Assessment of the Vulnerability of Oahu’s Coastal Bridges to Storm Waves and Tsunami Inundation

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And to my parents, thank you for everything.
Abstract

This thesis is part of an extensive research project by the University of Hawaii at Manoa Civil and Environmental Engineering and Ocean Resources Engineering Departments to analyze the vulnerability of Hawaii’s coastal bridges and ports to storm waves and tsunamis. The main focus of this thesis was the structural evaluation of coastal bridges on the island of Oahu, where the bridge capacities and potential demand during hurricanes and tsunami inundation were compared for potentially at-risk bridges. In addition, a method was been developed to facilitate the organization and archiving of survey data.

Of the 26 bridges surveyed, 11 bridges were selected based on the bridge location, number of communities in proximity to the bridge, and the critical service routes that the bridge provided. In addition, the bridges that were determined to be the most exposed to wave forces were favored during the selection process. The chosen bridges were: Kuliouou Stream Bridge, Kahaluu Stream Bridge, New South Punaluu Bridge, Ukoa Pond Bridge, Old Makaha #3A Bridge, New Makaha #3A Bridge, Maipalaoa Bridge, Moanalua Stream Bridge, Kalihi Stream Bridge, and the Nimitz Highway at Aloha Tower Slip Cover #2 and Slip Cover #3.

As-built drawings provided by the Hawaii Department of Transportation were used to calculate bridge weights, buoyancy forces, the lateral and vertical connection capacities, and the negative bending strength of the bridges’ decks and girders. As a preliminary check of the bridges vulnerability to failure, the capacities of each bridge were then compared to estimated 100-year storm wave forces. From this assessment, it was determined the estimated wave forces that will impact the Kahaluu Bridge, Old Makaha #3A Bridge, and the Maipalaoa Bridge are greater than the calculated bridge capacities and therefore all of these bridges are at risk of failing during a 100-year storm event. If submerged by tsunami inundation, the Ukoa Pond Bridge was determined to be at risk of failing due to buoyancy forces. The Kuliouou Stream Bridge, the New Makaha #3A Bridge, the New South Punaluu Bridge, the Moanalua Bridge, the Kalihi Bridge, and the Nimitz Highway Slip Covers #2 and #3 are all determined to have sufficient factors of safety against wave loads and are not at risk of becoming buoyant if submerged by tsunami inundation, which indicates that these bridges will likely survive a 100-year storm event.
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1 Introduction

On March 11, 2011 a 9.0 magnitude earthquake generated a tsunami that struck the east coast of Japan. At the time of writing, it is estimated that the tsunami has caused over $300 billion in damages, which includes damages to more than 400,000 homes and other structures [1]. In December of 2004, a similarly powerful earthquake, centered off the northeast coast of the Indonesian island of Sumatra caused a tsunami that killed 167,000 people, injured in excess of 500,000 and left many more homeless [2].

In 2005, Hurricane Katrina, a category 3 hurricane at landfall, hit southeast Louisiana, Mississippi and Alabama, flooding eighty percent of New Orleans and destroyed 100,000 homes [3]. Significant amounts of coastal infrastructure were damaged by inundation from storm surge and increased wave heights.

The aforementioned natural disasters have made it evident that coastal infrastructures are dangerously susceptible to damage by tsunamis and hurricane storm surge and waves, if they have not been designed for the increased loads experienced during such events. In particular, bridges and ports are lifelines and are of vital importance to coastal communities. Ports provide an important means of shipping goods to damaged areas. Also without bridges, the transport of goods to the surrounding communities is hindered. Moreover, the destruction of bridges can cut essential access to the injured that may need emergency medical aid.

The majority of Hawaii’s population resides near the coast due to its mountainous topography. On the most populated island of Oahu, there are over 26 coastal bridges, some of which provide the only vehicle access route to remote communities. Therefore a large hurricane or tsunami could have dire consequences for Hawaii if coastal bridges fail.
1.1 Research Objectives

The objectives of this study were to provide a procedure that facilitates the organization of survey data, determine the structural capacities of Oahu coastal bridges, calculate the buoyancy force acting on each bridge during tsunami inundation, and compare bridge capacities to storm wave loads calculated with established wave estimation methods.

1.2 Overview

This thesis is part of an extensive research project by the University of Hawaii at Manoa Civil and Environmental Engineering and Ocean Resources Engineering Departments to analyze the vulnerability of Hawaii’s coastal bridges and ports to storm surge and tsunamis. The main objectives of this project are

1) Survey the coastal bridges and commercial ports on the Islands of Oahu, Maui, Kauai, Hawaii, and Molokai to identify their exposure to inundation;

2) Determine design flow parameters, such as water depth, and fluid loads for at-risk facilities;

3) For bridges, evaluate the bridge designs for their resistance to the fluid loads, and recommend potential retrofit as needed;

4) For ports, provide risk assessment of debris (ships and containers) based on the fluid studies;

5) Document the methodology used so that it can be applied to other locations in the future.
The main focus of this thesis is the structural evaluation of coastal bridges on the island of Oahu, where the bridge capacities and potential demand during hurricanes and tsunami inundation are compared for potentially at-risk bridges. In addition, a method has been developed to organize survey data.

Eleven bridges around the island of Oahu have been surveyed and analyzed through the course of this study. The main criteria for selection of the bridges were: the bridge location, number of communities in proximity to the bridge, and the critical service routes that the bridge provided. In addition, the bridges that were determined to be the most exposed to wave forces were favored during the selection process. Based on these criteria, the chosen bridges were: Kuliouou Stream Bridge, Kahaluu Stream Bridge, New South Punaluu Bridge, Ukoa Pond Bridge, Old Makaha #3A Bridge, New Makaha #3A Bridge, Maipalaoa (Maili Channel) Bridge, Moanalua Stream Bridge, Kalihi Stream Bridge, Nimitz Highway at Aloha Tower Slip Cover #2 and Nimitz Highway at Aloha Tower Slip Cover #3.

This study has utilized structural capacity computational methods developed by the American Institute of Steel Construction [4], the Precast/Prestressed Concrete Institute [5], and the American Concrete Institute [6]. The hydrodynamic wave forces have been calculated using the methods developed by Douglass et al. [10], McPherson [11], and AASHTO 2008 Guide Specifications for Bridges Vulnerable to Coastal Storms [9].

Chapter 2 presents a review of available literature on storm and wave loads on bridges and other coastal structures. Chapter 3 presents the organization and web-based archival and documentation procedure for the survey data. To determine if a particular bridge will survive a storm or tsunami inundation event, the lateral and vertical connection capacities and the negative bending strength of the bridges’ decks and girders are calculated in Chapter 4. The capacities of each bridge are then compared against estimated storm wave forces calculated in Chapter 5. Chapter 6 presents conclusions drawn regarding the bridges’ survivability during a storm or tsunami event.
2 Literature Review

Existing literature on post-disaster surveys, reports regarding failure mechanisms of coastal bridges and methods used to estimate wave loads on bridge structures have been investigated. It was hoped that the information gathered from these reports would aid in understanding the behavior of waves and how they affect coastal bridges. In addition, reports analyzing the structural capacity of bridge structures during storm events have been examined. The procedures, calculations, and checks developed in these reports have been used as a guideline for this study, and have been applied to the coastal bridges around the island of Oahu to estimate a bridge’s vulnerability to storm and tsunami wave forces.

2.1 Robertson, et al. (2007): Coastal Bridge Performance during Hurricane Katrina [7]

Robertson, et al. [7], in a post-disaster survey, investigated the performance of bridges along the coast of the Gulf of Mexico during Hurricane Katrina. It was found that the main causes of damage to coastal infrastructures were a result of inundation due to storm surge and wave action. Inundation caused bridges to become submerged, resulting in an upward hydrostatic buoyancy force. Wave action caused both a hydrodynamic uplift and lateral load on the bridges. In the cases of the US90 Bridge over Biloxi Bay and the US90 Bridge approaching Pass Christian, air filled the voids under the bridge decks as the water levels rose. It was calculated that the volume of air decreased slightly due to the water head acting on the trapped air. The trapped air caused a greater volume of water to be displaced, increasing the hydrostatic uplift force experienced during inundation.

Because it is a low seismic zone, the Louisiana bridges were not designed to resist uplift forces and as a result did not have vertical restraints. In order to resist
lateral forces, many of the bridges relied on small connections and gravity load induced friction. However, once the bridges were subject to storm surge and wave loading, the bearing pads the bridges rested on provided little resistance to lateral movement.

The I-10 Onramp in Mobile, Alabama, was secured with steel angle restraints on either side of each exterior girder. However, the connections failed due to failure of anchor bolts, spalling of concrete around the anchor bolts, and poor construction. It was observed that the piles supporting a cast-in-place damaged section suffered no visible damage.

The only bridge reported to remain mostly intact was the Railroad Bridge over Biloxi Bay. The bridge deck had a small width and closely spaced girders. The smaller width of the bridge minimized the area on which wave loading could act, which reduced the hydrodynamic uplift force. The closely spaced girders minimized the amount of air trapped under the bridge, which reduced the hydrostatic uplift force. In addition, the bridge was built with large concrete shear keys to prevent lateral movement.

Based on the post-disaster survey of Hurricane Katrina and the Indian Ocean Tsunami, the authors recommended that low level bridges in danger of inundation should be restrained against uplift and be outfitted with shear keys in order to resist lateral forces. In addition, foundation pile designs must be reviewed in order to verify the adequacy of the bridge foundation to resist the hydrodynamic and hydrostatic uplift forces.

2.2 Hayes (2008): Assessing the Vulnerability of Delaware’s Coastal Bridges to Hurricane Forces [8]

Hayes [8] analyzed the vulnerability of Delaware’s coastal bridges in accordance with the American Association of State Highway and Transportation Officials (AASHTO) Guide Specifications for Bridges Vulnerable to Coastal Storms [9]. Three Delaware
bridges were selected based on the bridge’s deck clearance above water, proximity to the coastline, proximity to wave forming area, and bridge structure type.

The author performed a level I analysis on each of the three bridges. If the wave heights were found to be high enough to impact the bridge, a more detailed analysis would have been undertaken to calculate wave force magnitudes.

According to the AASHTO specifications [9], a level I analysis is the simplest and most conservative method. The method requires wind speed, surge height, local wind speed set-up, astronomical tides, water current speeds, bridge elevation, water depth at bridge location and fetch angle/lengths. The majority of input data used in Hayes’ calculations were based on the 100-year storm criteria.

The author found that during a 100-year storm, storm waves did not impact any of the bridge decks. The author made no significant recommendations. However, a similar approach to the one observed in Hayes’ report may be taken to determine the wave forces on Hawaii’s coastal bridges using the AASHTO Guide Specifications for Bridges Vulnerable to Coastal Storms.

2.3 Douglass et al. (2006): Wave Forces on Bridge Decks [10]

Douglass et al. [10], in a report prepared for the U.S. Department of Transportation, estimated the forces generated by storm waves and verified the damage mechanisms of coastal bridges during storms, by utilizing a combination of laboratory testing, post-storm bridge inspections, numerical models approximating wave/surge conditions during storms and existing methods for estimating wave loads.

The authors found wave loads to be the main source of bridge failure. Depending on the height of the waves and elevation of the bridge deck, the waves exhibited both an uplift and lateral force. The horizontal and vertical force components of the waves were enough to overcome the connections and self weight of the bridges. The repeated wave impacts caused the bridge decks to progressively slide, “bump,” or even “hop”
across the piles of the bridge until the decks flipped or slid off. As a secondary effect, the reduction of bridge self weight, due to buoyancy forces, was also found to be a contributing factor to bridge failure.

To estimate the magnitude of force produced by a wave impacting a bridge structure, Douglass assumed that wave forces are linearly proportional to the equivalent hydrostatic pressure load that an unbroken wave would impart on a bridge if there were air on the other side of the structure. The method requires the bridge deck cross section, bridge deck elevation and estimates of storm surge elevation and wave heights. The forces are given by

\[ F_v = c_{v-va} \times F_v^* \]  
\[ F_h = [1 + C_r \times (N - 1)] \times c_{h-va} \times F_h^* \]  
\[ F_v^* = \gamma \times (\Delta z_v) \times A_v \]  
\[ F_h^* = \gamma \times (\Delta z_h) \times A_h \]

where

\[ F_v \] = vertical wave-induced load [lbs]  
\[ F_h \] = horizontal wave-induced load [lbs]  
\[ F_v^* \] = a “reference” vertical load [lbs]  
\[ F_h^* \] = a “reference” horizontal load [lbs]  
\[ c_{v-va} \] = an empirical coefficient for the vertical “varying” load  
(recommended value is \( c_{v-va} = 1 \) for non conservative, \( 2 \) for conservative design)  
\[ c_{h-va} \] = an empirical coefficient for the horizontal “varying” load  
(recommended value is \( c_{h-va} = 1 \) for non conservative, \( 2 \) for conservative design)
$C_r = \text{a reduction coefficient for reduced horizontal load on the internal girders (recommended value is } C_r = 0.4)$

$N = \text{the number of girders supporting the bridge span deck}$

$\gamma = \text{unit weight of salt water [64 lb/ft}^3\text{]}$

$\Delta z_p = \text{difference between the elevation of the crest of the maximum wave and the elevation of the underside of the bridge deck (magnitude of inundation) [ft]}$

$A_v = \text{the area of the bridge contributing to vertical uplift, i.e. the projection of the bridge deck onto the horizontal plane [ft}^2\text{]}$

$\Delta z_h = \text{difference between the elevation of the crest of the maximum wave and the elevation of the centroid of } A_h \text{ [ft]}$

$A_h = \text{the area of the projection of the bridge deck onto the vertical plane [ft}^2\text{]}$

$\eta_{max} = 1.3H_s \text{ (where the maximum wave above the storm surge elevation can be no more than 0.8ds) [ft]}$

$H_s = \text{significant wave height [ft]}$

$ds = \text{still water level (including storm surge) [ft]}$
According to Douglass, his method calculates wave forces at the most critical condition, where the storm surge elevation is roughly near the bridge deck. The author makes note that the developed method is not conservative and a factor of safety of “2” should be used during calculations. However, the calculated wave forces continue to increase as inundation levels increase, but in reality the forces should decrease after some inundation depth. The method also makes the assumption that wave forces act through the centroid of the bridge cross section, which may not always be the case.

As test cases, Douglass’ method was applied to the I-10 on ramp near Mobile, Alabama, during Hurricane Katrina, the I-10 Bridge across Escambia Bay, Florida, during Hurricane Ivan and the US 90 bridge spanning Biloxi Bay, Mississippi during Hurricane Katrina [10].
The observed damage to the I-10 on ramp near Mobile, Alabama included northward movement of the lowest five simply supported spans. Post-disaster surveys indicated the failures to be caused by concrete breaking around bolt connections. The estimated resistance provided by the bolt connections was calculated to range from 200 - 400 kips. Wave forces were then computed for the five displaced decks and an additional non displaced deck. Based on Douglass’ equations, the displaced span having the lowest elevation experienced a maximum wave force of approximately 980 kips and the highest elevated displaced span experienced a maximum wave force of approximately 400 kips. The calculated forces both exceeded the capacity of the bolt connections, which corresponded to the observed displacement of the deck spans during Hurricane Katrina. The non displaced deck was subjected to a load of approximately 170 kips less than the maximum capacity of the bolt connections, which corresponded to the non-failure documented in the post-disaster survey.

Similar correlations between Douglass’ methodology and post-survey observations were found for both the I-10 Bridge across Escambia Bay and the US 90 bridge spanning Biloxi Bay. Therefore, Douglass’ method is a viable preliminary guide to estimating storm surge wave forces on bridge decks that may be used to assist in the design and analysis of coastal bridges.


McPherson [11], in a report continuing Douglass’ [10] research, used a large 3 dimensional testing basin to determine the validity of existing wave estimation methods. In the testing basin, McPherson varied wave conditions and water depths. The associated wave forces on a 1:20 scale bridge model and flat plate model were then measured. Subsequently, the recorded experimental wave forces were compared to wave estimation methods. The methods analyzed included Kaplan et al. (1995), Bea et al. (2001), McConnell et al. (2004), and Douglass et al. (2006).
McPherson found that none of the analyzed methods was able to properly estimate wave forces for all wave conditions. In addition, the author discovered that McConnell [12] and Douglass [10] overestimated the wave forces for a still water level at or above the bridge deck. However, overall, it was determined that Douglass’ method resulted in the closest estimated wave forces when compared to the experimental data.

Using Douglass’s method as a template, McPherson developed an improved method to estimate wave forces on a bridge structure. His method is as follows.

