0
ROOF DIAPHRAGM (ONLY IF 5 STORIES OR LESS)			
Adequate diaphragm transfer to steel frame	T, F, 0-10	5	5
Reinforcing at re-entrant corner	N/A, T, F, 0-10	0	N/A
Adequate reinforcing at openings	N/A, T, F, 0-5	0	N/A
Other diaphragms meet requirements	N/A, T, F, 0-5	2	T
Collectors	T, F, 0-5	2	2

PORT FACILITY ANALYSIS

Company Name:	Baldridge _Associates	**Date:**	February 20, 2007
Building Name:	South Freight Shed	**Job Number:**	06101
Street Address:	Kawaihae Harbor	**Engineer:**	Steve Baldridge
	Kawaihae, HI, Hawaii 96743	**PE Number/State:**	7876-S

MODIFIED FEMA-310 WORKSHEET

Category	Range	Typical	Modifier
UNUSUAL CONDITIONS			
Little deterioration of steel	T, F, 0-5	2	2
Little foundation damage	T, F, 0-5	2	2
Little foundation deterioration	T, F, 0-5	2	2
Adequate overturning resistance	T, F, 0-5	2	T
Ties between foundation elements	N/A, T, F, 0-5	2	T
Lateral force on deep foundations	N/A, T, F, 0-5	2	N/A
Pole buildings	N/A, T, F, 0-5	0	N/A
Insignificant sloping at site	N/A, T, F, 0-5	0	T
SITE DEPENDENT HAZARDS - ACTIVE FAULTS			
Surface fault rupture	N/A, 0-50	0	0
NONSTRUCTURAL EXTERIOR 'WALLS'			
Cladding, glazing, veneer	N/A, T, F, 0-10	5	5
Chimneys	N/A, T, F, 0-5	5	N/A
NONSTRUCTURAL INTERIOR 'WALLS'			
Partitions (HC tile)	N/A, T, F, 0-10	0	N/A
Partitions (pre-cast panels..)	N/A, T, F, 0-10	5	N/A
EXTERIOR ORNAMENTATION			
Parapets, cornices, and appendages	N/A, T, F, 0-10	0	T
INTERIOR ORNAMENTATION			
Building contents and furnishings	T, F, 0-10	5	5
Ceiling systems	T, F, 0-5	5	T
Light fixtures	T, F, 0-5	5	5
MECHANICAL AND ELECTRICAL SYSTEMS			
Mechanical and electrical equipment	T, F, 0-10	5	5
Piping and sprinklers	T, F, 0-5	2	2
Ducts	T, F, 0-5	2	2
Elevators	N/A, T, F, 0-5	2	N/A
HAZARDOUS EXPOSURES - POUNDING			
No adjacent buildings	N/A, T, F, 0-5	0	T

PORT FACILITY ANALYSIS

Company Name:	Baldridge _Associates	**Date:**	February 20, 2007
Building Name:	South Freight Shed	**Job Number:**	06101
Street Address:	Kawaihae Harbor	**Engineer:**	Steve Baldridge
	Kawaihae, HI, Hawaii 96743	**PE Number/State:**	7876-S

MODIFIED FEMA-310 WORKSHEET

Category	Range	Typical	Modifier
HAZARDOUS EXPOSURES - MATERIALS			
No hazardous materials	N/A, T, F, 0-10	0	0
OCCUPANCY (TYPE: WAREHOUSING)			
Interior Construction	-15-0	0	0
SITE DEPENDENT CHARACTERISTICS			
UBC Soil Class	A - E	D	DE
Liquefaction Resilience	Low - High	Low	Low
Liquefaction Susceptibility	V. Low-V. High	Very Low	Unknown
Depth to Water Table (ft)	0-1000+	50	5
Landslide Susceptibility	V. Low-V. High	Very Low	Low

PORT FACILITY ANALYSIS

Company Name:	Baldridge _Associates	**Date:**	February 20, 2007
Building Name:	South Freight Shed	**Job Number:**	06101
Street Address:	Kawaihae Harbor	**Engineer:**	Steve Baldridge
	Kawaihae, HI, Hawaii 96743	**PE Number/State:**	7876-S

VULNERABILITY SUMMARY

Component Modifier Summary

Base Class 90% Fractile Loss at MMI=IX (% of Value): 22

Modifiers to Base Class Loss

Item	Group Modifier (% of Loss)	Sigma (% of Loss)
1. Occupancy type:	0	2.0
2. Connections:	0	0.7
3. Walls:		
A. Exterior	0	2.3
B. Interior	0	0.0
4. Diaphragms:		
A. Floor(s)	-4	1.1
B. Roof	-1	2.2
5. Ornamentation:		
A. Exterior	0	1.7
B. Interior	-2	2.2
6. Mechanical/electrical systems:	0	3.2
7. Unusual conditions:	-3	1.5
8. Hazardous exposures:		
A. Tank and overhanging walls	0	1.7
B. Pounding and adjacent buildings	0	0.4
9. Site dependent hazards:		
A. Proximity of active fault	0	12.8
Total	-10	14.2

Modified Base Class 90% Fractile Loss at MMI=IX (% of Value): 20

Loss vs MMI

MMI	Loss to Facilities (% of Value) 90% Frac. Loss	Mean
V	0	0
VI	1	0
VII	7	4
VIII	14	8
IX	20	12
X	23	13
XI	26	15
XII	30	17

PORT FACILITY ANALYSIS

Company Name:	Baldridge Associates	**Date:**	February 20, 2007
Building Name:	South Freight Shed	**Job Number:**	06101
Street Address:	Kawaihae Harbor Kawaihae, HI, Hawaii 96743	**Engineer:**	Steve Baldridge
		PE Number/State:	7876-S

RISK SUMMARY

Expected Loss Table

Probability of Exceedence	MMI	Loss to Facilities (% of Value)			BI (months)
		PL	SUL	SEL	
50.0% in 30 years 43 year return period	VIII	7	13	7	0
10.0% in 30 years 285 year return period	IX	14	20	12	1
2.0% in 30 years 1485 year return period	X	23	23	13	1
10.0% in 50 years 475 year return period	IX-X	16	PML 21	12	1
2.0% in 50 years 2475 year return period	X	26	24	14	1

Event and Fault Table

Close and Significant Seismic Sources	Maximum Magnitude	Closest Distance (km)	Max. MMI	Max. SUL *	Max. SEL *	Maximum Business Interuption (months)	Percent Contribution **
Kilauea Gridded D***	7.0	10.0	X	24	14	0	18
Kilauea Gridded G	7.0	19.8	IX	20	12	0	<1
Kilauea Gridded B	7.0	20.3	IX	20	12	0	<1
Kilauea Gridded F	7.0	25.9	VIII-IX	18	10	0	<1
Kilauea Gridded Deep D	7.0	30.0	IX	19	11	0	2
Kilauea Gridded A	7.0	32.7	VIII-IX	15	9	0	<1
Hualalai (HUA)	7.0	33.5	VIII-IX	17	10	0	18
Kona (KON)	7.5	40.9	VIII-IX	18	10	0	28
Kilauea Gridded Deep B	7.0	48.6	VII-VIII	12	7	0	<1
Kaoiki (KAO)	7.0	63.2	VII-VIII	10	6	0	3
Kilauea Gridded Deep E	7.0	63.5	VII	8	5	0	<1
Kilauea Gridded Deep F	7.0	69.4	VII	7	4	0	<1
Kilauea Gridded Deep C	7.0	70.8	VII	7	4	0	<1
KAO+HLE+SFL	8.2	73.7	VIII-IX	16	9	0	28
Kilauea Gridded I	6.5	78.5	VI-VII	3	1	0	<1
Hilea (HLE)	7.0	79.7	VII	7	4	0	<1

* Losses to individual events are from shaking only.
** Percent contributions are for the probabilistic 475 year return period risk.
*** Event causing highest loss (from shaking only)

Average Annual Loss ($): 3,230 **Business Interruption Average Annual Loss ($): 11**
Return Period of Major Liquefaction/Landslide: 4689 Years

DISCLAIMERS and OTHER INFORMATION

RESULTS DISCLAIMER

This report, and the analyses, estimates and conclusions are based on scientific data, mathematical and empirical models, and experience of engineers, geologist and geotechnical specialist, using the input specified by the software licensee. Actual losses experienced during any earthquake may differ substantially from these estimates. Neither Risk Engineering, Inc., Degenkolb Engineers, nor any third party supplier of information to this software can be held liable for any inaccuracies in the results obtained by ST-RISK.

