# 2. SITE CHARACTERIZATION

## 2.1 Regional Geology

Hawaii, the largest island of the Hawaiian Archipelago, covers an area of approximately 4,000 square miles. This island was formed by the activity of the following five shield volcanoes: Kohala (long extinct), Mauna Kea (activity during recent geologic time), Hualalai (last erupted in 1801), and Mauna Loa and Kilauea (both still active).

The project site is located on the northeastern flank of the Mauna Kea Shield Volcano, which had been built up the successive accumulation of basaltic lava flows and pyroclastic materials. The most recent lava flows that comprise the ground surface and shallow subsurface at the project site are believed to be Pliocene to Pleistocene in age (approximately 1.2 million years to 2 million years before present time) and belong to the upper member of the Hamakua Volcanic Series.

The lava flows were subsequently covered by volcanic ash deposits, locally referred to as Pahala Ash. The Pahala Ash is an air-laid deposit of volcanic ash from the Hamakua Volcanic Series of Mauna Loa Mountain during the Pleistocene Epoch. The Pahala Ash in certain locations has high in-situ moisture contents and low in-situ densities. It generally has low shear strength, and when the moisture content is high enough, it becomes thixotropic, i.e., it loses strength when remolded. Therefore, this type of Pahala Ash may be potentially liquefiable during seismic events.

The lava formation appears to be of pahoehoe flow that is characterized by a smooth, rope-like or billowy surface and an internal structure of vesicular (porous) rock. Cavities are commonly encountered in pahoehoe lavas. Cavities are formed when the lava is still in a molten state and represent both lava tubes (intra-flow cavities) and interflow cavities (blisters and pockets). Lava tubes are formed when molten lava drains from the cooling flow, leaving a hollow tube-like structure that extends for a large longitudinal distance along the flow. Inter-flow cavities are generally smaller in horizontal extent.

## 2.2 Site Description

The Kaholo Bridge is located along Hawaii Belt Road (Route 19) in the District of Hamakua on the Island of Hawaii. The existing Kaholo Bridge structure traverses Kaholo Stream. The bridge site is shown on the Site Plan, Plate 2.

The existing bridge structure is a three-span bridge supported by two intermediate piers and abutments at both ends. Based on the available plans, the existing bridge structure is supported on shallow foundations. The bridge structure is approximately 220 feet in length and is approximately 29.7 feet in width. Abutment Nos. 1 (Hilo side) and 2 (Honokaa side) are located at the east and west ends, respectively. Pier No. 1 is located on the east side of Kaholo Stream Bank, and Pier No. 2 is located on the west side of Kaholo Stream Bank.

The existing ground surface elevations where the seismic retrofit improvements are planned range from about +715 to +724 feet Mean Sea Level (MSL) around the Hilo side abutment and about +738 to +744 feet MSL around the Honokaa side abutment.

The existing bridge is generally aligned in an east-west direction. However, the stream flows below the existing bridge in a southwest to northeast direction. At the time of our field exploration, the streambed was generally dry.

## 2.3 Subsurface Conditions

We explored the subsurface conditions by drilling and sampling four borings, designated as Boring Nos. 1 through 4, extending to depths ranging from about 76 to 102.5 feet below the existing ground surface. The approximate boring locations are shown on the Site Plan, Plate 2.

Boring Nos. 1 and 2 were drilled at the Hilo side abutment. In general, subsurface conditions encountered at the Hilo side abutment consist of 8 to 9 inches of asphaltic concrete over loose to medium dense gravel and sand fill extending to depths of about 1 to 3.5 feet below the existing ground surface. The fill was underlain by residual soils, saprolite and weathered basalt consisting of medium stiff to hard clayey silt and medium dense silty gravel extending to a depth of about 28 feet below the existing ground surface. The residual soils, saprolite and weathered basalt were underlain by highly to moderately weathered, soft to medium hard basalt rock interbedded with layers of clayey silt. The highly to moderately weathered basalt rock extended to depths of about 52 to 63 feet below the existing ground surface. The basalt rock was graded moderately to slightly weathered and medium hard to hard down to the maximum depth explored of about 102.5 feet below the existing ground surface.

Boring Nos. 3 and 4 were drilled at the Honokaa side abutment. In general, subsurface conditions encountered at the Honokaa side abutment consist of 4 to 8 inches of asphaltic concrete over medium dense silty gravel and stiff to hard clayey silt fills extending to depths of about 3.5 to 5 feet below the existing ground surface. The fills were underlain by saprolite and weathered basalt consisting of medium stiff to hard clayey silt and very dense to dense silty gravel extending to depths of about 36 to 43 feet below the existing ground surface. Highly to moderately weathered and soft to medium hard basalt rock was encountered below the saprolite and weathered basalt and extended to depths of about 49 to 50 feet below the existing ground surface. The basalt rock was graded moderately to slightly weathered and medium hard to hard down to the maximum depth explored of about 91 feet below the existing ground surface.