The vertical wave force is estimated by

\[ F_{\text{Total}} = F_{\text{Hydrostatic}} + F_{\text{Bridge}} + F_{\text{AirEntrainment}} \] (2.4-1)

\[ F_{\text{Hydrostatic}} = \gamma \delta_x A - F_w \] (2.4-2)

\[ F_{\text{Bridge}} = \gamma Vol_{\text{Bridge}} \] (2.4-3)

\[ F_{\text{AirEntrainment}} = (n - 1)0.5\gamma \delta_g A_g \] (2.4-4)

if \( h \leq h_{\text{model}} \),

\[ F_w = \frac{1}{2} \gamma \delta A \] (2.4-5)

and if \( h > h_{\text{model}} \),

\[ F_w = \frac{1}{2} \gamma \delta A + \gamma (h - h_{\text{model}})A \] (2.4-6)

The horizontal wave force is estimated by

\[ F_{\text{Total}} = F_{\text{Hydrostatic Front}} - F_{\text{Hydrostatic Back}} \] (2.4-7)

if \( \eta_{\text{max}} < h_{\text{deck}} \),

\[ F_{\text{Hydrostatic Front}} = 0.5 * (\eta_{\text{max}} + h - h_{\text{girders}})H_{\text{bridge}} L_{\text{bridge}} \gamma \] (2.4-8)
and if \( \eta_{\text{max}} > h_{\text{deck}} \),

\[
F_{\text{Hydrostatic, Front}} = 0.5 \ast [(\eta_{\text{max}} + h - h_{\text{girders}}) + (\eta_{\text{max}} - h_{\text{deck}})]L_{\text{bridge}}Y \quad (2.4-9)
\]

if \( SWL < h_{\text{girders}} \),

\[
F_{\text{Hydrostatic, back}} = 0 \quad (2.4-10)
\]

and if \( SWL > h_{\text{girders}} \),

\[
F_{\text{Hydrostatic, back}} = 0.5(h - h_{\text{girder}})^2L_{\text{bridge}}Y \quad (2.4-11)
\]

In the above equations

- \( \gamma \) = unit weight of salt water [64 lb/ft\(^3\)]
- \( \delta_x \) = distance from the top of the bridge deck to the top of the wave [ft]
- \( \delta_G \) = height of the bridge girders [ft]
- \( \delta \) = height of wave overtopping the bridge deck [ft]
- \( A \) = area of bridge impacted by vertical wave force [ft\(^2\)]
- \( A_G \) = cross sectional area of trapped air between girders [ft\(^2\)]
- \( n \) = number of girders
- \( h_{\text{model}} \) = distance from ground elevation to top of deck [ft]
- \( h \) = height from the ground elevation to the top of the still water level [ft]
- \( \eta_{\text{max}} \) = height of wave above the still water level [ft]
\[ h_{girders} = \text{height from the ground elevation to the bottom of the bridge girders [ft]} \]

\[ H_{bridge} = \text{height of bridge impacted by lateral wave forces [ft]} \]

\[ L_{bridge} = \text{length of bridge impacted by lateral wave forces [ft]} \]

\[ SWL = \text{still water level including storm surge [ft]} \]

**Figure 2-2:** McPherson Vertical Wave Estimation Method Diagram [11]

**Figure 2-3:** McPherson Horizontal Wave Estimation Method Diagram [11]
By including the upward buoyancy force and the downward force from overtopping waves, the author was able to recreate the non-linear variation in the vertical force observed in his experimental data. McPherson was also able to adjust Douglass’ overestimation of the horizontal forces by including the opposing hydrostatic force on the trailing edge of the bridge structure.


Boon-intra [13] synthesized relevant existing literature and numerical models to develop a method that estimates tsunami impact forces on bridge superstructures. Tsunami forces were formulated by combining equations used to calculate hydrostatic and hydrodynamic water pressure on a structure. The developed equations are as follows.

The horizontal tsunami force is estimated by

\[ F_H = F_h + F_d = \left[ 1 + C_r * (N - 1) \right] * \gamma * (\Delta h_{max}) * A_h + 0.5 * C_d * \rho * b(\Delta h * u^2)_{max} \] (2.5-1)

in which

\[ F_h = \text{hydrostatic horizontal force [lbs]} \]
\[ F_d = \text{hydrodynamic horizontal force [lbs]} \]
\[ C_r = \text{reduction coefficient for internal girder pressure (0.4 recommended value)} \]
\[ N = \text{number of girders supporting deck} \]
\[ C_d = \text{empirical drag coefficient (1.0 for deck-girder bridge type)} \]
\[ \rho = \text{seawater mass density [slug/ft}^3\text{]} \]
\[ (\Delta h * u^2)_{max} = \text{maximum flux momentum [ft}^3/\text{sec}^2\text{]} \]
The vertical tsunami force is estimated by

\[ F_v = \left[ \gamma \left( \Delta h_{max} \right) + \frac{1}{2} \rho u_{x,max}^2 \right] A_v \quad (2.5-2) \]

in which

- \( u_{x,max} \) = adjusted horizontal wave velocity = 3.5 \( u_{x,\text{max}}^* \) [ft/sec]
- \( u_{x,\text{max}}^* \) = horizontal wave velocity [ft/sec]

His method was validated by comparing values computed using the above equations to a finite element, two dimensional, compatible fluid dynamics model used to estimate tsunami impacts on full scale bridges.

The author notes that the horizontal force equation may underestimate or overestimate values because of the empirical coefficients, which are based on averaged data. For small vertical forces, the vertical equation was found to overestimate the potential tsunami force. The opposite was found for large vertical forces (i.e. underestimation of forces for large values). In addition, the equations are limited to deck-girder bridge types. The author recommends adding a factor of safety to accommodate for any uncertain forces that may develop during a tsunami.

Nonetheless, overall good agreement was found between the peak forces generated by the equations and the numerical model. Therefore, Boon-intra’s method is a feasible simplified guide to estimating tsunami wave forces on bridge decks that may be used to assist in the design and analysis of coastal bridges.

During numerical modeling, Boon-intra also observed bridge railings to be a significant factor in the magnitude of horizontal tsunami wave forces experienced by a bridge. The author found a 20% maximum increase in horizontal wave force for bridges with rigid railings as opposed to bridges without. The increase in force was likely caused by the cross sectional area the railings added to the bridges, which allowed a greater area to be impacted by a tsunami wave.
2.6 Bea et al. (1999): Wave Forces on Decks of Offshore Platforms [14]

Bea et al. [14], in a report seeking to refine the criteria used to estimate wave crest forces on lower decks of offshore platforms, analyzed the performance of oil platforms in the Gulf of Mexico during hurricane wave loading. Upon reviewing Hurricane Andrew, the authors found that wave crest impacts are isolated and an entire deck is not completely inundated by long-crested waves. This observation was reinforced by hurricane photographs, which depicted short-crested multidirectional waves impacting decks. The performance of three platforms in South Pass during Hurricane Camille and the failure of a UNOCAL platform during Hurricane Hilda provided Bea with cases to verify his proposed wave-in-deck horizontal force guidelines.

The modification proposed by Bea to the American Petroleum Institute (API) deck wave force guidelines was a culmination of laboratory data that measured wave forces on decks, columns and vertical walls. The total force ($F_{tw}$) can be estimated with the following equation

$$F_{tw} = F_b + F_s + F_d + F_l + F_i$$

(2.6-1)

in which

- $F_{tw}$ = total force
- $F_b$ = vertical buoyancy force
- $F_s$ = horizontal slamming force $= 0.5 \cdot \rho \cdot C_s \cdot A \cdot u^2$ (2.6-2)
- $F_d$ = horizontal drag force $= 0.5 \cdot \rho \cdot C_d \cdot A \cdot u^2$ (2.6-3)
- $F_l$ = vertical lift force $= 0.5 \cdot \rho \cdot C_l \cdot A \cdot u^2$ (2.6-4)
- $F_i$ = acceleration-dependent inertia force $= \rho \cdot C_m \cdot V \cdot \alpha$ (2.6-5)
- $C_s$ = slamming coefficient (range: $\pi - 2\pi$)
- $\rho$ = mass density of sea water [slugs/ft$^3$]
\[ A \] = vertical deck area subjected to wave crest \([\text{ft}^2]\)

\[ u \] = horizontal fluid velocity of the wave crest \([\text{ft/sec}]\)

\[ C_m \] = inertia coefficient

\[ V \] = volume of the deck inundated \([\text{ft}^3]\)

\[ a \] = water acceleration \([\text{ft/sec}^2]\)

The effective slamming force equation is modified for impact durations \((0.01 – 0.1 \text{ seconds})\) that are short in relation to the natural period of the decks. The effective force can be calculated using the following equation

\[ F_s' = F_v * F_s \]  \(\text{(2.6-6)}\)

in which

\[ F_v \] = dynamic loading factor = \[2 * \pi * \alpha * \left(\frac{t_d}{T_n}\right)\]

\[ \alpha \] = 0.5 (triangular loading) or \[\frac{2}{\pi}\] (half sine loading)

\[ t_d \] = impact duration \([\text{sec}]\)

\[ T_n \] = natural period of deck \([\text{sec}]\)

To verify his modifications to the API procedure, Bea used the ULSLEA (ultimate limit-state limit equilibrium analysis) computer program and data from four hurricanes and eight platforms. The details of the computer program are not pertinent to this thesis. However, the modified API procedure was able to produce results that closely match observed damage sustained by platforms during hurricanes. Previously, the API method was conservative and predicted a structure would fail even if only minor damage was documented.

Robertson et al. [15] presented a more in depth analysis of the failure mechanisms of engineered structures during Hurricane Katrina. Again the main causes of damage to coastal bridges were reported to be caused by hydrostatic uplift, hydrodynamic uplift/lateral loading, debris impact, and scour. The damage to structures during a hurricane and a tsunami has been found to be similar, making the methods described in this report, to some extent, applicable to both natural disasters.

To estimate the forces experienced during a hurricane, Bea et al. [14] proposed an equation that included the vertical buoyancy force, horizontal slamming force, horizontal hydrodynamic force, vertical hydrodynamic uplift force, and acceleration-dependent inertia force. However, from conclusions drawn from Douglass’ [10] report on bridge performance and observations made in Robertson et al. [15], the equation can be simplified to only include the horizontal hydrodynamic force, the vertical hydrodynamic uplift force and buoyancy.

The hydrodynamic horizontal wave forces on the Louisiana bridges were estimated to range from 2000 lb/ft to 4500 lb/ft. The vertical hydrodynamic wave forces were calculated to range from 3500 lb/ft to 10600 lb/ft. The vertical hydrodynamic forces on the majority of the bridges exceeded the bridge self weights by more than 30%. To withstand the hydrodynamic forces experienced during a hurricane, the authors suggested that low level bridges at risk of storm surge inundation be restrained against uplift and be provided with shear keys to resist lateral movements.

The vertical buoyancy force was reported to be a combination of the reduced self weight of concrete and the increased volume of displaced water caused by trapped air under the bridge decks. The volume of entrapped air was compressed due to differences in water pressure heads when the bridge was completely submerged, which reduced the air pocket slightly.
The buoyancy force reduced the residual weight of the US 90 Bridge in Biloxi Bay to 1.21 lb/ft, a 98.7% reduction in self weight. In contrast, the Railroad Bridge over Biloxi Bay was the only reported bridge to have more than 28% of its residual self weight. It was also the only recorded bridge to survive Hurricane Katrina mostly intact. As a mitigation measure, the authors suggested designing low lying bridges with bulkheads that will allow air to escape from below bridge decks, reducing the volume of trapped air.

Of notable significance are the failures of prestressed double tee floor systems used in the construction of parking garages in the Biloxi Gulfport region. Some of the garages were protected from wave action, but were still inundated by storm surge. The double tee geometry lent itself to trapping a large volume of air, resulting in an uplift force much greater than the submerged self weight of the double tee and concrete topping slab. The uplift force caused negative bending to develop, causing compression in the bottom of the tee and tension in the top. In addition, the pre-stressing in the double tees caused an upward bend, normally used to negate the effects of gravity loads. The combination of the buoyancy uplift force and the prestressing effect caused the double tees to fail. To avoid the failure mechanism, it was recommended that prestressed double tee systems, flat slab, and other concrete systems be designed to withstand the negative bending and shear caused by hydrodynamic and hydrostatic forces.

2.8 Robertson et al.: Case Study of Concrete Bridge Subjected to Hurricane Storm Surge and Wave Action [16]

Robertson et al. [16] performed an in depth structural analysis of an onramp to the freeway in Mobile, Alabama. The onramp deck sections were secured with bolted angle connections to the supporting bents. The connections failed during Hurricane Katrina, which allowed the five lowest deck spans to move northward. The spans did not
completely fall off their supports due to a wedging effect caused by the curved geometry of the bridge.

Partial depth bridging was used to connect the bridge girders together, allowing air pockets to form under the deck of the bridge. The volume of air was likely decreased due to holes through the bridging elements. The two exterior girders of each span were connected to a supporting bent by the 10.5 inch long, 6 inch x 8 inch x 1 inch thick galvanized steel angle. The angles were secured to the concrete bents vertically using 1.125 inch diameter bolts on either side of the girders and horizontally into the bulb of the girders using 0.875 inch diameter sleeve anchors.

Hurricane wave forces caused shear loads to develop in both the vertical and horizontal bolts of the connections. The shear loads caused the surrounding concrete to fail and spall off, which led to bolt pull outs, bolt bending and bolt rupture failures. A few angle connections were crushed by displaced decks. Due to the misalignment of bolt inserts, some connections were completely missing anchor bolts prior to hurricane damage. Poor field modifications to some connections resulted in enlarged bolt holes, allowing bolts to pull out without resistance.

The capacities of the observed failure mechanisms were then compared to forces computed using the equations developed by Douglass et al. [10]. The hurricane’s hydrodynamic lateral and vertical wave loads were estimated to be 183 kips and 388 kips, respectively.

Horizontal forces on each deck span were resisted by tension pull out of four groups of two sleeve anchors, vertical concrete spalling of two groups of anchor bolts, and vertical bending of two connections. The total horizontal resistance provided was computed to be 217 kips, which is only 34 kips above the estimated horizontal hurricane wave force. Therefore it was likely that repeated wave forces would cause progressive damage to the connections, resulting in lateral movement of the decks.
Vertical hydrodynamic wave forces on each deck span were resisted by the deck self weight and eight pairs of sleeve anchors. The total vertical resistance provided was calculated to be 400 kips. The vertical and horizontal resistances were only 3% and 19% greater than their respective hurricane wave force counterparts; as a consequence nearly no factor of safety was present to prevent failure.

During the event of complete submergence of a bridge, Douglass’ equations are no longer valid. Instead, hydrostatic buoyancy forces became more significant. The buoyancy force was computed by determining the volume of concrete and entrapped air between the girders. This volume was then multiplied by the specific weight of seawater (i.e. 64 lb/ft$^3$). The buoyancy value was then subtracted from the self weight of the bridge deck resulting in the residual self weight. The residual self weight of a typical I-10 bridge span was computed to be 22.3% of its original self weight. Thus, the authors concluded that buoyancy alone would not have dislodged the bridge deck.

The authors concluded that the connections on the I-10 Freeway were not adequate to resist the hydrodynamic lateral and vertical wave loads generated by Hurricane Katrina. The authors provided connection revisions that could strengthen the connections against the failure mechanism observed during their inspections.

2.9 Lehrman (2010): Laboratory Performance of Highway Bridge Girder Anchorages under Hurricane Induced Wave Loading [17]

Lehrman [17] focused his research on the failure mechanism of connections used to connect bridge superstructures to substructures. Full size AASHTO type III prestressed concrete girders were constructed and fitted with three different types of common anchors. The anchors selected were threaded insert/clip bolt anchorage (CB), headed stud anchorage (HS) and through bolt anchorage (TB).

The CB anchor fixed the bridge girder to a pile using an 8x6x1 inch steel angle. The girder bulbs were prefabricated with 7/8 inch diameter, 3 inch long threaded
inserts. A325 bolts were placed in the inserts through the vertical leg of the angle. The A325 bolts were then secured with 7/8 inch cap screws. Vertically the angles were secured to the pile using 1.25 inch diameter swedge bolts.

The HS anchor connected the girder of the bridge to a bearing plate by welding four-5/8 inch diameter, 6 inch long anchor studs to the plate. Note the studs were embedded in the bulb of the girder. The bearing plate was then secured to the pile using two-1 inch diameter, 15 inch long A307 bolts.

The TB anchor utilized threaded inserts that ran through the entire bulb of the girder. One inch diameter bolts were passed through the inserts and connected two 8x6x1 inch angles on either side of the girder. The angles were then connected vertically to the pile using 1.25 inch diameter, 11 inch long swedge bolts.

To test the capacity of the anchors and to observe the failure mechanism of the girders, the author loaded each of the specimens with an equivalent bridge deck weight and attached actuators. To simulate vertical wave forces a vertical actuator was attached to the girder. To simulate horizontal wave forces an actuator was attached to the face of the girder. Each anchor type was tested with four loading patterns, which were: vertical force only, horizontal force only, both vertical and horizontal forces, and real-time dynamic loading of horizontal and vertical components.

During the testing, concrete spalling was a common occurrence. The spalling led to the exposure of prestressing strands. Out of the three connection types, the author noted that the HS had the most robust performance. In addition, the failure of the HS connector was tied to the steel properties of the studs, making it easier to predict the failure mechanism and load.