SPRINKLER DAMAGE

Substantial building facilities loss has occurred in recent large earthquakes due to fire sprinkler damage. The figures presented herein may not adequately account for these potential losses. If the modifier for sprinklers in the Mechanical and Electrical Systems section of the Modified FEMA-310 Worksheet was 3 or higher, or '?', a more detailed evaluation of potential sprinkler damage should be made and additional loss anticipated.

THIRD PARTY DATA

Much of the data in this report is derived from data provided by the California Geological Survey (CGS), the US Geological Survey (USGS), the Geological Survey of Canada (GSC), as well as other parties. Most of the original data received was modified to make compatible with ST-RISK. None of these parties can be held liable for any inaccuracies inherent in the data or inherent in the modifications.

GLOSSARY

MMI	Modified Mercalli Intensity - A measure of ground motion intensity based on human perception of motion and observed structural damage.
PML	Probable Maximum Loss - The percentage monetary loss (damage/replacement cost x 100) that has a 10 percent chance of being exceeded for a 475-year ground motion.
PL	Probable Loss - For a given time interval, or return period, this is the amount of loss that a property is expected to meet or exceed on an average basis. This combines the probability distribution of hazard with the full damage distribution, representing the best overall assessment of risk.
SUL	Scenario Upper Loss - The percentage monetary loss (damage/replacement cost x 100) that has a 10 percent chance of being exceeded given any defined ground shaking intensity. Equal to PML for 475-year ground shaking.
SEL	Scenario Expected Loss - The expected, or mean, percentage monetary loss (damage/replacement cost x 100) that is predicted given any defined ground shaking intensity.
Mean Loss	The expected, or average, percentage monetary loss (damage/replacement cost x 100) that is predicted for a given ground shaking level.
Sigma	The range of building assessment variation covered by one standard deviation. This represents the uncertainty of characterizing the building properly. This does not include uncertainty in the expected ground motion intensities nor range of expected damage. It is implied that the distribution of uncertainty is truncated at 100% and 0% of building value.
BI	Business Interruption / Loss-of-Use - The number of months that the facility is out of operation.
Base Class Loss	The percentage monetary loss for 90% fractile (damage/replacement cost x 100) assigned to a building class that accounts for type of construction and important construction deficiencies.
Modified Base Class Loss	The percentage monetary loss for 90% fractile assigned to a building class that accounts for the Base Class Loss and location and minor construction deficiencies.
Probability of Exceedence	The probability that the ground shaking level or damage level will be exceeded.
Event Causing Highest Loss	The highest level of intensity due only to shaking that is experienced when considering all earthquakes given a median predicted shaking level.
Maximum Considered Earthquake (MCE)	Loss associated with a 2% in 50 year probability of exceedence.
Uniform Building Code (UBC)	Loss associated with a 10% in 50 year probability of exceedence as defined by new building design provisions found in the Uniform Building Code.
% Contribribution	Percent contribution of fault or fault segment to the 475-year return period risk.

PORT FACILITY ANALYSIS - Seismic Risk Analysis

Company Name:	Baldridge _Associates	**Date:**	February 20, 2007
Building Name:	Transit Shed	**Job Number:**	06101
Street Address:	Kawaihae Harbor Kawaihae, HI, Hawaii 96743	**Engineer:**	Steve Baldridge
		PE Number/State:	7876-S

INFORMATION SOURCES

SiteVisit: **Date:**
Interviewed: **Docs Reviewed:** Drawings dated 02 April 1959

BUILDING DESCRIPTION

Building Classification: S3(2B) - Steel Light Frame
Occupancy: Warehousing
Latitude/Longitude: 20.0230 -155.7401
Region: USA, Hawaii
Region Version: 1.00
Evaluation Lifetime (yrs): 30
Uniform Building Code Design Edition: 1959
Year Constructed: 1959
Year Retrofitted:
Building Height (ft): 25
Fundamental Period (s): 0.100000
Area (sf): 12,000
Replacement Cost ($): 1,200,000
Plan Dimensions: 196'x80'
Exterior North-South Walls: steel girts with metal siding
Exterior East-West Walls: steel girts with metal siding
Roof Deck/Framing: corrugated metal roof on steel channel purlins
Intermediate Floors/Framing: N/A
Ground Floors: concrete slab-on-grade
Columns: steel wide flange
Foundation: precast concrete piles
Basement Levels: N/A
Parking Structure: N/A

LATERAL FORCE RESISTING SYSTEM

Floors/Roof: Corrugated metal roof on steel purlins; slab-on-grade floor
Walls/Braces: Steel moment-resisting frame and end-wall tension-only braced frames (transverse direction); Tension-only braced frames (longitudinal direction)

BUSINESS INTERRUPTION

Max. Loss With No BI: 20
Min. Loss At Abandonment: 40
BI Months At Abandonment: 3
BI Revenue Loss Rate($/Month): 10,000

Company Name:	Baldridge _Associates	**Date:**	February 20, 2007
Building Name:	Transit Shed	**Job Number:**	06101
Street Address:	Kawaihae Harbor Kawaihae, HI, Hawaii 96743	**Engineer:**	Steve Baldridge
		PE Number/State:	7876-S

GEOTECHNICAL DESCRIPTION

Provider:
Date:
UBC Soil Class: DE
Liquefaction Resilience: Moderate
Liquefaction Susceptibility: Unknown [Assuming - Very Low]
Depth to Water Table (ft): 5
Landslide Susceptibility: Low

Topography:
Soil Conditions:

COMMENTS

Comments:

PORT FACILITY ANALYSIS

Company Name:	Baldridge _Associates	**Date:**	February 20, 2007
Building Name:	Transit Shed	**Job Number:**	06101
Street Address:	Kawaihae Harbor	**Engineer:**	Steve Baldridge
	Kawaihae, HI, Hawaii 96743	**PE Number/State:**	7876-S