We did not encounter groundwater at the time of our field exploration. However, it should be noted that water levels may vary with stream flow conditions, seasonal rainfall, time of year, and other environmental factors.

Detailed descriptions of our field exploration methodology and the Logs of Borings are presented in Appendix A. Results of the shear wave velocity profiling are presented in Appendix B. Results of the laboratory tests performed on selected soil/rock samples obtained from our field exploration are presented in Appendix C. Photographs of core samples are provided in Appendix D.

## 2.4 Seismic Design Considerations

Based on the AASHTO LRFD Bridge Design Specifications, the project site may be subject to seismic activity, and seismic design considerations will need to be addressed. The following subsections provide discussions on the seismicity, the potential for liquefaction at the project site, and the soil profile for seismic design.

### 2.4.1 Earthquakes and Seismicity

Generally, earthquakes that occur throughout the world are caused by shifts in the tectonic plates. In contrast, earthquake activity in Hawaii is linked primarily 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 they can only be detected by sensitive instruments. 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 that are directly associated with the movement of magma are concentrated beneath the active Kilauea and Mauna Loa Volcanoes on the Island of Hawaii. Because the 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 October 15, 2006 earthquakes. 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.

| 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 |

| DATE | LOCATION | MAGNITUDE |
|---|---|---|
| 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 north and west sides in the past 100 years, including two earthquakes greater than Magnitude 6 in 1929 and 1951. In addition, the October 15, 2006 earthquakes occurred in the northwestern portion of the island. Therefore, it may be concluded that the western side of the Island of Hawaii could experience moderate to severe earthquakes and associated ground shaking, depending on the earthquake's origin.

### 2.4.2 Liquefaction Potential

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:

| FACTORS | LIQUEFACTION SUSCEPTIBILITY |
|---|---|
| Grain Size Distribution | Fine and uniform sands and silts are more susceptible to liquefaction than coarse or well-graded sands. |
| Initial Relative Density | Loose sands and silts are most susceptible to liquefaction. Liquefaction potential is inversely proportional to relative density. |
| Magnitude and Duration of Vibration | Liquefaction potential is directly proportional to the magnitude and duration of the earthquake. |

Based on the subsurface conditions encountered, the phenomenon of soil liquefaction is not a design consideration for this project site. The risk for potential liquefaction is low based on the subsurface conditions encountered (relatively stiff residual soil and saprolite/weathered basalt overlying basalt rock formation within the depths of our borings).

2.4.3 Soil Profile

Seismic shear wave velocity profiling, using seismic piezocone penetration testing (SCPT) equipment, was performed at the project site to more closely analyze the seismic design considerations. Seismic shear wave velocity testing was performed in Boring No. 2 (Hilo side abutment). Based on the seismic shear wave velocity test results, the weighted average shear wave velocity of the materials within the upper 100 feet of the soil profile is on the order of about 1,274 feet per second.

Based on the results of the seismic shear wave velocity test, the project site may be classified from a seismic analysis standpoint as being a “Very Dense Soil and Soft Rock” site corresponding to a Site Class C soil profile type based on AASHTO 2020 LRFD Bridge Design Specifications, 9th Edition.

Based on the AASHTO 2020 LRFD Bridge Design Specifications, the seismic retrofitted bridge structure will need to be designed based on an earthquake return period of 1,000 years. Based on a 1,000-year return period and the anticipated Site Class, the following seismic design parameters were estimated and may be used for the seismic analysis of the bridge structure planned for the project.

| SEISMIC DESIGN PARAMETERS<br>KAHOLO BRIDGE<br>AASHTO 2020 LRFD BRIDGE DESIGN SPECIFICATIONS<br>1,000-YEAR RETURN PERIOD<br>(~7% PROBABILITY OF EXCEEDANCE IN 75 YEARS) | |
|---|---|
| **Parameter** | **Value** |
| Peak Bedrock Acceleration, PBA (Site Class B) | 0.456g |
| Spectral Response Acceleration (Site Class B), $S_S$ | 0.904g |
| Spectral Response Acceleration (Site Class B), $S_1$ | 0.352g |
| Site Class | "C" |
| Site Coefficient, $F_{pga}$ | 1.00 |
| Site Coefficient, $F_a$ | 1.04 |
| Site Coefficient, $F_v$ | 1.45 |
| Design Peak Ground Acceleration, PGA (Site Class C) or $A_s$ | 0.456g |
| Design Spectral Response Acceleration, $S_{DS}$ | 0.939g |
| Design Spectral Response Acceleration, $S_{D1}$ | 0.509g |
| Seismic Design Category | "D" |

END OF SITE CHARACTERIZATION