The TB connector failed at higher loads than the CB; however the TB connector caused more damage to the concrete girder. In some tests, the TB caused the entire bottom of the concrete girder to break off, exposing much of the prestressing. Both anchor types experienced cracking throughout and strand slip.
Through the study, the author found that if there is trapped air under a bridge deck, then all three anchor types (HS, CB, and TB) do not have enough capacity to resist wave loads estimated by the AASHTO Guide Specifications [9].
3 Organization of Survey Data

3.1 Project Website

Bridges around the island of Oahu were surveyed to assess possible inundation and potential damage. The surveys consisted of on-site inspections, review of bathymetry, topography, and inundation studies for each of the bridge sites. To document the site investigations, hundreds of photographs were taken. These pictures depicted detailed bridge geometry, construction, and surrounding land and water features. During the survey of Oahu, 26 bridges were examined. With the large amount of information compiled, there was a need to organize the material and data into a usable and easily accessible form. Ultimately, a web based solution was chosen.

Drupal, an open source management platform, was selected as the content management system for the project website. All aspects of the project were uploaded to the website in order to share information and progress with each of the project’s members. To facilitate management of the project, it was organized into several categories and divisions, which include: Overview, Description, Map of Oahu, Rapid Visual Assessment, Computational Modeling, Literature Database and Team.

The ‘Overview’ serves as the homepage of the website and provides a summary of the project (see Figure 3-1). Furthermore, the ‘Overview’ page introduces the user to the layout of the website, where links are provided both at the top and on the left of each page. The ‘Description’ page presents the project background, the project’s relevance/significance to the state of Hawaii, project objectives, and information regarding the documentation that has been developed during the project.
Coastal Infrastructure Vulnerability to Tsunami and Storm Surge Inundation

Research Project:

Surveys will be carried out on Hawaii coastal bridges and commercial ports on the Islands of Oahu, Maui, Kauai, Hawaii, and Molokai to identify their exposure to inundation and are being evaluated for potential damage. Calculation methods are being developed to determine the wave loads on coastal bridges during storms and tsunami impact.

Recommendations will be prepared based on these surveys and the calculations that will be provided for potential retrofit of structures deemed inadequate. Remedial measures will be suggested where these may reduce the effects of the inundation.

University of Hawaii at Manoa
Civil & Environmental Engineering
and
Ocean and Resources Engineering
Departments

Figure 3-1: Website Homepage
The ‘Map of Oahu’ page was created to streamline the process of finding any bridge at a specific location. This page consists of a large map with color coordinated markers indicating the location of each of the surveyed bridges (see Figure 3-2). A green marker represents a surveyed site with no additional studies planned. A yellow marker indicates a surveyed site that will be structurally analyzed. A red marker indicates a surveyed site that will be hydrodynamically modeled in addition to being structurally evaluated.

When the cursor hovers over a marker, the name of the bridge is displayed (see Figure 3-2). When clicked, the user is linked to a webpage containing survey pictures and bridge information.
Figure 3-2: Map of Oahu
The ‘Rapid Visual Assessment’ is the main organizational portion of the website, and arranges the bridge webpages into a usable catalog. To organize the photographs and information from the site inspections, a separate page was created for each of the surveyed bridges. The front page of this section provides a list of each bridge according to the date of survey (see Figure 3-3).
A list of the bridges is also provided in the left navigation bar (see Figure 3-3). After clicking the ‘Rapid Visual Assessment’ link, the navigation menu is expanded, revealing each of the bridge pages.

The bridges are organized geographically starting with the Alawai Canal Bridge, which is located near the South East coast of Oahu. The bridges are then listed in a counter clockwise order moving around the island. Organizing the bridges in geographical order helps the user achieve a better sense of the bridge locations.

On each of the bridge webpages, a dedicated map is provided showing the location of the site (see Figure 3-4). These dedicated maps were created to allow the user to see the location of the bridge without having to navigate away from the current page.
Figure 3.4: Mini Map Example

Ala Wai Canal Bridge

Survey Date: 11/19/2010
Further Study: None

Location: Rte. 92/Ala Moana Blvd.
Latitude: 21° 17’ 15.99”N
Longitude: 157° 20’ 24.90”W

*Click on the pictures below for a larger image
Each page also contains the bridge name, survey date, information on further studies, the bridge road location, and the bridge global coordinates. Ocean bathymetry diagrams are also provided for the four sites that will be modeled hydrodynamically. As seen in Figure 3-5, another navigation tool bar is provided (circled red), which allows the user to see the bridge map location and navigate to the ‘Map of Oahu’ page. The user is also given the option to move to the previous or next bridge, based on the counter clockwise geographical ordering. Similar links are provided at the bottom of the page. This feature allows the user to access the navigation tools without having to return to the top of the page.

Figure 3-5: Navigation Menu
Thumbnail versions of the survey pictures are provided to reduce the loading time of each page, while still displaying image previews. Each of the thumbnails can be clicked to open a larger full resolution version of the photo (see Figure 3-6). All of the pictures taken during the site inspections are displayed on each page. However, in order to keep each webpage to a manageable size, additional pages were created if there were more than 30 pictures for a particular site.

During the surveys it was important to document the bridge’s surrounding, structure, layout and notable features. Careful attention was paid to the bridge girder seats, as in some situations the bridges were connected to the foundation at these locations. If possible, the undersides of the bridges were photographed. This was done to determine the possibility of air becoming trapped under the bridge during a rise in water level. Bridge abutments, railings, and any bridge damage were also photographed.

*Figure 3-6: Thumbnail Picture Example*
Bridge surroundings were photographed to document water levels, ocean proximity, coastal arrangement, and landscape. Some bridges spanned small rivers or culverts and were a distance away from the coast. In these cases, photographs were taken to record the terrain between the shoreline and these bridges. Any other objects that may influence the bridges or water movements were also noted.

Figure 3-7: Survey Pictures
The remaining pages, ‘Computational Modeling,’ ‘Literature Database,’ and ‘Team,’ provide additional information about the project. The ‘Computational Modeling’ page explains the approach taken and methods used to determine the wave loads on coastal bridges during a natural disaster. The ‘Literature Database’ provides information about ‘Mendeley,’ a reference management application that the project uses to exchange reference material between members. The ‘Team’ page lists the project group members and their contact information.

A project timer is the last feature added to the website. At the bottom of each page the number of days until the project’s completion is displayed. The timer automatically updates every day, which serves as a reminder to the team of the amount of time left in the project.

Figure 3-8: Project Timer
3.2 Organization of Survey Data General Procedure

The following is a generalized procedure that may be followed to organize data for a survey project. This procedure is based on work done and lessons learned during the course of this thesis. Note that this process will not cover all aspects of each and every survey, but it does provide a preliminary guideline.

3.2.1 Pre-Survey

1) Identify the sites to be inspected.
2) Group the sites by geographical location.
3) Create a table with the following information to be filled in during the surveys:
   a) Site Name
   b) Survey Date
   c) Time of Survey
      ▪ This is important when dealing with water tide levels.
   d) Location
   e) Additional Information as needed

3.2.2 During Survey

1) Designate one person to photograph sites.
   ▪ This is done so multiple copies of similar photos are not created.
   ▪ This will also ensure consistent picture labeling notation.
2) Photograph Structure:
   a) Photograph entire front, back, left side, and right side of structure if possible.
      ▪ It may be beneficial to have a project member in each of the photos in order to give a sense of scale (record height of the member in the photo).
3) Photograph key locations:
   a) Structural support system
b) Connections

c) Visible damage

d) Any points of interest

  - Example: *For bridges, photograph underside of the bridge decks to determine potential air pocket locations.*

e) If the structure is to be structurally evaluated:

  - Photograph any form of lateral resistance.
    - Creep Blocks
    - Connections
    - Wing Walls
    - Support tie ins
    - Natural landscape
    - Etc.

  - Photograph any form of vertical resistance.
    - Connections
    - Additional loads placed on the structure
    - Support tie ins
    - Etc.

4) Photograph surrounding area:

   a) Photograph the area to the north, south, east, and west.

   b) Photograph intermediate directions as needed.

5) Photograph influential land marks:

   a) Structures or vital buildings in close proximity.

   b) Natural terrain that may inhibit water forces from impacting the site.

6) Record site data:

   a) Measure as many dimensions of the structure as possible.

      - *If construction drawings are not available, then it is suggested a laser range finder be used to determine overall bridge dimensions.*
b) If water is in close proximity, measure water depths at various locations along/around the site.

c) Measure the vertical distance from the still water level to the bottom of the structure.

d) Record time of measurement.

e) Make note of the condition of the structure.

f) Make note of damaged areas.
   - Spalling
   - Cracked sections

g) Note any potential failure mechanism
   - Broken connections
   - Improperly constructed areas
   - Scouring of foundation
     - *Important for structures near the coast.*

h) If the structure is to be structurally evaluated:
   - Make note of all aspects of the structure that will increase its resistance to lateral displacement, vertical displacement, and overturning.

### 3.2.3 Post Survey Organization (creation of a project website)

1) Create a basic website structure by breaking the project into several sections.

   a) Create a separate webpage for each of the following:
      - Project Description
      - Project Purpose
      - Map of ‘Surveyed Area’
      - Survey Data and Photographs

   b) Upload the survey photos to the server hosting the website.

   c) Create a link menu.
2) Create a separate webpage for each of the investigated sites.
   a) Input the data of the site at the top of the page.
      ▪ Name of site
      ▪ Location (Global coordinates, area, road location)
      ▪ Date and time of survey
      ▪ Additional information recorded
   b) Organize the photographs for the site.
      ▪ Create thumbnails for each photo.
      ▪ *Size the thumbnails appropriately. The thumbnails should not be too small, as the intention is to provide a preview of each picture.*
      ▪ Place the photos in vertical columns.
      ▪ *Keep the amount of photos per page to a maximum of 30 to keep each page a manageable size.*
      ▪ If more photos exist, create additional webpages as needed.
   c) Place each of the newly created webpages under the ‘Survey Data and Photographs’ portion of the website.
   d) Organize each of the webpages in geographical order.
      ▪ Choose a starting location.
      ▪ Organize the sites in either a counter clockwise or clockwise order.
   e) Return to the webpages created for each surveyed location:
      ▪ Create a navigation menu at the top and bottom of each site webpage (see Figure 3-5).
      ▪ Provide a link to the next site in the geographical ordering.
      ▪ Provide a link to the previous site in the geographical ordering.
   f) Create maps:
      ▪ Find a map of the entire area containing all of the surveyed sites.
• Place markers at each of the surveyed locations.
• Using an ‘image map,’ create a link at each of the markers. Link the marker to the appropriate webpage (see Figure 3-2).
• Place the map in the Map of ‘Surveyed Area’ section of the website.
  ▪ Create smaller maps for each of the surveyed sites.
• Create a marker at the surveyed location.
• Add a link to the map in the navigation menu of the appropriate webpage (see Figure 3-5).

  g) Check the website:
  ▪ Check that all links work and that they are linked to the correct webpages.
  ▪ Check that all photos work.

3.3 Summary

The purpose of the Drupal website was to organize collected data, survey information and photographs into a usable, organized form that allows members of the project to easily access the information. This was accomplished by organizing webpages of each site by geographical location. Links were provided to allow users to navigate between each of the sites as if they were traveling around the island of Oahu in a counterclockwise direction. Menus, navigational links, maps, and additional site information are provided to create a functional, easy to use website. By generalizing the process, the same methodology described in section 3.2, can be applied to other surveys.
4 Bridge Structural Analysis

4.1 Bridge Selection

Of the original 26 bridges surveyed, eleven were selected to be structurally evaluated. The bridges selected were the Makaha #3A Bridge, the New Makaha #3A Bridge, the Maipalaoa Bridge, the Kahaluu Bridge, the North Punaluu Bridge, Nimitz Highway at Aloha Tower, the Kuliouou Stream Bridge, the Ukoa Pond Bridge, the New South Punaluu Bridge, the Moanalua Bridge, and the Kalihi Bridge.

The main criteria for selection were: the bridge location, number of communities in proximity to the bridge, and the critical service routes that the bridge provided. In addition, the bridges that were determined to be the most exposed to wave forces were favored during the selection process.

The New Makaha #3A Bridge and the New South Punaluu Bridge were selected to determine the bridges’ vulnerability to storm and tsunami wave forces before the bridges are built. The Moanalua Bridge and the Kalihi Bridge were selected to evaluate bridge connections found during the site survey.
4.2 Calculation of Buoyancy Force

4.2.1 Approach

During a tsunami or storm event, an abnormal rise in water level occurs. During a storm event, water levels rise due to low pressure and water being pushed toward the shore by strong winds. This phenomenon is known as “storm surge” [18]. Storm surge and tsunami inundation can cause low lying areas to become inundated, which may potentially submerge coastal bridges. If submerged, a bridge’s stabilizing self weight is reduced due to an upward hydrostatic buoyancy force [10]. Buoyancy force is a function of the volume of water displaced by the bridge and air trapped under the bridge deck. The upward buoyancy force is determined from the total submerged volume multiplied by the specific weight of sea water (64 lbs/ ft³).

For all calculations, the bridges were assumed to be submerged to the top of the bridge deck. The trapped air was assumed to fill the entire volume between the bridge girders to the bottom of the lowest bridge diaphragm as shown in Figure 4-1. This worst case scenario results in the largest buoyancy force possible, and is therefore a conservative calculation when designing for failure.

Due to water pressure, the volume of entrapped air is compressed once the bridge is submerged [15]. Based on hydrostatics and the ideal gas law, the volume of compressed air can be computed from the following equations

\[ P2 = P1 + h \times \left(\frac{\gamma}{144}\right) \]  (4.2-1)

\[ A2 = \frac{P1 \times A1}{P2} \]  (4.2-2)

in which,

\[ h \quad = \quad \text{height from bottom of air pockets to water elevation [ft]} \]

\[ \gamma \quad = \quad \text{specific weight of water [64 lbs/ ft}^3] \]

\[ P1 \quad = \quad \text{atmospheric pressure at the water surface [14.7 psi]} \]
The bridge and air pocket volumes were computed from as-built drawings provided by the Hawaii Department of Transportation (HDOT).

To calculate the self-weight of each bridge, the volume of the bridge is multiplied by the specific weight of reinforced concrete (150 lbs/ft³). Any additional loads, such as bridge railings and road pavement, are added to the reinforced concrete weight, resulting in the total self-weight of the bridge superstructure.

If the buoyancy force is found to be greater than the self weight of the bridge then the bridge is considered buoyant. However, assuming that air fills the entire void under the bridge deck may not always be applicable. As water levels rise due to storm surge it is highly possible that a quantity of air will escape from under the bridge deck.

Therefore, if it is found that a particular bridge becomes buoyant with an assumed 100% air pocket, then a less conservative air volume of 50% is used [11]. The buoyancy force acting on the bridge is recalculated with only half of the previous air pocket volume.
pocket. If the bridge is still determined to be buoyant with air filling 50% of the volume under the bridge deck, then the bridge is deemed buoyant and therefore subject to possible failure.

The results of the buoyancy force calculations for each bridge can be found in the following section.