MODIFIED FEMA-310 WORKSHEET

S3(2B)Steel Light Frame

Category	Range	Typical	Modifier
GENERAL BUILDING FEATURES			
Complete load path	T, F	T	T
Interior mezzanines adequately braced	N/A, T, F	T	N/A
No vertical discontinuities	T, F	T	T
Only minor torsion	T, F	T	T
One story	T, F	T	T
LATERAL FORCE RESISTING SYSTEM			
Diagonals pass axial stress check	T, F, 0-20	15	F
No pre-Northridge moment connections	N/A, T, F, 0-20	15	15
Beam penetrations properly sized and located	N/A, T, F, 0-5	2	N/A
Compact members	T, F, 0-15	8	10
Out-of-plane bracing present	T, F, 0-5	5	3
Bottom beam flange bracing	T, F, 0-10	10	T
CONNECTIONS			
Adequate column anchorage	T, F, 0-10	0	T
Wall panels to foundation connection	T, F, 0-10	0	0
Roof panels adequately attached	T, F, 0-10	0	0
Wall panel attachments adequate for seismic forces	T, F, 0-10	0	0
Lateral load path at pile caps	N/A, T, F, 0-10	0	F
FLOOR DIAPHRAGMS			
Adequate diaphragm transfer to steel frame	T, F, 0-10	5	0
Reinforcing at re-entrant corner	N/A, T, F, 0-10	0	N/A
Adequate reinforcing at openings	N/A, T, F, 0-5	0	N/A
Other diaphragms meet requirements	N/A, T, F, 0-5	2	N/A
Collectors	T, F, 0-5	2	0
ROOF DIAPHRAGM (ONLY IF 5 STORIES OR LESS)			
Adequate diaphragm transfer to steel frame	T, F, 0-10	5	5
Reinforcing at re-entrant corner	N/A, T, F, 0-10	0	N/A
Adequate reinforcing at openings	N/A, T, F, 0-5	0	N/A
Other diaphragms meet requirements	N/A, T, F, 0-5	2	T
Collectors	T, F, 0-5	2	2

PORT FACILITY ANALYSIS

Company Name:	Baldridge _Associates	**Date:**	February 20, 2007
Building Name:	Transit Shed	**Job Number:**	06101
Street Address:	Kawaihae Harbor	**Engineer:**	Steve Baldridge
	Kawaihae, HI, Hawaii 96743	**PE Number/State:**	7876-S

MODIFIED FEMA-310 WORKSHEET

Category	Range	Typical	Modifier
UNUSUAL CONDITIONS			
Little deterioration of steel	T, F, 0-5	2	2
Little foundation damage	T, F, 0-5	2	2
Little foundation deterioration	T, F, 0-5	2	2
Adequate overturning resistance	T, F, 0-5	2	T
Ties between foundation elements	N/A, T, F, 0-5	2	2
Lateral force on deep foundations	N/A, T, F, 0-5	2	2
Pole buildings	N/A, T, F, 0-5	0	N/A
Insignificant sloping at site	N/A, T, F, 0-5	0	T
SITE DEPENDENT HAZARDS - ACTIVE FAULTS			
Surface fault rupture	N/A, 0-50	0	0
NONSTRUCTURAL EXTERIOR 'WALLS'			
Cladding, glazing, veneer	N/A, T, F, 0-10	5	5
Chimneys	N/A, T, F, 0-5	5	N/A
NONSTRUCTURAL INTERIOR 'WALLS'			
Partitions (HC tile)	N/A, T, F, 0-10	0	N/A
Partitions (pre-cast panels..)	N/A, T, F, 0-10	5	N/A
EXTERIOR ORNAMENTATION			
Parapets, cornices, and appendages	N/A, T, F, 0-10	0	T
INTERIOR ORNAMENTATION			
Building contents and furnishings	T, F, 0-10	5	5
Ceiling systems	T, F, 0-5	5	T
Light fixtures	T, F, 0-5	5	5
MECHANICAL AND ELECTRICAL SYSTEMS			
Mechanical and electrical equipment	T, F, 0-10	5	5
Piping and sprinklers	T, F, 0-5	2	2
Ducts	T, F, 0-5	2	2
Elevators	N/A, T, F, 0-5	2	N/A
HAZARDOUS EXPOSURES - POUNDING			
No adjacent buildings	N/A, T, F, 0-5	0	T

PORT FACILITY ANALYSIS

Company Name:	Baldridge _Associates	**Date:**	February 20, 2007
Building Name:	Transit Shed	**Job Number:**	06101
Street Address:	Kawaihae Harbor	**Engineer:**	Steve Baldridge
	Kawaihae, HI, Hawaii 96743	**PE Number/State:**	7876-S

MODIFIED FEMA-310 WORKSHEET

Category	Range	Typical	Modifier
HAZARDOUS EXPOSURES - MATERIALS			
No hazardous materials	N/A, T, F, 0-10	0	0
OCCUPANCY (TYPE: WAREHOUSING)			
Interior Construction	-15-0	0	0
SITE DEPENDENT CHARACTERISTICS			
UBC Soil Class	A - E	D	DE
Liquefaction Resilience	Low - High	Low	Moderate
Liquefaction Susceptibility	V. Low-V. High	Very Low	Unknown
Depth to Water Table (ft)	0-1000+	50	5
Landslide Susceptibility	V. Low-V. High	Very Low	Low

PORT FACILITY ANALYSIS

Company Name:	Baldridge _Associates	**Date:**	February 20, 2007
Building Name:	Transit Shed	**Job Number:**	06101
Street Address:	Kawaihae Harbor	**Engineer:**	Steve Baldridge
	Kawaihae, HI, Hawaii 96743	**PE Number/State:**	7876-S

VULNERABILITY SUMMARY

Component Modifier Summary

Base Class 90% Fractile Loss at MMI=IX (% of Value): 19

Modifiers to Base Class Loss

Item	Group Modifier (% of Loss)	Sigma (% of Loss)
1. Occupancy type:	0	2.0
2. Connections:	2	1.2
3. Walls:		
A. Exterior	0	2.3
B. Interior	0	0.0
4. Diaphragms:		
A. Floor(s)	-4	1.1
B. Roof	-1	2.2
5. Ornamentation:		
A. Exterior	0	1.7
B. Interior	-2	2.2
6. Mechanical/electrical systems:	0	3.2
7. Unusual conditions:	-2	1.8
8. Hazardous exposures:		
A. Tank and overhanging walls	0	1.7
B. Pounding and adjacent buildings	0	0.4
9. Site dependent hazards:		
A. Proximity of active fault	0	12.8
Total	-7	14.3

Modified Base Class 90% Fractile Loss at MMI=IX (% of Value): 18

Loss vs MMI

MMI	Loss to Facilities (% of Value) 90% Frac. Loss	Mean
V	0	0
VI	1	0
VII	7	4
VIII	12	7
IX	18	10
X	21	12
XI	24	14
XII	27	15

PORT FACILITY ANALYSIS

Company Name:	Baldridge _Associates	**Date:**	February 20, 2007
Building Name:	Transit Shed	**Job Number:**	06101
Street Address:	Kawaihae Harbor Kawaihae, HI, Hawaii 96743	**Engineer:**	Steve Baldridge
		PE Number/State:	7876-S

RISK SUMMARY

Expected Loss Table

Probability of Exceedence	MMI	Loss to Facilities (% of Value)			BI (months)
		PL	SUL	SEL	
50.0% in 30 years 43 year return period	VIII	6	11	7	0
10.0% in 30 years 285 year return period	IX	13	18	10	0
2.0% in 30 years 1485 year return period	X	21	21	12	1
10.0% in 50 years 475 year return period	IX-X	15	PML 19	11	0
2.0% in 50 years 2475 year return period	X	23	22	13	1