4.2.2 Buoyancy Force Calculation Results

The dimensions for each bridge are given in Table 4.2-1. Table 4.2-2 summarizes the calculated bridge superstructure volume, air volume, and submerged volume of each bridge. Table 4.2-3 summarizes the computed buoyancy force acting on each bridge with air filling 100% of the volume under the bridge. Table 4.2-4 summarizes the recalculated buoyancy force with a less conservative 50% air pocket.
<table>
<thead>
<tr>
<th>Bridge Dimensions:</th>
<th>Number of Girders</th>
<th>Bridge Width (ft)</th>
<th>Bridge Length (ft)</th>
<th>Girder Height (ft)</th>
<th>Deck Thickness (ft)</th>
<th>Railing Height (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kuliouou Stream Bridge:</td>
<td>12</td>
<td>68.75</td>
<td>48.40</td>
<td>3.00</td>
<td>0.67</td>
<td>2.71</td>
</tr>
<tr>
<td>Kahaluu Stream Bridge:</td>
<td>8</td>
<td>46.00</td>
<td>318.00</td>
<td>4.50</td>
<td>0.50</td>
<td>2.13</td>
</tr>
<tr>
<td>New South Punaluu Bridge:</td>
<td>30</td>
<td>50.00</td>
<td>170.00</td>
<td>1.75</td>
<td>0.88</td>
<td>3.00</td>
</tr>
<tr>
<td>Ukoa Pond Bridge:</td>
<td>7</td>
<td>49.00</td>
<td>270.00</td>
<td>4.83</td>
<td>0.54</td>
<td>3.50</td>
</tr>
<tr>
<td>New Makaha Stream #3A Bridge:</td>
<td>1</td>
<td>46.83</td>
<td>70.00</td>
<td>2.33</td>
<td>0.46</td>
<td>1.17</td>
</tr>
<tr>
<td>Old Makaha Stream #3A Bridge:</td>
<td>12</td>
<td>32.83</td>
<td>78.83</td>
<td>1.50</td>
<td>0.50</td>
<td>0.00</td>
</tr>
<tr>
<td>Maipalaoa Bridge:</td>
<td>16</td>
<td>64.33</td>
<td>100.67</td>
<td>3.00</td>
<td>0.50</td>
<td>2.00</td>
</tr>
<tr>
<td>Moanalua Bridge:</td>
<td>9</td>
<td>64.33</td>
<td>215.00</td>
<td>1.83</td>
<td>0.67</td>
<td>3.73</td>
</tr>
<tr>
<td>Kalihi Bridge:</td>
<td>13</td>
<td>88.33</td>
<td>188.00</td>
<td>1.83</td>
<td>0.67</td>
<td>3.73</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2:</td>
<td>11</td>
<td>Variable</td>
<td>Variable</td>
<td>2.50</td>
<td>1.29</td>
<td>1.67</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3:</td>
<td>13</td>
<td>Variable</td>
<td>Variable</td>
<td>2.50</td>
<td>1.29</td>
<td>1.67</td>
</tr>
</tbody>
</table>
Table 4.2-2: Calculated Bridge Volumes

<table>
<thead>
<tr>
<th>Bridge Volumes:</th>
<th>Bridge Volume (cubic ft)</th>
<th>Air Pocket Volume (cubic ft)</th>
<th>Submerged Volume (cubic ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kuliouou Bridge:</td>
<td>4825.18</td>
<td>6950.69</td>
<td>11489.90</td>
</tr>
<tr>
<td>Kahaluu Stream Bridge:</td>
<td>23960.20</td>
<td>38322.20</td>
<td>60770.80</td>
</tr>
<tr>
<td>New South Punaluu Bridge:</td>
<td>14662.83</td>
<td>8804.49</td>
<td>22264.30</td>
</tr>
<tr>
<td>Ukoa Pond Bridge:</td>
<td>17828.70</td>
<td>35445.60</td>
<td>51665.10</td>
</tr>
<tr>
<td>New Makaha Stream #3A Bridge:</td>
<td>7745.12</td>
<td>0.00</td>
<td>7354.08</td>
</tr>
<tr>
<td>Old Makaha Stream #3A Bridge:</td>
<td>6933.77</td>
<td>2334.34</td>
<td>9002.97</td>
</tr>
<tr>
<td>Maipalaoa Bridge:</td>
<td>8694.72</td>
<td>12485.40</td>
<td>20621.30</td>
</tr>
<tr>
<td>Moanalua Bridge:</td>
<td>21900.94</td>
<td>22241.75</td>
<td>43362.10</td>
</tr>
<tr>
<td>Kalihi Bridge:</td>
<td>25790.98</td>
<td>13717.98</td>
<td>38816.90</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2:</td>
<td>13797.00</td>
<td>0.00</td>
<td>13473.00</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3:</td>
<td>14499.00</td>
<td>0.00</td>
<td>14067.00</td>
</tr>
</tbody>
</table>

*Note: Air pocket volumes are compressed values*
<table>
<thead>
<tr>
<th>Bridge Name</th>
<th>Self Weight (kips)</th>
<th>Buoyancy Force (kips)</th>
<th>Residual Weight (kips)</th>
<th>% Retained Weight</th>
<th>Buoyant?</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kuliouou Bridge:</td>
<td>723.777</td>
<td>735.35134</td>
<td>-11.57434</td>
<td>-1.60%</td>
<td>Yes</td>
</tr>
<tr>
<td>Kahaluu Stream Bridge:</td>
<td>3811.5</td>
<td>3889.33</td>
<td>-77.83</td>
<td>-2.04%</td>
<td>Yes</td>
</tr>
<tr>
<td>New South Punaluu Bridge:</td>
<td>2336.99</td>
<td>1424.91</td>
<td>912.08</td>
<td>39.03%</td>
<td>No</td>
</tr>
<tr>
<td>Ukoa Pond Bridge:</td>
<td>2674.31</td>
<td>3306.57</td>
<td>-632.26</td>
<td>-23.64%</td>
<td>Yes</td>
</tr>
<tr>
<td>New Makaha Stream #3A Bridge:</td>
<td>1161.77</td>
<td>470.66</td>
<td>691.11</td>
<td>59.49%</td>
<td>No</td>
</tr>
<tr>
<td>Old Makaha Stream #3A Bridge:</td>
<td>279.83</td>
<td>576.19</td>
<td>-296.36</td>
<td>-105.91%</td>
<td>Yes</td>
</tr>
<tr>
<td>Maipalaoa Bridge:</td>
<td>1406.69</td>
<td>1319.76</td>
<td>86.93</td>
<td>6.18%</td>
<td>No</td>
</tr>
<tr>
<td>Moanalua Bridge:</td>
<td>3338.16</td>
<td>2775.17</td>
<td>562.99</td>
<td>16.87%</td>
<td>No</td>
</tr>
<tr>
<td>Kalihi Bridge:</td>
<td>3955.61</td>
<td>2484.28</td>
<td>1471.33</td>
<td>37.20%</td>
<td>No</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2:</td>
<td>2069.453</td>
<td>862.272</td>
<td>1207.181</td>
<td>58.33%</td>
<td>No</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3:</td>
<td>2180.023</td>
<td>900.288</td>
<td>1279.735</td>
<td>58.70%</td>
<td>No</td>
</tr>
</tbody>
</table>
## Buoyancy Force Calculations: (50% air pocket)

<table>
<thead>
<tr>
<th>Bridge Type</th>
<th>Self Weight (kips)</th>
<th>Buoyancy Force (kips)</th>
<th>Residual Weight (kips)</th>
<th>% Retained Weight</th>
<th>Buoyant?</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kuliouou Bridge:</td>
<td>723.777</td>
<td>531.23</td>
<td>192.54</td>
<td>26.60%</td>
<td>No</td>
</tr>
<tr>
<td>Kahaluu Stream Bridge:</td>
<td>3811.5</td>
<td>2759.76</td>
<td>1051.74</td>
<td>27.59%</td>
<td>No</td>
</tr>
<tr>
<td>New South Punaluu Bridge:</td>
<td>2336.99</td>
<td>1220.16</td>
<td>1116.83</td>
<td>47.79%</td>
<td>No</td>
</tr>
<tr>
<td>Ukoa Pond Bridge:</td>
<td>2674.31</td>
<td>2275.30</td>
<td>399.01</td>
<td>14.92%</td>
<td>No</td>
</tr>
<tr>
<td>New Makaha Stream #3A Bridge:</td>
<td>1161.77</td>
<td>495.69</td>
<td>666.08</td>
<td>57.33%</td>
<td>No</td>
</tr>
<tr>
<td>Old Makaha Stream #3A Bridge:</td>
<td>279.83</td>
<td>518.46</td>
<td>-238.63</td>
<td>-85.28%</td>
<td>Yes</td>
</tr>
<tr>
<td>Maipalaoa Bridge:</td>
<td>1406.69</td>
<td>955.99</td>
<td>450.70</td>
<td>32.04%</td>
<td>No</td>
</tr>
<tr>
<td>Moanalua Bridge:</td>
<td>3338.16</td>
<td>2113.40</td>
<td>1224.76</td>
<td>36.69%</td>
<td>No</td>
</tr>
<tr>
<td>Kalihi Bridge:</td>
<td>3955.61</td>
<td>2089.60</td>
<td>1866.01</td>
<td>47.17%</td>
<td>No</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2:</td>
<td>2069.453</td>
<td>883.01</td>
<td>1186.45</td>
<td>57.33%</td>
<td>No</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3:</td>
<td>2180.023</td>
<td>927.94</td>
<td>1252.09</td>
<td>57.43%</td>
<td>No</td>
</tr>
</tbody>
</table>
4.2.3 Buoyancy Force Calculation: Analysis of Results

Column 3 (Residual Weight) of Table 4.2- is computed by subtracting the Buoyancy Force (column 2) from the Self Weight (column 1). The residual weight represents a critical measure of a bridge’s stability, where a negative value indicates that the bridge is buoyant. Column 4 is the percentage of the Residual Weight (column 3) divided by the original Self Weight (column 1). Again, a negative value indicates that the bridge has become buoyant and therefore further structural investigation is required.

After completing the initial buoyancy force calculations, it was found that the Kuliouou Stream Bridge, the Kahaluu Stream Bridge, the Ukoa Pond Bridge, and the Old Makaha Stream #3A Bridge all become buoyant once submerged (see Table 4.2-3). The air pocket volumes for each of these bridges were then halved. The buoyancy force acting on each bridge was recalculated with the reduced air volume (Table 4.2-4). After performing this less conservative calculation it was found that only the Old Makaha #3A Bridge remains buoyant. Nonetheless, in every situation the stabilizing self weight of each of these bridges is reduced due to buoyancy forces. The result is that much smaller wave forces can displace these bridges if they become submerged.

The buoyancy of the Old Makaha Stream #3A Bridge reveals an expected outcome, as the bridge is constructed mainly of Douglas fir wood. The Douglas fir wood used in Hawaii has a low specific gravity of 0.5. This indicates that, regardless of the amount of entrapped air under the bridge deck, the bridge will be buoyant once submerged.

The Ukoa Pond Bridge’s relatively large buoyancy force is attributed to the AASHTO Type III and Keehi Type IV girders used in the construction of the bridge deck. The two types of girders have differing heights of 3.75 feet and 4.83 feet, respectively. These tall girder heights allow large volumes of air to become trapped under the bridge deck, which causes large buoyancy forces to develop.
During Hurricane Katrina, the Railroad Bridge over Biloxi Bay had more than 28% of its original self weight once submerged and as a consequence was table to survive structurally intact [15]. If 28% Retained Weight is used as an initial benchmark for survivability, then the Kuliouou Stream Bridge (26.6%) and the Kahaluu Bridge (27.59%) are both susceptible to failure during storm or tsunami inundation, even though they are not fully buoyant.

4.3 Overview of Subsequent Bridge Analysis Process

To determine if a particular bridge will survive a storm or tsunami inundation event, the lateral and vertical connection capacity and the negative bending strength of the bridge decks and girders are calculated in the remainder of this chapter. The capacity of each bridge is then compared against estimated storm wave forces calculated in Chapter 5. Chapter 6 presents, conclusions drawn regarding the bridges’ survivability during a storm or tsunami event.
4.4 Bridge Resistances to Vertical and Horizontal Wave Loads

4.4.1 Approach

Waves impart both a lateral force (horizontal force) and upward force (vertical force) on a bridge deck. These forces also produce an overturning moment. To estimate the worst case scenario, the overturning moment is calculated at the far edge of the bridge deck (i.e. opposite side of incoming waves). It is assumed that the bridge foundation (bridge substructure) will remain intact during a storm event and that failure will occur if the bridge deck (bridge superstructure) is displaced or severely damaged. Further information on wave force estimation calculations can be found in Section 5 of this thesis.

To determine the lateral and vertical capacities of each bridge, the plans provided by the HDOT were examined for connections and any other sources of vertical or lateral resistance. The strength of each structural component was calculated by utilizing structural capacity computational methods developed by the American Institute of Steel Construction [4], the Precast/Prestressed Concrete Institute [5] and the American Concrete Institute [6].

If the resistance provided by the bridge self weight, friction, connections, wing walls, or any other form of structural component, is greater than the estimated wave forces and associated overturning moment, then it is concluded that the bridge will survive the storm event. The results of the bridge capacity calculations for each bridge are shown in the following sections.
4.4.2 Kuliouou Stream Bridge Structural Analysis

The Kuliouou Stream Bridge is a simply supported single span bridge and is approximately 50 feet long and 68.75 feet wide. The bridge is located on the south east coast of Oahu and is part of the main route between Hawaii Kai and Honolulu. Figure 4-2 shows the map location of the bridge and Figure 4-3 shows a picture of the bridge taken during the site survey. Figure 4-4, Figure 4-5 and Figure 4-6, show the construction drawings, where east of the bridge is denoted as ‘Koko Head’ and west of the bridge is indicated as ‘Diamond Head’.

The bridge deck is composed of three sections (Figure 4-4). The center portion was constructed in 1936, and was poured integrally in place with the reinforced concrete tee girders. To accommodate increasing traffic, the Kuliouou Stream Bridge was widened in 1963 through the construction of the outermost sections. The widened sections are composed of reinforced concrete decks attached to prestressed girders using #5 and #4 stirrups. After constructing the widened sections, the three separate bridge segments were connected using #4 reinforcing bars and epoxy. Throughout the calculations of the Kuliouou Stream Bridge, it was assumed that the bridge deck performed as an integrated single span.

The girders are supported at each of the abutments by reinforced concrete shelves. At the Koko Head abutment each girder rests on a 9” x ¾” x 19” neoprene pad. The neoprene pads provide a coefficient of friction of approximately 0.1, allowing the bridge to expand and contract without developing cracks.
**Figure 4-2**: Kuliouou Stream Bridge map location

**Figure 4-3**: Picture of Kuliouou Stream Bridge looking north
Figure 4-5: Kuliouou Bridge Plan View

Figure 4-4: Kuliouou Bridge Profile View
4.4.2.1 Kuliouou Stream Bridge: Lateral Resistance

At the Diamond Head abutment, the girders are connected to the foundation shelves using \(\frac{3}{4}\)" thick galvanized steel plates (see Figure 4-6). A plate is attached directly to the bridge girder using two \(\frac{3}{4}\)" diameter x 6" long embedded steel studs. A similar plate is attached to the abutment with another pair of \(\frac{3}{4}\)" diameter x 6" long steel studs. The two plates are welded together on both sides of the girder using 3/8" by 9" long fillet welds. There are a total of 8 bearing plates in total at the Diamond Head abutment.

![Figure 4-6: Diamond Head abutment bearing plates (profile view)](image)

Lateral resistance provided by the bearing plates is computed by determining the stud shear strength, weld strength, base metal shear strength, stud block shear, and weld block shear capacities. Concrete side break out was not considered because of sufficient reinforcement in the area of the studs in both the girder and the abutment (ACI 318-08 section RD 6.2.9). Results of the lateral resistance calculations are summarized in Table 4.4-1.
The results in Table 4.4-1 show that the overall capacity of the bearing plates is dependent on the studs’ shear strength. The shear failure capacity of the two studs embedded into the abutment is calculated to be 43.2 kips. In total, the 8 bearing plates at the Diamond Head abutment provide 345 kips of lateral resistance.

At both abutments of the Kuliouou Stream Bridge, wing walls have been constructed to restrain lateral movement. The wing walls at each abutment vary in shape and length due to the offset of the bridge. Waves will only strike the south facing or ocean side of the bridge. Therefore, only the North facing wing walls will provide lateral resistance against displacement (see Figure 4-7). In the situation of drawdown after a tsunami inundation, loads will be applied towards the coast. In this case, only the South facing walls will provide lateral resistance.

Table 4.4-1: Bearing Plate Lateral Capacity

<table>
<thead>
<tr>
<th>Capacity Calculation</th>
<th>Capacity (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Stud Shear Strength (in abutment)</strong></td>
<td>43.16</td>
</tr>
<tr>
<td>Stud Shear Strength (in girder)</td>
<td>48.82</td>
</tr>
<tr>
<td>Stud Block Shear</td>
<td>572.34</td>
</tr>
<tr>
<td>Weld Strength</td>
<td>200.44</td>
</tr>
<tr>
<td>Weld Block Shear</td>
<td>877.50</td>
</tr>
<tr>
<td>Base Metal Shear Strength</td>
<td>394.00</td>
</tr>
</tbody>
</table>

Figure 4-7: Lateral Wave Load Direction (profile view)
The lateral capacity of the wing walls is calculated by computing the failure strength of two planes. The horizontal plane will most likely fail in shear, while the vertical plane will fail due to flexure. The failure planes can be seen in Figure 4-8 and Figure 4-9.

Figure 4-8: Wing Wall Horizontal Failure Plane (profile view)

Figure 4-9: Wing Wall Vertical Failure Plane (plan view)
The capacity of the horizontal plane is computed by determining the shear strength of the concrete in the cross-sectional area of the plane. The capacity of the vertical plane is determined by computing the moment capacity of the wing wall, as if it were a cantilever beam. The moment capacity of the simulated cantilever beam is then divided by the distance from the point where the girder will impact the wing wall to the abutment. The sum of the shear and flexure capacities results in the total strength of each of the wing walls. The results of these calculations are summarized in Table 4.4-2.

Table 4.4-2: Summary of Wing Wall Capacities

<table>
<thead>
<tr>
<th>Wing Wall Capacity</th>
<th>Capacity (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Koko Head Horizontal Plane (Shear)</td>
<td>31.55</td>
</tr>
<tr>
<td>Koko Head Vertical Plane (Flexure)</td>
<td>27.85</td>
</tr>
<tr>
<td><strong>Koko Head Total Capacity</strong></td>
<td><strong>59.40</strong></td>
</tr>
<tr>
<td>Diamond Head Horizontal Plane (Shear)</td>
<td>24.20</td>
</tr>
<tr>
<td>Diamond Head Vertical Plane (Flexure)</td>
<td>41.76</td>
</tr>
<tr>
<td><strong>Diamond Head Total Capacity</strong></td>
<td><strong>65.96</strong></td>
</tr>
</tbody>
</table>

As seen in Figure 4-4, the bridge deck is sloped toward the ocean to allow proper drainage of water. This causes the bridge to creep downward over time. To counteract the downward movement, 2 ‘creep blocks’ were installed at the Koko Head abutment (see Figure 4-10). As a secondary effect, the creep blocks also provide lateral resistance to wave impact forces.
When the bridge deck is impacted, the wave loads are transferred to the bridge girders, which ultimately transfers the load to the creep blocks. The creep blocks provide lateral resistance through shear friction. These blocks were poured monolithically with the bridge foundation and were reinforced with 6 - #4 stirrups.

However, the bridge girder web may fail in transverse flexure before the creep blocks. A 45 degree line from the end of the creep block to the top of the beam web represents the most likely concrete cracking plane (see Figure 4-11).
For simplicity, the wave load is applied at the creep block where a failure is most likely to occur (see Figure 4-12). Applying a lateral load causes the girder to bend at the top of the beam web. The only reinforcement in the girders that provides flexural resistance in this orientation is the #5 shear stirrups in the web. The web was analyzed as a rectangular cantilever beam, with a width of 39.4 inches and a thickness of 6 inches. The width of the cantilever beam is taken as the distance from the end of the girder to the top of the cracking plane as seen in Figure 4-11. The cantilever beam thickness is taken as the thickness of the girder web.
The moment capacity of the beam web is then divided by the distance from the centroid of the creep block to the top of the girder web (see Figure 4-12). The results of the creep block calculations are summarized in Table 4.4-3.

**Table 4.4-3: Creep Block Region Capacities**

<table>
<thead>
<tr>
<th>Capacity Calculation</th>
<th>Capacity (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Creep Block Shear Friction</td>
<td>134.40</td>
</tr>
<tr>
<td>Girder Web Flexure</td>
<td>20.51</td>
</tr>
</tbody>
</table>

From Table 4.4-3, the total lateral resistance provided by a creep block, is limited by the beam web flexural capacity. For both creep block regions the total lateral resistance is 41.0 kips.

The total computed weight of the Kuliouou Stream Bridge is 724 kips. The bridge’s self weight creates a vertical stabilizing force against the upward wave force component. At the Koko Head abutment the bridge girders rest on neoprene pads. Therefore only half of the bridge’s self weight provides a horizontal frictional force, which was computed to be 36 kips. This frictional force provides an additional source of lateral resistance against the horizontal force of impacting waves.
4.4.2.2 Kuliouou Stream Bridge: Vertical Resistance

The vertical resistance provided by the bearing plates at the Diamond Head Abutment is determined from the following calculations: the concrete breakout tension strength, the stud pullout strength, the steel stud tension strength, and the flexural capacity of the steel plates. The results of these calculations are summarized in Table 4.4-4.

Table 4.4-4: Bearing Plate Vertical Capacity

<table>
<thead>
<tr>
<th>Capacity Calculation</th>
<th>Capacity (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Break Out in Tension (girder)</td>
<td>67.62</td>
</tr>
<tr>
<td><strong>Concrete Break Out in Tension (abutment)</strong></td>
<td><strong>54.18</strong></td>
</tr>
<tr>
<td>Pull Out Strength of Stud (girder)</td>
<td>169.65</td>
</tr>
<tr>
<td>Pull Out Strength of Stud (abutment)</td>
<td>84.82</td>
</tr>
<tr>
<td>Steel Strength of Stud in Tension</td>
<td>57.43</td>
</tr>
<tr>
<td>Steel Plate Flexure Capacity</td>
<td>178.75</td>
</tr>
</tbody>
</table>

After completion of the vertical capacity calculations, the abutment tensile concrete breakout strength is found to be very close to the stud tensile strength. To be conservative, the lower of these two values is taken as the maximum vertical capacity of the bearing plates. Therefore, the vertical capacity of the bearing plates is controlled by the concrete strength of the abutment. The 8 bearing plates at the Diamond Head abutment are computed to provide a total of 433 kips of vertical resistance.
4.4.2.3 Discussion of Kuliouou Stream Bridge

The Kuliouou Stream Bridge is only restrained vertically at the Diamond Head abutment. If a vertical force impacts the bridge deck, the Koko Head side will lift, causing a large moment to occur at the opposite side. The moment will likely cause the bearing plates to fail. Therefore to be conservative, it is assumed that the bridge’s self weight is the only source of vertical resistance.

Additionally, since the bearing plates are only at the Diamond Head abutment, the Koko Head side will be free to rotate when impacted by a lateral force. The bridge will continue to rotate until it contacts the Koko Head wing wall. Because of the probable bridge rotation, the lateral resistance of the bridge is taken as a combination of the Koko Head wing wall, girder web flexure capacity at creep blocks, Diamond Head bearing plates, and the gravity load induced friction.

The resistance to an overturning moment was computed by multiplying the self weight of the bridge by the distance from the centroid to the far end of the bridge (see Figure 4-13).

Figure 4-13: Stabilizing Moment Calculation Diagram

A summary of the estimated structural resistance of the Kuliouou Bridge is given in Table 4.4-5.
**Table 4.4-5: Kuliouou Bridge Structural Resistance to Wave Loads**

<table>
<thead>
<tr>
<th>Bridge Resistance</th>
<th>Capacity (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bearing Plates</td>
<td>345.31</td>
</tr>
<tr>
<td>Koko Head Wing Wall</td>
<td>59.40</td>
</tr>
<tr>
<td>Girder Web Flexure</td>
<td>41.01</td>
</tr>
<tr>
<td>Friction</td>
<td>36.19</td>
</tr>
<tr>
<td><strong>Total Lateral Resistance</strong></td>
<td><strong>481.91</strong></td>
</tr>
<tr>
<td><strong>Total Vertical Resistance (self weight)</strong></td>
<td><strong>723.78</strong></td>
</tr>
<tr>
<td><strong>Overturning Moment Resistance</strong></td>
<td><strong>24,879.9 kip-ft</strong></td>
</tr>
</tbody>
</table>
4.4.3 Kahaluu Stream Bridge

The Kahaluu Stream Bridge is composed of three 106 ft long by 46 ft wide spans connected by #9 and #6 reinforcement bars. The center portion of the bridge is supported by two concrete pile caps spaced 106 ft apart. The bridge is located on the eastern coast of Oahu and provides the main route from Kaneohe to the North Shore of Oahu. Figure 4-14 shows the map location of the bridge. Figure 4-15 is a photo taken during the site survey.

The bridge superstructure is composed of a reinforced concrete deck attached to eight pre-stressed girders using #5 bent stirrups. Numerous reinforcements are used to connect each of the spans to the adjacent deck thus creating an integrated uniform span. Therefore for calculation purposes, it is assumed that the three bridge spans act as a continuous section over the center supports.

![Kahaluu Bridge Map Location](image)

*Figure 4-14: Kahaluu Bridge Map Location*
4.4.3.1 Kahaluu Bridge: Lateral Resistance

At both abutments the girders are supported by Flurocarbon uni-ton bearing pads (see Figure 4-16 and Figure 4-17). The bearing pad is a “pot bearing” type pad and is comprised of a neoprene disk confined on all sides by a shallow steel ring. The top plate is secured to the bottom of the girder using six ½" diameter x 8” long steel Nelson studs. The masonry plate is attached to the abutment using another set of six ½” diameter x 8” long steel Nelson studs.
Figure 4-16: Fluorocarbon Uni-Ton Bearing Pads (profile view)

Figure 4-17: Fluorocarbon Uni-Ton Bearing Pad (close up view)
The steel ring around the neoprene pad confines lateral movement and transfers the horizontal loads to the abutment and girders. The steel ring capacity, Nelson stud shear capacity, steel plate block shear, weld capacity, and concrete breakout strength were calculated to determine the lateral capacity of the bearing pads. The results of these calculations are summarized in Table 4.4-6.

**Table 4.4-6: Bearing Pad Lateral Capacity**

<table>
<thead>
<tr>
<th>Capacity Calculation</th>
<th>Capacity (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel Ring Capacity</td>
<td>91.2</td>
</tr>
<tr>
<td><strong>Stud Shear Strength (6 studs)</strong></td>
<td><strong>47.89</strong></td>
</tr>
<tr>
<td>Steel Plate Block Shear</td>
<td>1067.1</td>
</tr>
<tr>
<td>Weld Capacity</td>
<td>178.2</td>
</tr>
<tr>
<td>Girder Combination Capacity</td>
<td>62.99</td>
</tr>
<tr>
<td><strong>Abutment Combination Capacity</strong></td>
<td><strong>48.14</strong></td>
</tr>
</tbody>
</table>

As seen in Figure 4-18, the most likely failure mechanism of the bearing pads is a combination of concrete break out and stud shear. The capacity of this failure mechanism is labeled as “Abutment Combination Capacity” and “Girder Combination Capacity” in Table 4.4-6. However, this situation only applies to the two bearing pads on the West side of the bridge (i.e. opposite side of wave impact). The west side bearing pads have been constructed 8.5 inches away from the edge of the abutment, and are therefore susceptible to concrete break out. In contrast, the interior bearing pads have sufficient concrete to reduce the likelihood of concrete break out. Therefore the 14 remaining bearing pads will fail due to shearing of all six Nelson studs.

There are 16 total bearing pads on the Kahaluu Bridge. The resulting lateral resistance provided by these bearing pads is computed to be 767 kips.
Similar to the Kuliouou Stream Bridge, wing walls were constructed on the Kahaluu Bridge to provide resistance against lateral movement. The same process described in section 4.4.2 was undertaken to compute the capacity of the wing walls. Again, the horizontal plane was computed to fail in shear, while the vertical plane was calculated to fail in flexure. The wing walls at each abutment have the same geometry, which results in the same computed capacities. The results of these calculations are summarized in the following table.

**Table 4.4-7: Kahaluu Bridge Wing Wall Capacity**

<table>
<thead>
<tr>
<th>Wing Wall Capacity</th>
<th>Capacity (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal Plane (Shear)</td>
<td>59.2</td>
</tr>
<tr>
<td>Vertical Plane (Flexure)</td>
<td>90.43</td>
</tr>
<tr>
<td><strong>Total Capacity (per wing wall)</strong></td>
<td><strong>149.63</strong></td>
</tr>
</tbody>
</table>

### 4.4.3.2 Kahaluu Bridge: Vertical Resistance

In the vertical direction, the steel piston of the Flurocarbon uni-ton bearing pad is free to move vertically. Therefore the bearing pads do not provide any vertical resistance to upward wave loads. The only source of vertical resistance comes from the...
bridge’s self weight, which was calculated to be 3,812 kips. The gravity induced lateral frictional force resulting from the neoprene pad interface was computed to be 382 kips.

4.4.3.3 Discussion of Kahaluu Stream Bridge

The steel ring confining the piston of each bearing pad is only 0.5 inches high. Therefore it is possible that the bridge will be lifted out of the bearing pads. To be conservative, the lateral resistance provided by the bearing pads has been ignored. In addition, once the bridge is lifted out of the bearing pads, the top steel plate is no longer in contact with the neoprene pad. Instead, the top steel plate will rest on the concrete abutment. This increases the coefficient of friction to 0.4 (i.e. steel to concrete interface), resulting in an increased lateral frictional force.

The lateral resistance provided by the Kahaluu Bridge was taken as the sum of the capacity of the wing walls and the gravity induced frictional force. In the vertical direction, only the self weight of the bridge provides resistance.

A summary of the estimated structural resistance of the Kahaluu Bridge can be found in Table 4.4-8.

Table 4.4-8: Kahaluu Bridge Structural Resistance to Wave Loads

<table>
<thead>
<tr>
<th>Bridge Resistance</th>
<th>Capacity (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wing Walls (2)</td>
<td>299.26</td>
</tr>
<tr>
<td>Friction</td>
<td>1,524.62</td>
</tr>
<tr>
<td><strong>Total Lateral Resistance</strong></td>
<td><strong>1,823.9</strong></td>
</tr>
<tr>
<td><strong>Total Vertical Resistance (self weight)</strong></td>
<td><strong>3,811.5</strong></td>
</tr>
<tr>
<td>Overturning Moment Resistance</td>
<td>87,665.7 kip-ft</td>
</tr>
</tbody>
</table>
4.4.4 Ukoa Pond Bridge

The Ukoa Pond Bridge is a four span simply supported bridge. The bridge is located on the northern coast of Oahu and is sheltered from wave forces by thick brush. Due to its location, the Ukoa Pond Bridge may not be exposed to wave forces, but may become submerged by tsunami inundation. Therefore this bridge was structurally evaluated only to determine its vulnerability to buoyancy.

From section 4.2.2, the bridge’s self weight and buoyancy force with a 100% air pocket is computed to be 2,674 kips and 3,307 kips, respectively. This results in a negative residual weight, indicating that the bridge is buoyant. The residual weight of the Ukoa Pond Bridge is computed to be -632 kips. This relatively large buoyancy force can be attributed to the large volume of air that may become trapped under the bridge deck (see Figure 4-20). To determine if the bridge would fail due to buoyancy the bridge plans were inspected for any sources of vertical resistance.
The only source of vertical resistance, in addition to the bridge's self weight, results from vertical hinge restrainers. At each abutment four vertical hinge restrainers are embedded into both the bridge superstructure and substructure. The restrainers are composed of 6 x 19 wire strands wound into a ¾ inch continuous looped galvanized cable (see Figure 4-21). The tensile breaking strength of the hinge restrainers is estimated to be 46 kips. In total there are 8 vertical hinge restrainers securing each bridge span to the foundation, which produces a total vertical capacity of 368 kips. The sum of the self weight and the tensile capacity of the hinge restrainers is computed to be 3,042 kips. However, the buoyancy force is still 264 kips greater than the total vertical bridge capacity. This indicates that, if air fills the entire volume of voids under the bridge, the resulting upward buoyancy force will break the hinge restrainers, and will likely cause the bridge deck to float off of its supports.
As water levels rise due to storm surge or a tsunami, it is possible that a quantity of air will escape from beneath the bridge deck. For this reason, the buoyancy force calculation is performed again with a non-conservative air volume of 50%. The recalculated buoyancy force is computed to be 2,275 kips, which is a 31.2% reduction from the previous force. The resulting buoyancy force is 767 kips less than the total vertical bridge capacity. Therefore, with the non-conservative calculation, the bridge is not buoyant and not at risk of failing. However, because the bridge is approximately 1,400 feet away from the shore, the rise in water elevation during a storm event will be gradual. The slow rise in water will likely cause more air to become trapped under the bridge deck. Therefore, an air volume of 50% may underestimate the volume of entrapped air. For the buoyancy force to exceed the total vertical bridge capacity, a minimum of 83.8% of the void volume under the bridge deck needs to be filled with air.

![Figure 4-21: Vertical Hinge Restrainer](image)
4.4.4.1 Discussion of Ukoa Pond Bridge

The main concern with the Ukoa Pond Bridge is its potential buoyancy. Even though the bridge is sheltered from wave forces, storm surge or tsunami inundation may cause the water levels to rise in the area of the bridge. If the bridge becomes submerged, air will become trapped under the bridge deck, possibly making it buoyant.

The bridge will fail if 84% of the volume under the bridge deck is filled with air. Under this condition, the upward buoyancy force is greater than the total vertical bridge capacity. In addition, the vertical hinge restrainers will likely fail, which will allow water current to displace the bridge. As a consequence, if air fills 84% or more of the volume under the bridge deck during bridge submergence, then the Ukoa Pond Bridge may be unusable after a storm or tsunami event.
4.4.5 Old Makaha #3A Bridge

The Old Makaha #3A Bridge is a single span wood framed simply supported bridge and is approximately 79 ft long and 33 ft wide. The bridge is located along the western coast of Oahu on Farrington Highway (see Figure 4-22).

Figure 4-22: Old Makaha #3A Bridge Map Location

Figure 4-23: Photograph of Old Makaha #3A Bridge Looking South East
The majority of the bridge was constructed using Douglas fir wood. Douglas fir wood in Hawaii has a typical specific gravity of 0.5. The underside of the bridge deck was reinforced with twenty W10x22 steel girders. The steel girders were added to provide additional structural capacity after existing 16 x 8 wood stringers developed horizontal cracks.

Small “L” shaped concrete abutments were constructed at both ends of the bridge. Each abutment is connected to the bridge deck using ten ¾ inch diameter x 18 inch long bolts. The concrete abutments are 3 feet tall by 2.25 feet wide, and are not secured to the ground (see Figure 4-24). The resistance to vertical displacement provided by the abutments is taken as the weight of the concrete. The abutment weight is added to the self weight of the bridge.

![Figure 4-24: Concrete Abutment](image)

The center portion of the bridge deck is supported by two wooden bents, which rest on concrete footings (Figure 4-25). The bottoms of the bents are connected to the footings using four 7/8 inch diameter x 22 inch long drift bolts. The drift bolts provide both lateral and vertical resistance.
4.4.5.1 Old Makaha #3A Bridge: Vertical Resistance

The vertical resistance is computed by finding the ultimate withdrawal load of drift bolts in wood. The bolts are imbedded into the wood bents a depth of 6 inches. It was determined that the maximum withdrawal capacity of each bolt is 8.7 kips. In total there are 8 drift bolts, which results in a total vertical resistance of 69.3 kips.

4.4.5.2 Old Makaha #3A Bridge: Lateral Resistance

The horizontal resistance is computed by determining the shear resistance provided by the bolts. The lateral resistance is computed to be 1.8 kips per bolt. In total the 8 bridge bolts provide 14.4 kips of lateral resistance.