Event and Fault Table

Close and Significant Seismic Sources	Maximum Magnitude	Closest Distance (km)	Max. MMI	Max. SUL *	Max. SEL *	Maximum Business Interuption (months)	Percent Contribution **
Kilauea Gridded D***	7.0	10.0	X	21	12	0	18
Kilauea Gridded G	7.0	19.8	IX	18	10	0	<1
Kilauea Gridded B	7.0	20.3	IX	18	10	0	<1
Kilauea Gridded F	7.0	25.9	VIII-IX	16	9	0	<1
Kilauea Gridded Deep D	7.0	30.0	IX	18	10	0	2
Kilauea Gridded A	7.0	32.7	VIII-IX	14	8	0	<1
Hualalai (HUA)	7.0	33.5	VIII-IX	15	9	0	18
Kona (KON)	7.5	40.9	VIII-IX	16	9	0	28
Kilauea Gridded Deep B	7.0	48.6	VII-VIII	10	6	0	<1
Kaoiki (KAO)	7.0	63.2	VII-VIII	9	5	0	3
Kilauea Gridded Deep E	7.0	63.5	VII	8	4	0	<1
Kilauea Gridded Deep F	7.0	69.4	VII	7	4	0	<1
Kilauea Gridded Deep C	7.0	70.8	VII	6	4	0	<1
KAO+HLE+SFL	8.2	73.7	VIII-IX	14	8	0	28
Kilauea Gridded I	6.5	78.5	VI-VII	2	1	0	<1
Hilea (HLE)	7.0	79.7	VII	6	4	0	<1

* Losses to individual events are from shaking only.
** Percent contributions are for the probabilistic 475 year return period risk.
*** Event causing highest loss (from shaking only)

Average Annual Loss ($): 7,150 **Business Interruption Average Annual Loss ($): 7**
Return Period of Major Liquefaction/Landslide: 4689 Years

DISCLAIMERS and OTHER INFORMATION

RESULTS DISCLAIMER

This report, and the analyses, estimates and conclusions are based on scientific data, mathematical and empirical models, and experience of engineers, geologist and geotechnical specialist, using the input specified by the software licensee. Actual losses experienced during any earthquake may differ substantially from these estimates. Neither Risk Engineering, Inc., Degenkolb Engineers, nor any third party supplier of information to this software can be held liable for any inaccuracies in the results obtained by ST-RISK.

SPRINKLER DAMAGE

Substantial building facilities loss has occurred in recent large earthquakes due to fire sprinkler damage. The figures presented herein may not adequately account for these potential losses. If the modifier for sprinklers in the Mechanical and Electrical Systems section of the Modified FEMA-310 Worksheet was 3 or higher, or '?', a more detailed evaluation of potential sprinkler damage should be made and additional loss anticipated.

THIRD PARTY DATA

Much of the data in this report is derived from data provided by the California Geological Survey (CGS), the US Geological Survey (USGS), the Geological Survey of Canada (GSC), as well as other parties. Most of the original data received was modified to make compatible with ST-RISK. None of these parties can be held liable for any inaccuracies inherent in the data or inherent in the modifications.

GLOSSARY

MMI	Modified Mercalli Intensity - A measure of ground motion intensity based on human perception of motion and observed structural damage.
PML	Probable Maximum Loss - The percentage monetary loss (damage/replacement cost x 100) that has a 10 percent chance of being exceeded for a 475-year ground motion.
PL	Probable Loss - For a given time interval, or return period, this is the amount of loss that a property is expected to meet or exceed on an average basis. This combines the probability distribution of hazard with the full damage distribution, representing the best overall assessment of risk.
SUL	Scenario Upper Loss - The percentage monetary loss (damage/replacement cost x 100) that has a 10 percent chance of being exceeded given any defined ground shaking intensity. Equal to PML for 475-year ground shaking.
SEL	Scenario Expected Loss - The expected, or mean, percentage monetary loss (damage/replacement cost x 100) that is predicted given any defined ground shaking intensity.
Mean Loss	The expected, or average, percentage monetary loss (damage/replacement cost x 100) that is predicted for a given ground shaking level.
Sigma	The range of building assessment variation covered by one standard deviation. This represents the uncertainty of characterizing the building properly. This does not include uncertainty in the expected ground motion intensities nor range of expected damage. It is implied that the distribution of uncertainty is truncated at 100% and 0% of building value.
BI	Business Interruption / Loss-of-Use - The number of months that the facility is out of operation.
Base Class Loss	The percentage monetary loss for 90% fractile (damage/replacement cost x 100) assigned to a building class that accounts for type of construction and important construction deficiencies.
Modified Base Class Loss	The percentage monetary loss for 90% fractile assigned to a building class that accounts for the Base Class Loss and location and minor construction deficiencies.
Probability of Exceedence	The probability that the ground shaking level or damage level will be exceeded.
Event Causing Highest Loss	The highest level of intensity due only to shaking that is experienced when considering all earthquakes given a median predicted shaking level.
Maximum Considered Earthquake (MCE)	Loss associated with a 2% in 50 year probability of exceedence.
Uniform Building Code (UBC)	Loss associated with a 10% in 50 year probability of exceedence as defined by new building design provisions found in the Uniform Building Code.
% Contribribution	Percent contribution of fault or fault segment to the 475-year return period risk.

Appendix F
Excerpts from Seismic Mitigation References

Tables

F-1 ASCE Liquefaction Remediation Measures .....
F-2 INA Liquefaction Remediation Measures .....

Figures

F-1 Basic Strategy for Liquefaction Remediation .....
F-2 Examples of Soil Improvement for Waterfront Retaining Structures .....
F-3 Improvement Area for Sheet-Pile Walls.....
F-4 Liquefaction Remediation Measures for Sheet-Pile-Type Wharf .....
F-5 Liquefaction Remediation Example for Caisson Quay Wall.....
F-6 Lateral Spreading Failure of Piles.....
F-7 Port Experiences with Natural Disasters Since 1998.....
F-8 Port Elements .....

TABLE 4-5: LIQUEFACTION REMEDIATION MEASURES (AFTER FERRITTO, 1997b)

Method	Principle	Most Suitable Soil Conditions or Types	Maximum Effective Treatment Depth	Relative Costs
1) Vibratory Probe a) Terraprobe b) Vibrorods c) Vibrowing	Densification by vibration; liquefaction-induced settlement and settlement in dry soil under overburden to produce a higher density.	Saturated or dry clean sand; sand.	20 m routinely (ineffective above 3-4 m depth); > 30 m sometimes; vibrowing, 40 m.	Moderate
2) Vibrocompaction a) Vibrofloat b) Vibro-Composer system.	Densification by vibration and compaction of backfill material of sand or gravel.	Cohesionless soils with less than 20% fines.	> 20 m	Low to moderate
3) Compaction Piles	Densification by displacement of pile volume and by vibration during driving, increase in lateral effective earth pressure.	Loose sandy soil; partly saturated clayey soil; loess.	> 20 m	Moderate to high
4) Heavy tamping (dynamic compaction)	Repeated application of high-intensity impacts at surface.	Cohesionless soils best, other types can also be improved.	30 m (possibly deeper)	Low
5) Displacement (compaction grout)	Highly viscous grout acts as radial hydraulic jack when pumped in under high pressure.	All soils.	Unlimited	Low to moderate
6) Surcharge/buttress	The weight of a surcharge/buttress increases the liquefaction resistance by increasing the effective confining pressures in the foundation.	Can be placed on any soil surface.	Dependent on size of surcharge/buttress	Moderate if vertical drains are used
7) Drains a) Gravel b) Sand c) Wick d) Wells (for permanent dewatering)	Relief of excess pore water pressure to prevent liquefaction. (Wick drains have comparable permeability to sand drains). Primarily gravel drains; sand/wick may supplement gravel drain or relieve existing excess pore water pressure. Permanent dewatering with pumps.	Sand, silt, clay.	Gravel and sand > 30 m; depth limited by vibratory equipment; wick, > 45 m	Moderate to high
8) Particulate grouting	Penetration grouting-fill soil pores with soil, cement, and/or clay.	Medium to coarse sand and gravel.	Unlimited	Lowest of grout methods
9) Chemical grouting	Solutions of two or more chemicals react in soil pores to form a gel or a solid precipitate.	Medium silts and coarser.	Unlimited	High
10) Pressure injected lime	Penetration grouting-fill soil pores with lime	Medium to coarse sand and gravel.	Unlimited	Low