Figure 4-25: Wooden Bent
4.4.5.3 Discussion of Old Makaha #3A Bridge:

From Section 4.2.2 it was determined that the Old Makaha #3A Bridge is buoyant once submerged. Upon recalculating the buoyancy force with a reduced air pocket volume, the bridge was still found to be buoyant. This is an expected result as the specific gravity of the wood used to construct the bridge is 0.5. The buoyancy force exceeds the entire self weight of the bridge by 239 kips. The difference between the buoyancy force and self weight is far greater than the 69.3 kips of vertical resistance provided by the drift bolts. For this reason, once the bridge is submerged the drift bolts will fail, making the Old Makaha #3A Bridge very susceptible to failure due to buoyancy.

If the bridge is not fully submerged then the bridge will be impacted by lateral and vertical wave forces. In the horizontal direction the total resistance provided by the bridge is taken as the total of the gravity induced frictional force plus the shear resistance of the drift bolts. A coefficient of friction of 0.2 was used for the wood bent to wood plank interface (see Figure 4-25). In the vertical direction the total resistance provided by the bridge is taken as the sum of the bridge self weight plus the ultimate withdrawal load of the drift bolts. The overturning moment resistance is computed by multiplying the self weight of the bridge by half the bridge width. In addition, the vertical resistances provided by the bolts are added to the moment capacity. A summary of the calculations can be found in Table 4.4-9.

Table 4.4-9: Old Makaha #3A Bridge Structural Resistance to Wave Loads

<table>
<thead>
<tr>
<th>Bridge Resistance:</th>
<th>Capacity (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bolt Shear Capacity</td>
<td>14.41</td>
</tr>
<tr>
<td>Friction</td>
<td>55.97</td>
</tr>
<tr>
<td><strong>Total Lateral Resistance</strong></td>
<td><strong>70.37</strong></td>
</tr>
<tr>
<td>Self Weight</td>
<td>279.83</td>
</tr>
<tr>
<td>Withdrawal Load</td>
<td>69.30</td>
</tr>
<tr>
<td><strong>Total Vertical Resistance</strong></td>
<td><strong>349.13</strong></td>
</tr>
<tr>
<td>Overturning Moment Resistance</td>
<td>5,731.55 kip-ft</td>
</tr>
</tbody>
</table>
4.4.6 New Makaha #3A Bridge

The New Makaha #3A Bridge was designed to replace the bridge described in section 4.4.5. The bridge deck will be constructed using 9 prestressed hollow core planks that span the entire 70 ft length of the bridge. The planks are 4.83 ft wide by 2.33 ft high and have two 16 inch diameter void holes as shown in Figure 4-26. A 5.5 inch thick reinforced concrete topping will be secured to the top of the prestressed planks using #4 and #5 stirrups (see Figure 4-26). Due to the geometry of the hollow core planks, air will not become trapped under the bridge deck, reducing the buoyancy force acting on the bridge structure if it becomes submerged.

![Figure 4-26: Hollow Core Plank Cross Section](image)

The bridge deck will be attached to reinforced concrete abutments using two layers of #6 bent reinforcement bars. These bars will be embedded into both the concrete deck and the abutment. This provides both lateral and vertical resistance to wave loads. Each abutment is heavily reinforced and will be connected directly to a concrete foundation buried a few feet underground. For this reason, it is assumed that wave forces will likely cause deck displacement before the abutments fail.
The only source of lateral and vertical resistances is provided by the abutment to bridge deck interface.

### 4.4.6.1 New Makaha #3A Bridge: Lateral Resistance

The abutment will provide horizontal resistance to wave loads through shear friction. Over the width of the bridge deck, there are 420 - #6 bent reinforcement bars connecting the deck to the two abutments. The resulting shear friction capacity for both abutments is 8,870 kips.
4.4.6.2 New Makaha #3A Bridge: Vertical Resistance

The vertical capacity is dependent on the tensile strength of the #6 reinforcement. The minimum force needed to break all of the reinforcement bars at both abutments in tension was computed to be 16,630 kips; when added to the self weight of the bridge deck, the total vertical resistance results in a value of 17,800 kips. The overturning moment resistance provided by both the self weight and #6 rebars was computed to be 416,640 kip-ft.

4.4.6.3 New Makaha #3A Bridge: Prestressed Deck Capacity

However, the hollow core planks may fail before they are displaced. The prestressing in the hollow core planks are designed with an upward camber (negative bending) normally used to negate the downward sag resulting from gravity loads. During a storm event, upward wave loads adversely cause an additive effect to the prestressing force, which increases the negative bending of the planks (see Figure 4-28). Due to buoyancy forces, the self weight of the bridge will be reduced. This reduction in self weight decreases the counteractive positive bending effect caused by dead loads. The combination of the associated storm wave forces and prestressing effect may ultimately cause the planks to fail in negative bending [15].

![Negative Bending](image)

*Figure 4-28: Negative Bending*
Negative bending causes tensile stresses to develop in the top of the plank and compressive stresses in the bottom. To compute these stresses the following equations are commonly used [5]:

\[
f_{\text{top}} = -\frac{P_e}{A_c} \left( 1 - \frac{e \cdot c_t}{r^2} \right) - \frac{M_T}{S_t}
\]

(4.4-1)

\[
f_b = -\frac{P_e}{A_c} \left( 1 + \frac{e \cdot c_b}{r^2} \right) + \frac{M_T}{S_b}
\]

(4.4-2)

In which

- \( f_{\text{top}} \) = stress at the top fibers [psi]
- \( f_b \) = stress at the bottom fibers [psi]
- \( P_e \) = effective prestressing force after losses [lbs]
- \( A_c \) = cross sectional area of concrete [in²]
- \( e \) = distance from centroid of prestressing to centroid of the plank [in]
- \( c_t \) = distance from top of plank to centroid (rectangular section = \( \frac{1}{2} h \)) [in]
- \( c_b \) = distance from bottom of plank to centroid (rectangular section = \( \frac{1}{2} h \)) [in]
- \( r^2 \) = moment of inertia divided by the cross sectional area of concrete [in²]
- \( M_T \) = Total moment acting at the center of the plank span [lb-in]
- \( S_t \) = moment of inertia divided by \( c_t \) [in³]
- \( S_b \) = moment of inertia divided by \( c_b \) [in³]
The maximum permissible tensile stress is taken as:

\[ f_t = 12 \times \sqrt{f_c'} \]

The maximum permissible compressive stress is taken as:

\[ f_c = -0.85 \times f_c' \]

In which

\[ f_c' = \text{specified 28 day compressive strength of concrete [psi]} \]

If the aforementioned stresses are exceeded, structural cracks will develop in the concrete. With repeated wave impacts the cracks will widen, which will likely lead to severe bridge damage. It should be noted that the hollow core planks are also reinforced with non-prestressed steel. The reinforcing steel will aid in controlling the size of the cracks and add to the bending resistance, but will not prevent cracks from developing [5].

To determine the minimum wave load needed to cause deck failure, Equation 4.4-1 was set equal to \( f_t \) and Equation 4.4-2 was set equal to \( f_c \). Tensile stresses were taken as a positive value, and compressive stresses were taken as negative. Equation 4.4-1 and Equation 4.4-2 were then rearranged to solve for \( M_T \).

To simplify calculations, individual planks were analyzed separately. A distributed upward wave load was applied to the underside of the hollow core plank and the sum of the dead loads was applied as a distributed downward load (see Figure 4-29).

![Figure 4-29: Distributed Loads Acting On Bridge Deck](image-url)
The maximum moment \((M_T)\) at the center of the span was computed as a function of the wave load. For distributed loads applied on the top and bottom of the bridge deck, the resulting \(M_T\) is as follows:

\[
M_T = M_D - \frac{W_w L^2}{8}
\]  

(4.4-3)

in which

\[M_D = \text{positive moment due to dead loads [lb-in]}\]
\[w_D = \text{distributed dead load [lbs/in]}\]
\[L = \text{length of bridge [in]}\]
\[W_w = \text{distributed wave load [lbs/in]}\]

To determine the minimum wave load that will cause the stresses in the concrete to exceed the permissible limits, the stress limit equations and the moment equation were combined to solve for the wave loads. The following equations were developed as a result of this process:

\[W_{FT} = \frac{8}{L^2} \left[ M_D + (S_t * f_t) - \left( \frac{S_t * P_e}{A_c} \right) \left( \frac{e_c}{r^2} - 1 \right) \right] \]  

(4.4-4: Tensile Limit)

\[W_{FC} = \frac{8}{L^2} \left[ M_D - (S_b * f_c) - \left( \frac{S_b * P_e}{A_c} \right) \left( 1 + \frac{e_c}{r^2} \right) \right] \]  

(4.4-5: Compression Limit)

The resulting \(W_{FT}\) and \(W_{FC}\) are in units of pounds per inch. The lower of the two values controls the minimum wave load needed to cause failure. In other words, Equation 4.4-4 and Equation 4.4-5 determine the negative bending capacity of a prestressed concrete member.

To determine the negative bending capacity of the New Makaha #3A Bridge the water elevation was taken at the bottom of the hollow core planks. In this situation the
deck is not submerged and therefore buoyancy was not considered. Using Equation 4.4-4, the wave load limit in tension is computed to be 149 lbs/inch per plank. Using Equation 4.4-5, the wave load limit in compression is computed to be 480 lbs/inch per plank. The lower of the two computed values is chosen as the capacity of the hollow core member.

It was found that in most cases the tensile limit (Equation 4.4-4) will control the capacity of a prestressed member. This is a reasonable result, as concrete is weak in tension while strong in compression.

To determine the capacity of the entire bridge deck, the 149 lbs/inch per plank capacity value is multiplied by the length of the bridge. This results in a value of 125 kips per plank. The deck of the New Makaha #3A Bridge is composed of 9 prestressed hollow core planks. Therefore, by combining the capacity of all 9 planks, the total negative capacity of the entire bridge deck is computed to be 1,127 kips.

The negative bending capacity must also be determined when the bridge is submerged. In this situation the bridge’s self weight is reduced. The buoyancy force computed in Section 4.2.2 is subtracted from the dead loads. This results in a smaller value of $w_D$, effectively lowering the negative bending capacity of the deck. Using Equation 4.4-4, the wave load limit in tension is computed to be 86 lbs/inch per plank. Using Equation 4.4-5, the wave load limit in compression is computed to be 383 lbs/inch per plank. Again, the lower of the two computed values is chosen as the capacity of the hollow core member. The total negative bending capacity of the entire bridge deck when submerged is 654 kips.

In the final analysis, the resulting negative bending capacity of the bridge deck is far below the lateral and vertical resistance provided by the deck to abutment interface. Therefore, the bridge deck will likely fail in negative bending before it is displaced from the abutment supports.
4.4.6.4 Discussion of New Makaha #3A Bridge

The New Makaha #3A Bridge deck is properly restrained in both the lateral and vertical direction. However, it was found that the bridge deck will likely fail in negative bending before it is displaced from its supports.

Once submerged, the tensile stress limit negative bending capacity of the deck is reduced by 42% and the compressive stress limit negative bending capacity is reduced by 20% (see Table 4.4-11). As a consequence, if the bridge becomes submerged the bridge’s negative bending capacity is significantly reduced. This will allow much smaller forces to damage or even fail the bridge deck.

A summary of the results from section 4.4.6 can be found in the following tables.

Table 4.4-10: New Makaha #3A Bridge Structural Resistance to Wave Loads

<table>
<thead>
<tr>
<th>Bridge Resistance</th>
<th>Capacity (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Lateral Resistance (shear friction)</td>
<td>8,870.40</td>
</tr>
<tr>
<td>Self Weight</td>
<td>1,161.80</td>
</tr>
<tr>
<td>Tensile Strength of Rebar</td>
<td>16,632</td>
</tr>
<tr>
<td>Total Vertical Resistance</td>
<td>17,793.80</td>
</tr>
<tr>
<td>Overturning Moment Resistance</td>
<td>416,671.5 kip-ft</td>
</tr>
</tbody>
</table>

Table 4.4-11: New Makaha #3A Bridge Negative Bending Capacity

<table>
<thead>
<tr>
<th>Deck Capacity</th>
<th>Capacity (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile Capacity</td>
<td>1,127.13</td>
</tr>
<tr>
<td>Compressive Capacity</td>
<td>3,625.89</td>
</tr>
<tr>
<td>Tensile Capacity (submerged)</td>
<td>653.572</td>
</tr>
<tr>
<td>Compressive Capacity (submerged)</td>
<td>2,898.44</td>
</tr>
<tr>
<td>Tensile % Loss once submerged</td>
<td>42.01%</td>
</tr>
<tr>
<td>Compressive % Loss once submerged</td>
<td>20.06%</td>
</tr>
</tbody>
</table>
4.4.7 New South Punaluu Bridge

Originally, the existing North Punaluu Bridge was selected for structural evaluation. However, since as-built plans were not available for analysis, the New South Punaluu Bridge was chosen instead. Similar to the New Makaha #3A Bridge analyzed in section 4.4.6, the New South Punaluu Bridge is yet to be built and is planned to replace an existing bridge on the north east coast of Oahu.

The New South Punaluu Bridge will be constructed using 5 foot wide triple tee prestressed members (Tridecks). The bridge deck will be composed of three spans, with the center span being the longest. In total, the bridge will be 50 feet wide and 170 feet long.

The bridge will be supported by two abutments and two concrete piers. Figure 4-30 shows a section of the heavily reinforced triple tees. The triple tee stems are 21 inches in height, allowing air to be trapped beneath the bridge deck.

![Figure 4-30: Typical Triple Tee (Trideck)](image)

The bridge superstructure will be secured to the abutments using #10 reinforcing bars, which will provide both lateral and vertical resistance against wave loads. The bridge will also be restrained laterally by wing walls and a precast tub. It is not likely that
the precast tub was designed for this purpose. However, being 2.5 feet wide and heavily reinforced, the tub will provide some lateral resistance (see Figure 4-31: Precast Tub).

4.4.7.1 New South Punaluu Bridge: Negative Bending Capacity

Based on the results from section 4.4.6, it is concluded that the New South Punaluu Bridge is much more likely to fail in negative bending than being displaced. In addition, the triple tee girders are able to trap air, which will increase the upward buoyancy force acting on the bridge superstructure if it becomes submerged.

Each of the bridge spans will be constructed using 10 Tridecks (Figure 4-32). The center span is the most susceptible to failure, as it has the longest length. The longer length will cause a greater upward moment to develop at the middle of the span. Therefore, during calculations, only the center span is analyzed. If the center span fails, then so does the entire bridge.
To determine the negative bending capacity of the New South Punaluu Bridge, the procedure developed in Section 4.4.6 was followed. Each of the Tridecks was analyzed separately, and recombined after completing the negative bending calculations to determine the total deck capacity. The dead load of each Trideck includes the deck self weight and the weight of an asphalt wearing surface.

When not submerged, the tensile capacity of the center span is reached when the uplift load is 1,263 kips. In compression, the capacity of the deck is reached when the uplift force is 3,777 kips. The resulting capacities indicate that the resultant of a distributed upward wave force will have to be greater than or equal to 1,263 kips to cause the stresses in the concrete to exceed the permissible values and fail the bridge.

Unlike the flat underside of the New Makaha #3A Bridge, the geometry of the prestressed triple tee members, used in the construction of the New South Punaluu Bridge, allows air pockets to form under the bridge deck. As a consequence a larger buoyancy force results when the bridge becomes submerged. To compute the negative bending capacity of the submerged case, an air pocket volume of 100% was assumed. Assuming that air will fill the entire volume under the bridge deck may not be realistic;
however, it will result in the greatest buoyancy force possible, and is therefore conservative.

The resulting buoyancy force, caused by the total submerged volume, is subtracted from the dead loads of the bridge. Again, Equation 4.4-4 and Equation 4.4-5 are used to compute the negative bending capacities. For tension failure, the capacity of the center span is computed to be 813 kips. For compression failure, the capacity of the deck is computed to be 3,327 kips.

The results of the calculations are listed in the following table.

*Table 4.4-12: New South Punalu’u Bridge Negative Bending Capacity (Center Span)*

<table>
<thead>
<tr>
<th>Deck Capacity</th>
<th>Capacity (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Tensile Capacity</strong></td>
<td>1,262.99</td>
</tr>
<tr>
<td>Compressive Capacity</td>
<td>3,777.11</td>
</tr>
<tr>
<td><strong>Tensile Capacity (submerged)</strong></td>
<td>812.81</td>
</tr>
<tr>
<td>Compressive Capacity (submerged)</td>
<td>3,326.98</td>
</tr>
<tr>
<td>Tensile % Loss once submerged</td>
<td>35.64%</td>
</tr>
<tr>
<td>Compressive % Loss once submerged</td>
<td>11.92%</td>
</tr>
</tbody>
</table>
4.4.7.2 Discussion of New South Punaluu Bridge

The New South Punaluu Bridge will be constructed using prestressed triple tee members. When the upward force caused by the prestressing tendons is combined with an upward wave load, the maximum stresses in the concrete may be exceeded. It was found that a wave force resultant of 1,263 kips is needed to cause the tensile stress limit at the top of the bridge deck to be exceeded. This will cause concrete cracks to develop. With repeated wave impacts the cracks will continue to widen, which may ultimately lead to failure of the bridge.