Method	Principle	Most Suitable Soil Conditions or Types	Maximum Effective Treatment Depth	Relative Costs
11) Electrokinetic injection	Stabilizing chemical moved into and fills soil pores by electro-osmosis or colloids in to pores by electrphoresis.	Saturated sands, silts, silty clays.	Unknown	Expensive
12) Jet grouting	High-speed jets at depth excavate, inject, and mix a stabilizer with soil to form columns or panels.	Sands, silts, clays.	Unknown	High
13) Mix-in-place piles and walls	Lime, cement or asphalt introduced through rotating auger or special in-place mixer.	Sand, silts, clays, all soft or loose inorganic soils.	> 20 m (60 m obtained in Japan)	High
14) Vibro-replacement stone and sand columns a) Grouted b) Not grouted	Hole jetted into fine-grained soil and backfilled with densely compacted gravel or sand hole formed in cohesionless soils by vibro techniques and compaction of backfilled gravel or sand. For grouted columns, voids filled with a grout.	Sands, silts, clays.	> 30 m (limited by vibratory equipment)	Moderate
15) Root piles, soil nailing	Small-diameter inclusions used to carry tension, shear, compression.	All soils.	Unknown	Moderate to high
16) Blasting	Shock waves an vibrations cause limited liquefaction, displacement, remolding, and settlement to higher density.	Saturated, clean sand; partly saturated sands and silts after flooding.	> 40 m	Low

Table F-1 ASCE Liquefaction Remediation Measures

Source: Ferritto 1997b, Table 4-5, as cited in ASCE TCLEE No.12.

Table T6.1. Remedial measures against liquefaction.

Principle	Methods Designation	Depth	Summary	Influence on the surrounding areas	Remarks
Compaction	Sand compaction pile method	About GL-35 m	Insert steel pipe casing underground Install sand compaction pile by forcing out sand during extraction of casing, and compact the sand pile and the natural soil in a horizontal direction simultaneously	This method produces high levels of noise and vibration The extent of noise and vibration differs according to the type of construction equipment used	Compaction efficiency is high for soil with fines content of less than 25 to 30%. SPT *N*-value increases to about 25 to 30
	Vibro-rod method	About GL-20 m	Compaction of soil is conducted using vibratory penetration of a rod and filling with additional sand from the ground surface	This method produces noise and vibration that are slightly less than those produced by the sand compaction pile method	Compaction efficiency is high with respect to soil having fines content of less than 15 to 20%. SPT *N*-value increases to about 15 to 20
	Vibroflotation method	About GL-20 m	The surrounding soil is compacted with water sprayed from nozzle tip end and with horizontal vibration from vibrator with a built-in eccentric load Additional sands are filled into the voids in the subsoil profile	This method produces little noise and vibration compared to other methods based on compaction	Compaction efficiency is high with respect to soil having fines content of less than 15 to 20%. SPT *N*-value increases to about 15 to 20
	Dynamic compaction method	About GL-10 m	The soil is compacted with an impact load by dropping a weight of 10 to 30 tf from a height of about 25 to 40 m above the ground	This method produces high levels of vibration and large impact	Compaction is difficult when the fines content is high
Pore water pressure dissipation	Gravel drain method	About GL-20 m	The gravel pile is installed by inserting gravel into a casing placed at prescribed position and then extracting the casing. Excess pore water pressure is dissipated through the gravel pile during earthquakes	Influence on the surrounding areas is minimal	This method is often used when compaction is difficult. If fines content of the subsoil is high and permeability is low, application of this method is difficult
	Attachment of drainage device for steel pipes or sheet piles	—	Piles with a drainage device are inserted into the subsoil as liquefaction remedial measures	Noise and vibration depend on the installation method used	The pile prevents uplift and settlement of upper-structures.
Cementation and solidification	Deep mix method	About GL-30 m	Stabilizing material such as cement is mixed and solidified in the soil. There are two approaches: one is total improvement which solidifies all the soil, the other is partial improvement which makes a solidified wall in the subsoil profile	Influence on the surrounding areas is minimal	Liquefaction remediation is possible even when there is a possibility of liquefaction in the soil under the existing structure, such as for an embankment, by installing an improvement wall at the periphery

Table T6.1. Continued.

Principle	Methods Designation	Depth	Summary	Influence on the surrounding areas	Remarks
	Premix method	—	Landfill is deposited by adding and mixing a stabilizing material such as cement in advance in the soil. Liquefaction remediation following the landfill becomes unnecessary	Water quality control is necessary since sand mixed with a stabilizing material is dropped into water	This method can reduce the construction period because liquefaction remediation can be accomplished simultaneously with landfilling, and treatment can be performed in a large volume
Replacement	Replacement method	About GL-5 m	The liquefiable soil is replaced with a non-liquefiable material	Handling of the excavated soil is necessary	The construction is reliable since the liquefiable soil is replaced with non-liquefiable material
Lowering of ground water level	Deep well method	Water level lowering of about 15–20 m	The perimeter of the liquefiable soil area is surrounded with a cut off retaining wall constructed with sheet piles, and the ground water level is lowered using deep wells	Caution should be exercised with regard to consolidation and settlement of the soil and influence on the surrounding area from the lowering of the ground water level	This method can be applied to the subsoils below existing structures. The long-term operating cost should be considered
Shear strain restraint	Underground wall method	—	Liquefaction remediation can be achieved by surrounding the liquefiable soil using underground walls made of grouting or sheet piles, and restraining the shear deformation of subsoil during earthquakes	Influence on the surrounding areas is minimal	This method can also prevent the propagation of excess pore water pressure from the surrounding liquefied soil
Preload	Soil overlay	Depending on embank-ment width	Preload is applied by overlaying soils in order to overconsolidate the subsoil	Influence on the surrounding areas is minimal	The stress propagation in the soil should be considered since there is load dispersion
	Deep well method	Water level lowering of about 15–20 m	The ground water level of liquefiable soil is lowered temporarily and thereafter restored	Caution should be exercised with regard to consolidation and settlement of the soil and influence on the surrounding area from the lowering of the ground water level	This method can be applied to the subsoils below existing structures
Structural strengthening	Pile foundation	—	The strength of the pile increases resistance even if liquefaction occurs	Influence on the surrounding areas is minimal	—
	Sheet pile	—	Shear deformation after liquefaction is restrained by surrounding the soil with sheet piles	There is noise and vibration during sheet pile installation	This method can also prevent propagation of excess pore water pressure from the surrounding liquefied soil
	Top-shaped foundation	—	Used for spread foundation by laying top-shaped concrete blocks on the soil	Influence on the surrounding areas is minimal	This method is applied to small buildings such as residential housing

Table F-2 Liquefaction Remediation Measures

Ref: INA Seismic Design Guidelines Table T6.1

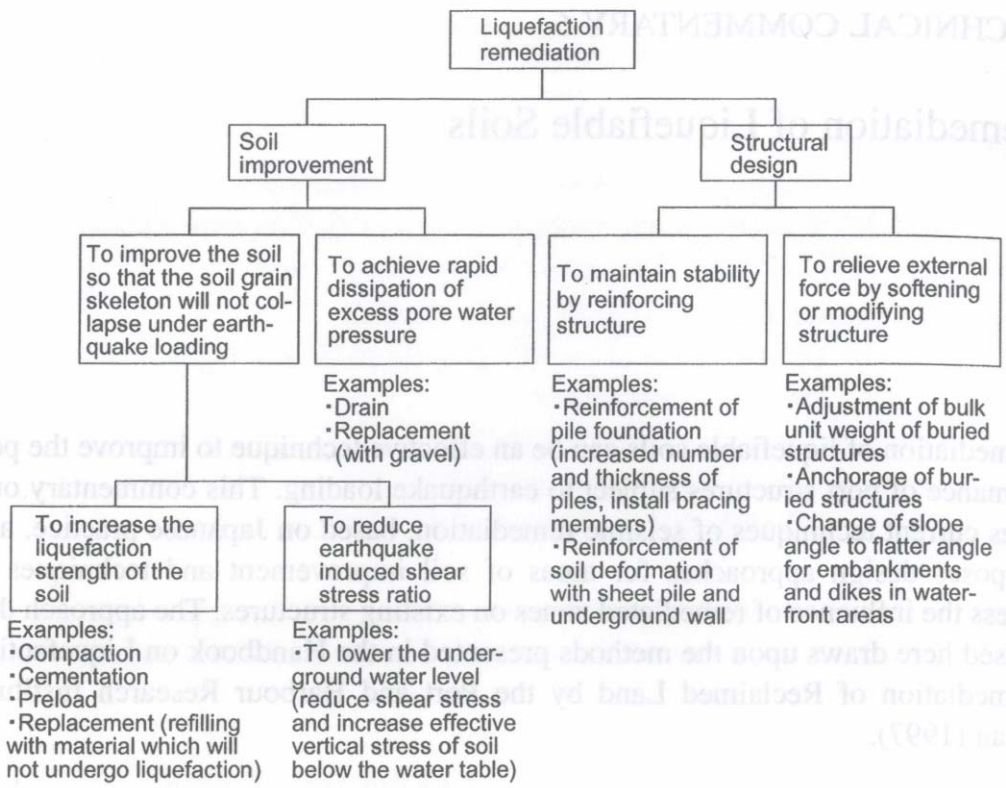


Fig. T6.1. Basic strategy for liquefaction remediation.

Figure F-1 Basic Strategy for Liquefaction Remediation

Source: INA ____, Figure T6.1.

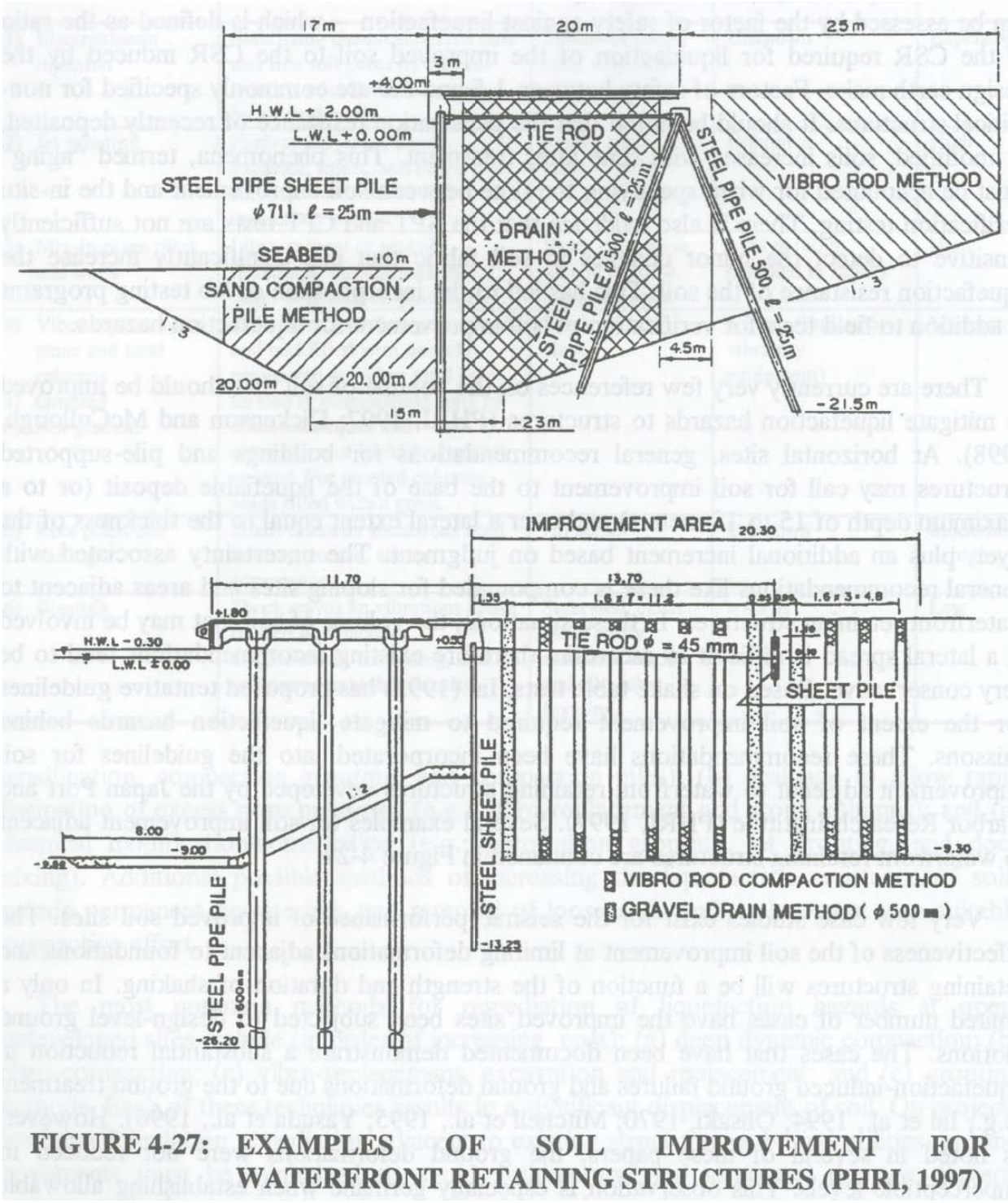


Figure F-2 Examples of Soil Improvement for Waterfront Retaining Structures

Ref: ASCE TCLEE No.12 Figure 4-27, PHRI (1997)