The New South Punaluu is constructed only 4.92 feet above the mean sea level. As a result, during a storm event, submergence of the bridge deck is highly probable. However, even with an assumed maximum air pocket volume, it was determined that the bridge will not fail due to upward buoyancy forces alone.

Nonetheless, failure of similarly constructed parking garages in the Biloxi Gulfport region was observed during a post disaster survey. The parking garages were constructed using prestressed double tees. Some of the garages were protected from wave action by surrounding structures, but were still inundated by storm surge. The double tee geometry lent itself to trapping a large volume of air, which resulted in an adverse uplift force much greater than the submerged self weight of the prestressed members. The uplift force caused negative bending to develop. The combination of the buoyancy uplift force and the prestressing effect caused the double tees to fail. To avoid the failure mechanism, it is recommended that any prestresesed double tee systems, flat slab, and other concrete systems be designed to withstand the negative bending and shear caused by hydrodynamic and hydrostatic forces [15].
4.4.8 Maipalaoa (Maili Channel) Bridge

The Maipalaoa Bridge is a simply supported bridge and has a north - south orientation. The bridge is approximately 64 feet wide and 101 feet long. The bridge is located along the south western coast of Oahu and provides the only regular access to the Nanakuli region and Waianae communities (see Figure 4-33).

![Figure 4-33: Maipalaoa Bridge Map Location](image)

The bridge superstructure is a two span girder supported deck that is 101 feet long. It is supported at both ends by reinforced concrete abutments and at the center by a reinforced concrete pier. The bridge deck is reinforced with #6 rebars and #4 stirrups at the center pier. This center pier location serves as the transition zone between the two sets of girders (see Figure 4-34).
The bridge superstructure is not secured to the foundation vertically or horizontally. The only resistance to vertical wave loads is provided by the self weight of the bridge. The resistance to lateral wave loads is provided by gravity induced friction and creep blocks constructed at the abutments.

### 4.4.8.1 Maipalaoa Bridge: Vertical Capacity

The Maipalaoa Bridge is a gravity type bridge, where the self weight is the only source of vertical resistance to upward wave loads. The total self weight of the bridge is calculated to be 1,407 kips.
4.4.8.2 Maipalaoa Bridge: Lateral Capacity

The girders rest on neoprene pads at both abutments and at the center pier. The neoprene pads provide a coefficient of friction of 0.1. The resulting gravity induced lateral frictional resistance is computed to be 141 kips.

Similar to the Kuliouou Stream Bridge (section 4.4.2), creep blocks are constructed at each of the Maipalaoa Bridge abutments to prevent the bridge from moving laterally over time (see Figure 4-36). To determine the lateral capacity provided by the creep blocks, the failure capacities of the creep blocks and the prestressed tee girders were computed.

![Figure 4-36: Maipalaoa Bridge Creep Blocks](image)

The creep blocks are reinforced with seven #5 U shape stirrups and were poured monolithically with the bridge foundation. The main source of lateral capacity is provided by shear friction. The resulting capacity for each of the creep blocks is computed to be 243 kips. There are 4 creep blocks at each abutment, which results in a total lateral capacity of 1,944 kips for the entire bridge.

Unlike the girders on the Kuliouou Stream Bridge; there are no obvious weak planes on the prestressed tee girders. For this reason, a number of different calculations were performed to determine the lateral capacity of the bridge girders.

To simplify calculations, the web of the girder is oriented horizontally and the wave loads were applied directly at the creep block (see Figure 4-37). In this orientation, it is possible to analyze the girder web as a prestressed/reinforced concrete slab.
The first calculation assumes that the creep block will cause a punch out failure in the prestressed beam web. According to the American Concrete Institute, the critical failure area is determined by taking a distance $d_p/2$ away from the edges of the column, where $d_p$ is the distance from the centroid of the prestressing strands to the tension surface of the web (see Figure 4-38). The punch out calculation resulted in a failure capacity of 137 kips per girder.
Because the prestressing will only provide structural support a set distance away from the jacking zone (i.e. tendon stressing location), a similar, more conservative calculation is performed, which determines the punch out failure of a concrete slab with no additional reinforcement. The resulting punch out capacity is computed to be 66.9 kips.

The beam web capacity was also determined by summing the shear capacity of two failure planes (see Figure 4-39). The strength of the horizontal failure plane is determined by computing the shear capacity of the concrete in the cross sectional area of the plane. This calculation results in a value 24.8 kips. To determine the shear capacity of the vertical failure plane, the cross sectional area has to be analyzed as a rectangular prestressed beam. The resulting shear capacity of the pseudo prestressed beam is computed to be 21.7 kips. Due to the adjusted orientation of the beam web, the #4 stirrups used to connect the prestressed tee girders to the bridge deck, act as shear reinforcement. The total shear capacity provided by both planes and the #4 stirrups results in a value of 51.8 kips per girder.

![Figure 4-39: Failure Planes](image)

However, after the vertical plane has sheared, it is possible that the horizontal plane will fail in flexure (see Figure 4-40). Again, the cross sectional area of the horizontal failure plane is analyzed as a rectangular prestressed beam. The resulting flexure capacity is calculated to be 61.6 kips. The sum of the shear capacity of the
vertical plane and the flexure capacity of the horizontal plane results in a value of 86.4 kips per girder.

![Wave Load Direction](image)

**Figure 4-40: Horizontal Plane Flexure Failure**

The last calculation determines the flexure capacity at the top of the beam web. The top of the beam web represents the maximum moment location. A 45 degree angle line drawn from the end of the creep block to the top of the beam web represents the most likely concrete cracking plane (see Figure 4-41). The resulting flexure length is computed to be 48.5 inches from the end of the beam. Only the #4 stirrups attaching the prestressed tee girders to the bridge deck provide lateral flexure resistance. The resulting flexure capacity of the beam web is determined to be 15.3 kips per bridge girder.

The abovementioned flexure capacity is lower than the other calculated capacities. Therefore the capacity provided by the creep blocks is limited by the flexure capacity of the girder webs. The total resulting capacity provided by all 8 creep block regions is computed to be 123 kips.

A summary of the calculations performed to determine the capacity of the beam web can be found in the Table 4.4-13.
**Table 4.4-13: Summary of Creep Block Region Calculations**

<table>
<thead>
<tr>
<th>Capacity Calculation</th>
<th>Capacity (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Creep Block Shear Friction</td>
<td>243.04</td>
</tr>
<tr>
<td>Beam Web Prestressed Slab Punch Out</td>
<td>137.3</td>
</tr>
<tr>
<td>Beam Web Non-reinforced Slab Punch Out</td>
<td>66.93</td>
</tr>
<tr>
<td>Beam Web Independent Failure Plane (shear-shear)</td>
<td>50.48</td>
</tr>
<tr>
<td>Beam Web Independent Failure Plane (shear-flexure)</td>
<td>86.35</td>
</tr>
<tr>
<td><strong>Beam Web Flexure</strong></td>
<td><strong>15.34</strong></td>
</tr>
</tbody>
</table>

**Figure 4-41: Beam Web Cracking Plane**

![Beam Web Cracking Plane Diagram](image-url)
Figure 4-42: Beam Web Flexure
4.4.8.3 Discussion of Maipalaoa Bridge

The Maipalaoa Bridge is not secured to the center pier or abutments. The only source of vertical capacity results from the 1,407 kips of self weight. The lateral capacity provided by the creep blocks was determined to be limited by the transverse flexure strength of the girder webs. This resulted in a total capacity of 123 kips.

From the structural analysis of this bridge, it was concluded that the Maipalaoa Bridge is at risk of failing due to wave loads. The bridge has a limited amount of structural components securing the bridge against lateral and vertical displacement. What little resistances provided by the creep blocks and self weight can be overcome by wave loads and buoyancy forces.

A summary of the lateral and vertical resistances of the Maipalaoa Bridge are provided in Table 4.4-14.

Table 4.4-14: Maipalaoa Bridge Structural Resistance to Wave Loads

<table>
<thead>
<tr>
<th>Bridge Resistance</th>
<th>Capacity (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Total Vertical Resistance (self weight)</strong></td>
<td>1,406.70</td>
</tr>
<tr>
<td>Beam Web Flexure Capacity</td>
<td>122.68</td>
</tr>
<tr>
<td>Gravity Induced Friction</td>
<td>140.67</td>
</tr>
<tr>
<td><strong>Total Lateral Resistance</strong></td>
<td>263.35</td>
</tr>
<tr>
<td>Overturning Moment Resistance</td>
<td>45,248.85 kip - ft</td>
</tr>
</tbody>
</table>
4.4.9 Moanalua Bridge

The Moanalua Bridge is located in the southern region of Oahu and is sheltered from direct ocean waves by Keehi Lagoon (see Figure 4-43). The bridge superstructure is composed of eight independent reinforced concrete spans (Figure 4-44). Each bridge span is evaluated as a separate structural element. Each span is approximately 64.33 feet wide and 27 feet long and simply supported on reinforced concrete pier caps.

![Map Location of Moanalua Bridge](image)

*Figure 4-43: Map Location of Moanalua Bridge*

![Bridge Spans](image)

*Figure 4-44: Bridge Spans*
Originally, the Moanalua Bridge was chosen to be structurally evaluated because of connections found during the site survey. The connections shown in Figure 4-45 appear to connect the bridge deck to its foundation, which potentially could provide lateral and vertical resistance to wave loads. However, upon further investigation and review of the as-built plans, these connections could not be found. It is feasible that the connections were added after the original construction of the bridge. Or possibly, modifications were made to a water main support connection seen in Figure 4-46 and Figure 4-47. Ultimately, the capacity of the connections could not be determined due to lack of available information and updated drawings.

Figure 4-45: Moanalua Bridge Connections
Figure 4-46: Water Main Support Connection Detail

Figure 4-47: Water Main Support Profile View
4.4.9.1 Moanalua Bridge: Vertical Capacity

Of the eight deck spans, spans 2 and 7 are the most vulnerable to failure. Two - 1 inch diameter x 4 foot long steel dowels are used to connect each of the nine bridge girders to a supporting pier cap (see Figure 4-48). However, on spans 2 and 7, these dowels are only provided at one end. If the unrestrained side of the span lifts due to a wave impact, a moment will be caused at the fixed end, which can fail the dowels. Therefore, any resistance provided by the dowels is ignored for spans 2 and 7. As a result the only source of vertical resistance to upward wave forces is provided by the self weight of each span, which is computed to be 417 kips. The associated overturning moment resistance resulting from the self weight is computed to be 13,421 kip – ft.

Spans 3, 4, 5, and 6 are restrained at both ends by eighteen - 1 inch diameter x 4 foot long steel dowels. In total, there are 36 dowels connecting each span to the supporting pier caps. However, the pier caps themselves are not secured to the bridge foundation piles. Instead, the concrete pier caps have been constructed with female recesses, which allow the foundation piles to butt into the pier caps (see Figure 4-48). With this type of construction, if the deck spans are displaced upward, then so will the pier caps. Therefore additional resistance to vertical loads results from the self weight of the pier caps, which is computed to be 79.2 kips. The total vertical resistance to wave loads for spans 3, 4, 5, and 6 is the sum of the span self weight and the self weight of two supporting pier caps. This results in a value of 576 kips per span. The associated overturning moment resistance resulting from the total self weight is computed to be 18,520 kip – ft.
4.4.9.2 Moanalua Bridge: Lateral Capacity

For spans 2 and 7, the only source of lateral resistance to wave loads is provided by gravity induced friction. Each of the girders is in contact with the pier caps, which forms a concrete to concrete interface. This interface has a coefficient of friction of 0.8. The resulting lateral capacity provided by friction is 334 kips per span.

The steel dowels of spans 3, 4, 5, and 6 provide lateral resistance through shear friction. The shear friction capacity is computed to be 1,593 kips. When this is added to the gravity induced frictional resistance of each span, the total lateral resistance is computed to be 1,926 kips.
4.4.9.3 Discussion of Moanalua Bridge

The Moanalua Bridge is located in a harbor area (see Figure 4-49). This sheltered location protects the bridge from direct storm ocean waves. However, during a storm event, the still water level may raise enough to submerge low lying areas that normally protect the region from wave action. Therefore, this will allow waves that develop in the open ocean to impact the normally sheltered bridge locations.

Reinforced concrete pedestrian bridges are in the path of the waves for both the Moanalua and Kalihi Bridges. The pedestrian bridges will provide some protection from wave loads to a certain extent. However, if wave loads are large enough, these pedestrian bridges will fail. Eventually the bridges may get pushed into the Moanalua and Kalihi Bridges. It should also be noted that additional debris from the harbor may impact the bridge during a storm or tsunami event. The addition of wave and debris impact forces can potentially cause massive damage to the Moanalua Bridge, the Kalihi Bridge and the surrounding area.

Figure 4-49: Moanalua Bridge Location
The Moanalua Bridge, is composed of eight separate spans, of which spans 2 and 7 are the most likely to fail for reasons explained above. The only source of resistance results from the self weight of the spans. In Section 4.2 the percent retained weight of the Moanalua Bridge when submerged was computed to be 16.87% (Table 4.2). Ultimately, this reduction in self weight will allow much smaller wave forces to exceed the resistance of the bridge. In addition, with repeated wave impacts, it is likely that the deck spans will be displaced until removed completely from the pier cap supports [10]. If spans 2 and 7 fail, the entire Moanalua Bridge will be unusable.

A summary of the Moanalua Bridge structural calculations is provided in Table 4.4-15.

Table 4.4-15: Moanalua Bridge Structural Resistance to Wave Loads

<table>
<thead>
<tr>
<th>Spans 2 &amp; 7 Resistance</th>
<th>Capacity (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Total Vertical Resistance (self weight)</strong></td>
<td>417.27</td>
</tr>
<tr>
<td><strong>Total Lateral Resistance (friction)</strong></td>
<td>333.82</td>
</tr>
<tr>
<td><strong>Overturning Moment Resistance</strong></td>
<td>13,422 kip - ft</td>
</tr>
<tr>
<td>Spans 3, 4, 5, &amp; 6 Resistance</td>
<td>Capacity (kips)</td>
</tr>
<tr>
<td><strong>Total Vertical Resistance (total self weight)</strong></td>
<td>575.67</td>
</tr>
<tr>
<td><strong>Total Lateral Resistance</strong></td>
<td>1926.46</td>
</tr>
<tr>
<td><strong>Overturning Moment Resistance</strong></td>
<td>18,517 kip - ft</td>
</tr>
</tbody>
</table>
4.4.10 Kalihi Bridge

The Kalihi Bridge is also located in Keehi Lagoon, and is approximately 1,500 feet east of the Moanalua Bridge. The bridge is composed of seven independent deck spans. Each span is a reinforced concrete deck poured integrally with 13 tee girders and is 88.3 feet wide by 27 feet long. The spans are supported at both ends by reinforced concrete pier caps. All seven spans are very similar in construction and geometry.

Similar to the Moanalua Bridge, the Kalihi Bridge is connected to the pier caps using two - 1 inch diameter x 4 foot long steel dowels per girder. However, only three of the 13 girders are connected to the pier caps (Figure 4-50). The dowels at the opposite side of the wave load direction were likely installed to keep the bridge from sliding laterally over time. If a large enough wave force impacts the underside of the bridge, its unrestrained side will lift. As the bridge lifts, a moment will be caused at the opposite edge of the bridge deck. The resulting moment will cause the dowels to yield or shear. Therefore, similar to spans 2 and 7 of the Moanalua Bridge, any lateral and vertical capacities provided by the dowels on the Kalihi Bridge are ignored.

Small, 4 inch high, unreinforced creep blocks were constructed at both abutments and at piers 2, 4, and 5. However, with such a low profile, it is possible that the bridge will be lifted over these creep blocks as the bridge is impacted by wave forces. For this reason and to be conservative, the lateral capacity provided by the creep blocks is ignored.

Therefore, the only resistance to vertical and lateral wave loads is provided by the self weight of each bridge span.
4.4.10.1 Kalihi Bridge: Vertical Resistance

Since the steel dowels are at a risk of failing when storm waves impact the bridge, only the self weight of the Kalihi Bridge spans provides persistent vertical resistance. The resulting self weight of a single span is 565 kips and the associated overturning moment resistance is 24,950 kip-ft.

4.4.10.2 Kalihi Bridge: Lateral Resistance

The only source of lateral resistance to wave loads is provided by friction. The girders rest directly on the pier caps, forming a concrete to concrete interface. The resulting gravity induced friction resistance is 452 kips.

4.4.10.3 Discussion of Kalihi Bridge

The Kalihi Bridge’s primary source of resistance to lateral and vertical wave forces is provided by the self weight of each span. From Section 4.2 the percent retained weight of the Kalihi Bridge was computed to be 37.2% and is not at risk of being buoyant. However, if the self weight is reduced to 37.2%, it will significantly diminish the Kalihi Bridge’s only resistance to wave loads, making the bridge susceptible to failure. Careful considerations must be made if the Kalihi Bridge becomes submerged.
A summary of the computed capacities of each bridge span is provided in Table 4.4-16.