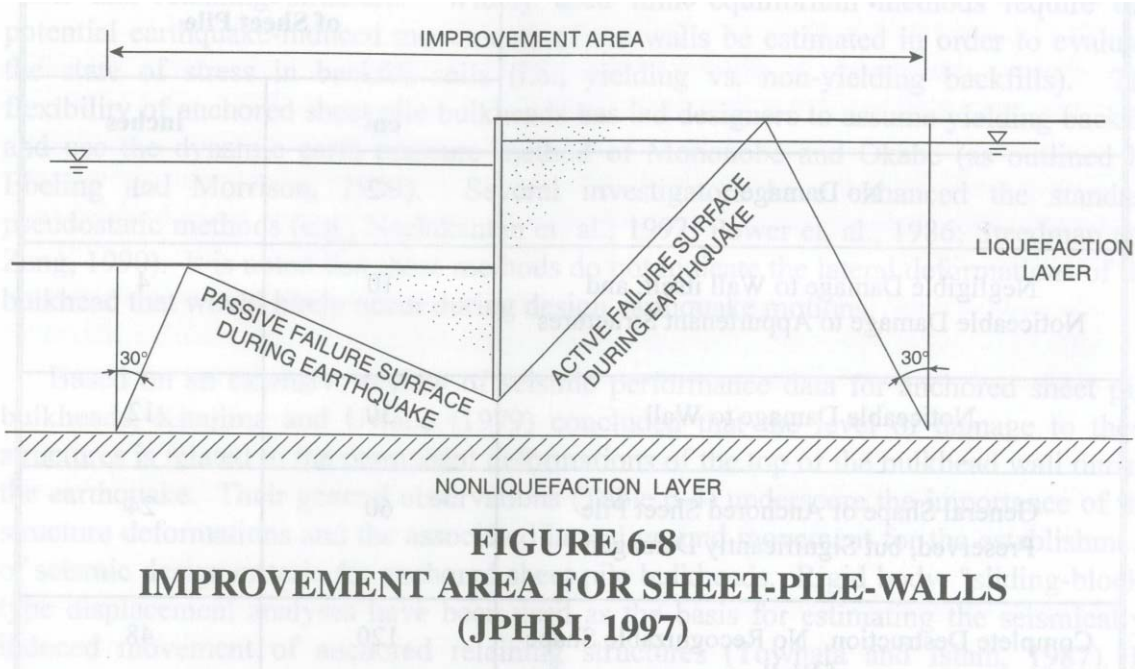


FIGURE 6-8
IMPROVEMENT AREA FOR SHEET-PILE-WALLS
(JPHRI, 1997)

Figure F-3 Improvement Area for Sheet-Pile Walls

Ref: ASCE TCLEE No.12 Figure 6-8, PHRI (1997)

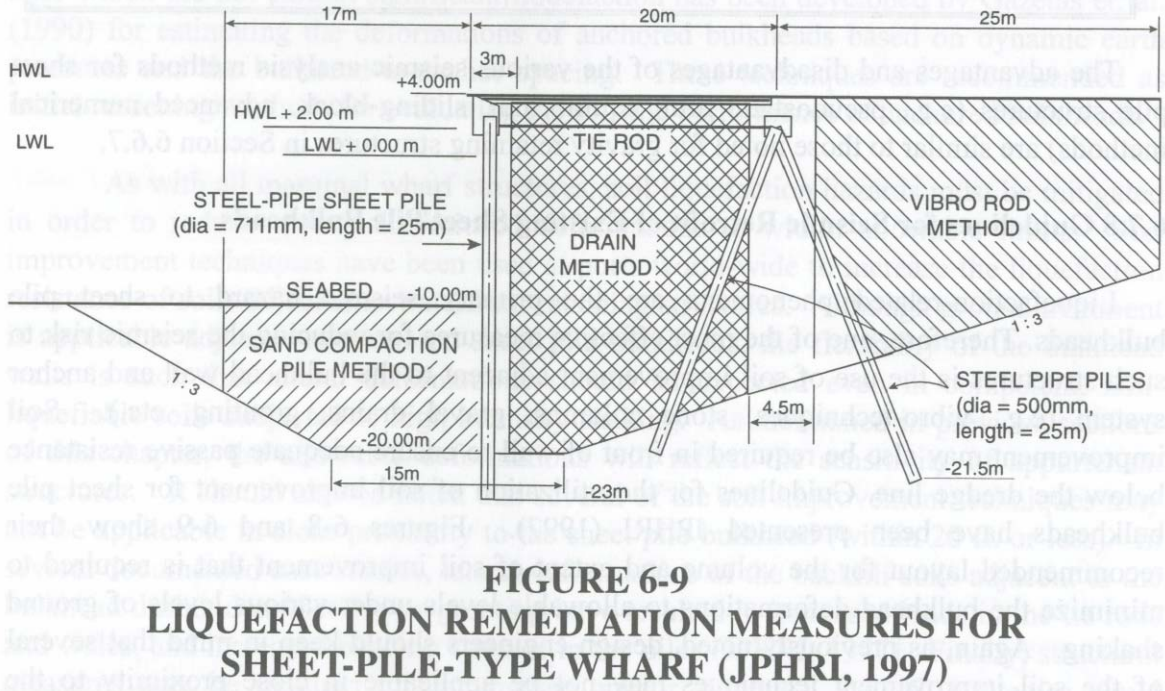


Figure F-4 Liquefaction Remediation Measures for Sheet-Pile-Type Wharf

Ref: ASCE TCLEE No.12 Figure 6-9, JPHRI (1997)

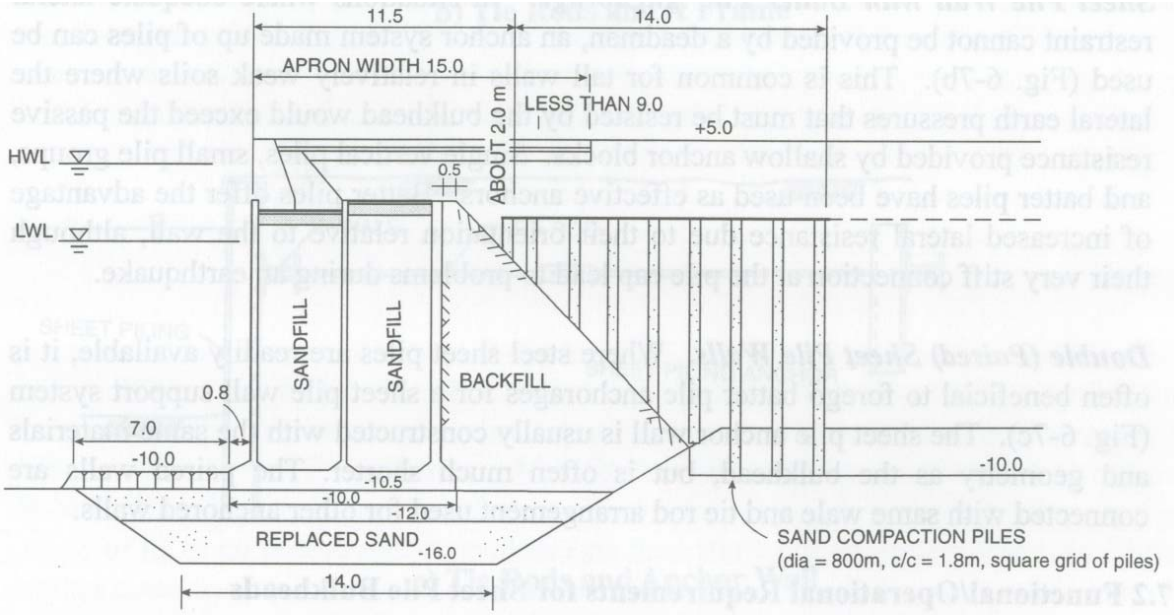
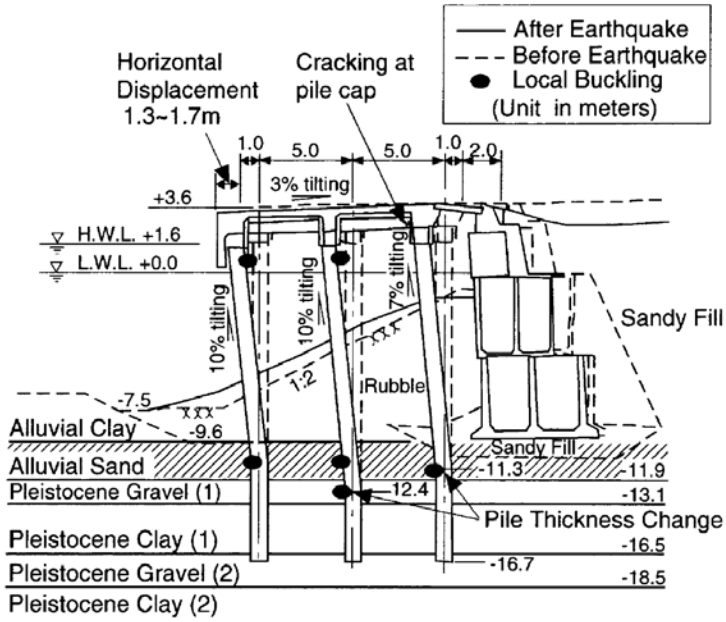


FIGURE 6-6
EXAMPLE APPLICATION OF SOIL IMPROVEMENT TO REDUCE LIQUEFACTION HAZARDS AT CAISSON QUAY WALL (JPHRI, 1997)

Figure F-5 Liquefaction Remediation Example for Caisson Quay Wall

Ref: ASCE TCLEE No.12 Figure 6-6, JPHRI (1997)



3. Cross section of a pile-supported wharf at Takahama Wharf, Kobe Port, Japan, and damage during the Great Hanshin earthquake of 1995.

4. Damage to piles at Takahama Wharf, Kobe Port (after extraction for inspection).

Figure F-6 Lateral Spreading Failure of Piles

Ref: ANI Seismic Design Guidelines, Figures 2.13 and 2.14

Figure 2: Port Experiences with Natural Disasters Since 1998

Ports	Threats	Events	Challenges						
Freeport, TX		Frances/Rita							
Gulfport, MS		Katrina							
Houston, TX		Rita							
Jacksonville, FL		Ernesto							
Los Angeles, CA									
Miami, FL		Katrina							
		Rita							
Mobile, AL		Katrina							
Morgan City, LA		Katrina							
		Rita							
New Orleans, LA		Katrina							
		Rita							
		Wilma							
Oakland, CA									
Pascagoula, MS		Katrina							
Port Arthur, TX		Rita							
Richmond, VA									
San Diego, CA									
Savannah, GA									
Tacoma, WA		Nisqually							
Wilmington, NC		Ophelia, Katrina							
		Ernesto, Floyd							

Threat of hurricane; No recent events; Personnel; Utilities (power); Coordination; Gray icons indicate negligible or no challenge

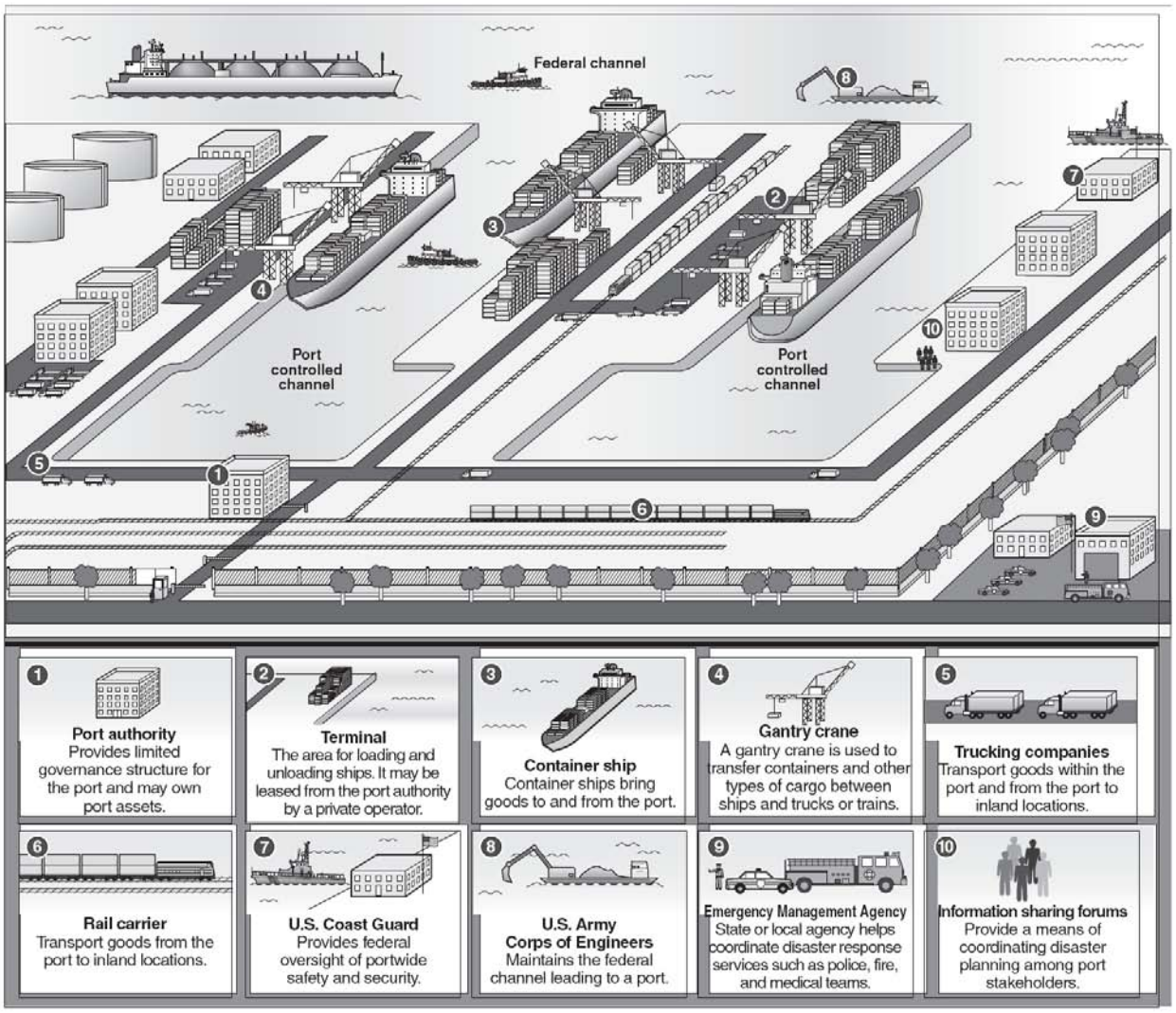
Threat of earthquake; Infrastructure; Communication; Utilites (water); Clearing debris/ aids to navigation

Source: GAO.

Figure F-7 Port Experiences with Natural Disasters Since 1998

Ref: GAO Port Risk Management, Figure 2, 2007

Figure 3: Port Elements



Source: GAO.

Figure F-8 Port Elements

Ref: GAO Port Risk Management, Figure 3, 2007

References

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