Table 4.4-16: Kalihi Bridge Structural Resistance to Wave Loads

<table>
<thead>
<tr>
<th>Typical Span Resistance</th>
<th>Capacity (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Vertical Resistance (self weight)</td>
<td>565.09</td>
</tr>
<tr>
<td>Total Lateral Resistance (friction)</td>
<td>452.07</td>
</tr>
<tr>
<td>Overturning Moment Resistance</td>
<td>24,958.14 kip-ft</td>
</tr>
</tbody>
</table>
4.4.11 Nimitz Highway at Aloha Tower Slip Cover #2 and Slip Cover #3

Slip Cover #2 and Slip Cover #3 are located in the southern region of Oahu and have a west-east orientation (see Figure 4-51). The slip covers are sheltered within a small harbor area and are part of Nimitz Highway. Nimitz Highway is a high traffic roadway that provides the main route from Honolulu International Airport, through Honolulu’s business area, industrial area, and most of Honolulu Harbor. Slip Cover #2 and Slip Cover #3 are comprised of 10 and 12 separate spans respectively. For both slip covers, the spans are not significantly attached to one another. Therefore the spans are structurally analyzed individually.

![Figure 4-51: Nimitz Highway Slip Covers #2 & #3 Map Location](image)

The slip covers are constructed of a reinforced concrete deck supported by reinforced concrete pier caps. The deck is connected to the pier caps using ¾ inch diameter by 2 foot long dowels spaced through the entire length of the pier caps. The concrete caps have been constructed with slots, which allow the foundation piles to rest within the caps. However, the caps are not connected to the foundation piles.
The pier caps run in a north south orientation (see Figure 4-52 and Figure 4-53); that is, they run parallel to the wave direction. In addition, the geometry and configuration of the slip covers decks allows air to escape from underneath when inundated. This reduces the possibility of the slip covers becoming buoyant.

Figure 4-52: Nimitz Highway Slip Cover #2

Figure 4-53: Nimitz Highway Slip Cover #3
A sea wall prevents waves from passing completely under the decks of the slip covers. As waves strike this sea wall, the wave heights will increase 2.0 to 2.3 times of the original wave height (see Figure 4-54). Therefore, even if the waves are too low to originally impact the bridge deck, the redirected wave will still cause an upward force on the far edge of the slip covers. Because the slip covers are not restrained vertically, the upward force of the redirected waves may cause the bridge deck to blow out, causing failure.

![Wave Direction](image)

*Figure 4-54: Wave Direction*

**4.4.11.1 Nimitz Highway at Aloha Tower Slip Cover #2 and Slip Cover #3: Vertical Resistance**

Nimitz Highway at Aloha Tower Slip Covers #2 and #3 are both gravity type structures, where the only source of vertical resistance is provided by self weight. The self weight of each slip cover is the sum of the bridge deck, road way, side walk, and pier caps. The self weight of each of the 10 spans of Slip Cover #2 is computed to be 206.5 kips. Similarly, the 12 spans of Slip Cover #3 are each computed to weigh 218 kips. The average associated overturning moment due to the self weight of each span of Slip Cover #2 and #3 are computed to be 12,390 kip-ft, and 7,450 kip-ft respectively.
4.4.11.2  **Nimitz Highway at Aloha Tower Slip Cover #2 and Slip Cover #3: Horizontal Resistance**

In the horizontal direction, each slip cover is restrained by existing sea walls and roadways. This provides a large resistance to lateral wave loads. Therefore, the exact lateral resistance of each slip cover was not calculated.

4.4.11.3  **Discussion of Nimitz Highway at Aloha Tower**

Slip covers #2 and #3 are both gravity type structures, where the only source of vertical resistance is provided by the self weight of each bridge. Therefore the most likely failure mechanism of these slip covers will be due to uplift. The energy of each wave will be directed upward once the waves strike the sea wall. This will cause uplift wave forces and pressures to develop. The only means for the pressure to be relieved is for the slip covers to displace upward. With repeated wave impacts, the recurring uplift loads could cause the bridge decks to fail.

This type of failure was observed during a post-disaster survey of Japan after the Tohoku Tsunami of March 11, 2011. Many of the ports were constructed with access panels, which were placed between pile supported structures and a landside sea wall. When the tsunami impacted the port area the access panels acted as blow out panels. This allowed pressures under the structure to dissipate [19].
It is possible that the slip covers will behave in a similar fashion as the access panels described above. However, in the situation of the Japan ports, the failure of the panels relieved pressure and saved surrounding structures. If the slip covers blow out and fail due to a similar build up in pressure, then the Nimitz Highway will be unusable and cause major traffic disruptions.
4.5 Bridge Superstructure Capacity

As wave loads impact a bridge, two possible modes of failure can occur. The first type of failure is displacement of the bridge off of its foundation, which was previously analyzed in Section 4.4. The second mode of failure is the destruction of the bridge superstructure itself, which will primarily be caused by upward wave loads impacting the bottom of the bridge deck. The upward wave load will cause negative bending to occur in the deck and girders of the bridge.

The capacity of a bridge superstructure has been broken into three calculations. The first calculation analyzes the bridge deck as a continuous one-way slab (Section 4.5.1). The second calculation computes the strength of the connection between the deck and the bridge girders. The third calculation determines the negative bending capacity of the bridge girders (Section 4.5.3).

If the estimated upward wave load is greater than any of the calculated bridge capacities, then the bridge superstructure is at risk of failing.
4.5.1 Bridge Deck Capacity

The main source of damage to bridge decks will result from upward wave loads. Waves are assumed to impact the entire underside of the bridge, which will produce a distributed force. If a distributed wave load is able to cause a negative moment greater than the negative moment capacity of the deck, then the bridge will fail. Therefore, as part of the structural evaluation, the negative moment capacity of each bridge is computed.

The majority of the bridge decks are composed of reinforced concrete. The deck span between the bridge girders is analyzed as a continuous one way slab. Table 4.5-1 and Table 4.5-2 summarize the dimensions and reinforcement used in the construction of each bridge deck, respectively. The negative moment capacities can be found in Table 4.5-3.

The negative moment capacity is used to back calculate a distributed upward load. However, in order for a negative moment to develop, the upward wave load must be greater than the self weight of the bridge (see Figure 4-56). Therefore, the net upward wave load that will cause a moment greater than the negative moment capacity of the bridge deck, is the sum of the back calculated load plus the self weight of the deck slab. The result of this calculation for each bridge is in column 4 of Table 4.5-3.

![Figure 4-56: Loads Acting on Bridge Decks](image)

Figure 4-56: Loads Acting on Bridge Decks
## Bridge Deck Properties:

<table>
<thead>
<tr>
<th>Bridge Name</th>
<th>Deck Thickness (in.)</th>
<th>Span Width (in.)</th>
<th>Span Length (in.)</th>
<th>Top Concrete Cover (in.)</th>
<th>Bottom Concrete Cover (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kuliouou Bridge: (widening)</td>
<td>8.00</td>
<td>51.00</td>
<td>580.75</td>
<td>1.50</td>
<td>1.25</td>
</tr>
<tr>
<td>Kuliouou Bridge: (existing)</td>
<td>8.00</td>
<td>88.00</td>
<td>580.75</td>
<td>1.50</td>
<td>1.50</td>
</tr>
<tr>
<td>Kahaluu Stream Bridge:</td>
<td>6.00</td>
<td>49.00</td>
<td>3816.00</td>
<td>1.50</td>
<td>1.00</td>
</tr>
<tr>
<td>New South Punalu Bridge:</td>
<td>6.50</td>
<td>59.63</td>
<td>792.00</td>
<td>3.00</td>
<td>1.00</td>
</tr>
<tr>
<td>Ukoa Pond Bridge:</td>
<td>6.50</td>
<td>60.00</td>
<td>1080.00</td>
<td>1.50</td>
<td>1.25</td>
</tr>
<tr>
<td>New Makaha Stream #3A Bridge:</td>
<td>5.50</td>
<td>58.00</td>
<td>840.00</td>
<td>2.50</td>
<td>1.75</td>
</tr>
<tr>
<td>Old Makaha Stream #3A Bridge: (Wood Deck)</td>
<td>6.00</td>
<td>394.50</td>
<td>946.00</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Maipalaoa Bridge: (widening)</td>
<td>6.00</td>
<td>314.00</td>
<td>1208.00</td>
<td>1.50</td>
<td>0.75</td>
</tr>
<tr>
<td>Maipalaoa Bridge: (existing)</td>
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<td>458.00</td>
<td>1208.00</td>
<td>1.50</td>
<td>1.00</td>
</tr>
<tr>
<td>Moanalua Bridge:</td>
<td>8.00</td>
<td>72.00</td>
<td>324.00</td>
<td>2.00</td>
<td>1.50</td>
</tr>
<tr>
<td>Kalihi Bridge:</td>
<td>8.00</td>
<td>66.00</td>
<td>324.00</td>
<td>2.00</td>
<td>1.50</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2:</td>
<td>14.00</td>
<td>206.00</td>
<td>720.00</td>
<td>2.00</td>
<td>2.00</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3:</td>
<td>14.00</td>
<td>206.00</td>
<td>410.40</td>
<td>2.00</td>
<td>2.00</td>
</tr>
<tr>
<td>Bridge Name</td>
<td>Concrete fc' (psi)</td>
<td>Reinforcing Steel fy (psi)</td>
<td>Top Reinforcement</td>
<td>Bottom Reinforcement</td>
<td>Stirrups</td>
</tr>
<tr>
<td>----------------------------------</td>
<td>-------------------</td>
<td>----------------------------</td>
<td>-------------------</td>
<td>----------------------</td>
<td>---------------------</td>
</tr>
<tr>
<td>Kuliouou Bridge: (widening)</td>
<td>3000</td>
<td>40000</td>
<td>#4 @ 18” o.c.</td>
<td>#4 @ 8” o.c.</td>
<td>none</td>
</tr>
<tr>
<td>Kuliouou Bridge: (existing)</td>
<td>3000</td>
<td>40000</td>
<td>7 - #3</td>
<td>11 - #4</td>
<td>#4 @ 10” o.c.</td>
</tr>
<tr>
<td>Kahaluu Stream Bridge:</td>
<td>4000</td>
<td>60000</td>
<td>3 - #6</td>
<td>3 - #6</td>
<td>none</td>
</tr>
<tr>
<td>New South Punaluu Bridge:</td>
<td>9000</td>
<td>270000</td>
<td>10 - #5</td>
<td>prestressing</td>
<td>#4 @ 12” o.c.</td>
</tr>
<tr>
<td>Ukoa Pond Bridge:</td>
<td>3750</td>
<td>60000</td>
<td>6 - #5, 2 - #10</td>
<td>8 - #5</td>
<td>none</td>
</tr>
<tr>
<td>New Makaha Stream #3A Bridge:</td>
<td>8000</td>
<td>270000</td>
<td>4 - #7</td>
<td>4 - #8, 53 - prestress</td>
<td>#5 at edge @ 3” o.c.</td>
</tr>
<tr>
<td>Old Makaha Stream #3A Bridge:</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Maipalaoa Bridge: (widening)</td>
<td>3000</td>
<td>40000</td>
<td>14 - #7, 12 - #4</td>
<td>42 - #4</td>
<td>none</td>
</tr>
<tr>
<td>Maipalaoa Bridge: (existing)</td>
<td>3000</td>
<td>40000</td>
<td>#4 @ 15” o.c.</td>
<td>#5 @ 12” o.c.</td>
<td>none</td>
</tr>
<tr>
<td>Moanalua Bridge:</td>
<td>3000</td>
<td>40000</td>
<td>3 - #4</td>
<td>5 - #5</td>
<td>#5 @ 12” o.c.</td>
</tr>
<tr>
<td>Kalihi Bridge:</td>
<td>3000</td>
<td>40000</td>
<td>3 - #4</td>
<td>5 - #5</td>
<td>#5 @ 12” o.c.</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2:</td>
<td>3500</td>
<td>40000</td>
<td>14 - 1/2” dia.</td>
<td>15 - 5/8” dia.</td>
<td>none</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3:</td>
<td>3500</td>
<td>40000</td>
<td>15 - 1/2” dia.</td>
<td>15 - 5/8” dia.</td>
<td>none</td>
</tr>
<tr>
<td>Bridge Name</td>
<td>Positive Moment Capacity Mn (k-ft)</td>
<td>Negative Moment Capacity Mn (k-ft)</td>
<td>Positive load capacity (kips)</td>
<td>Negative load capacity (kips)</td>
<td>Bridge Vertical Resistance (kips)</td>
</tr>
<tr>
<td>-----------------------------------</td>
<td>-----------------------------------</td>
<td>-----------------------------------</td>
<td>-------------------------------</td>
<td>-------------------------------</td>
<td>----------------------------------</td>
</tr>
<tr>
<td>Kuliouou Bridge: (widening)</td>
<td>25.26</td>
<td>12.32</td>
<td>67.55</td>
<td>756.71</td>
<td>342.68</td>
</tr>
<tr>
<td>Kuliouou Bridge: (existing)</td>
<td>44.40</td>
<td>15.87</td>
<td>68.80</td>
<td>740.23</td>
<td>471.88</td>
</tr>
<tr>
<td>Kahaluu Stream Bridge:</td>
<td>28.96</td>
<td>25.65</td>
<td>8.21</td>
<td>3819.40</td>
<td>3811.55</td>
</tr>
<tr>
<td>New South Punaluu Bridge:</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>New South Punaluu Bridge: (Prestressed)</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Ukoa Pond Bridge:</td>
<td>59.50</td>
<td>93.45</td>
<td>51.83</td>
<td>2752.11</td>
<td>3042.3</td>
</tr>
<tr>
<td>New Makaha Stream #3A Bridge:</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>New Makaha Stream #3A Bridge:</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Old Makaha Stream #3A Bridge:</td>
<td>118.56</td>
<td>118.56</td>
<td>12.02</td>
<td>303.37</td>
<td>349.13</td>
</tr>
<tr>
<td>Maipalaoa Bridge: (widening)</td>
<td>134.13</td>
<td>145.09</td>
<td>26.21</td>
<td>600.50</td>
<td>572.15</td>
</tr>
<tr>
<td>Maipalaoa Bridge: (existing)</td>
<td>156.51</td>
<td>72.95</td>
<td>30.58</td>
<td>848.79</td>
<td>834.54</td>
</tr>
<tr>
<td>Moanalua Bridge:</td>
<td>31.10</td>
<td>11.37</td>
<td>98.79</td>
<td>453.39</td>
<td>417.27</td>
</tr>
<tr>
<td>Kalihi Bridge:</td>
<td>31.02</td>
<td>11.36</td>
<td>147.60</td>
<td>619.13</td>
<td>565.09</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2:</td>
<td>178.80</td>
<td>108.80</td>
<td>23.84</td>
<td>221.01</td>
<td>206.5</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3:</td>
<td>178.80</td>
<td>116.52</td>
<td>41.82</td>
<td>245.26</td>
<td>218</td>
</tr>
</tbody>
</table>
4.5.2 Bridge Deck Capacity: Analysis of Results

It should be noted that during the calculation of the negative load capacities, the bridge deck was assumed to be simply supported, where the maximum moment is calculated by $wL^2/8$, where ‘$w$’ is the distributed load and ‘$L$’ is the length of the bridge deck. In reality, the bridges are fixed at both ends, where the moment at the center is computed by $wL^2/24$. As an effect, the true bridge deck capacities will be greater than the values given in Table 4.5-3. Therefore, the listed capacities are conservative.

Column 4 of Table 4.5-3 summarizes the resultant load necessary to exceed the negative moment capacity of each bridge. Column 5 lists each bridge’s vertical resistance to displacement by wave loads, which were calculated from previous sections. Column 6 compares the negative load capacity (column 4) to the bridge vertical resistance (column 5). If column 4 is found to be greater than column 5, then it is more likely that the bridge will fail via displacement. If the opposite is true, then the bridge deck is more likely to fail due to a wave load causing a negative moment greater than its capacity.

From column 6 of Table 4.5-3, it is observed that in most cases, it is more likely a bridge will be displaced before its deck fails. Most of the analyzed bridges are gravity type structures, where the main source of vertical resistance is provided by the bridge self weight. As described in Section 4.5.1, the negative load capacity is determined by taking the sum of the self weight and the back calculated load. Therefore, if the self weight is the only source of vertical resistance to wave loads, then the negative load capacity will always be greater. Thus, the Kuliouou Bridge, the Kahaluu Bridge, the Maipalaoa Bridge, the Kalihi Bridge, and Slip Covers #2 and #3 are more likely to be displaced by vertical wave loads.

The Old Makaha #3A Bridge is determined to fail via deck failure before being displaced. The bridge has additional vertical resistance supplementing the resistance provided by the bridge self weight. This causes the vertical resistance of the bridge to be greater than the negative load capacity. Thus, if an upward wave load is not large
enough to displace the Old Makaha #3A Bridge, then the bridge deck may fail due to a negative moment.

4.5.3 Bridge Negative Bending Capacity

The majority of the analyzed girders are comprised of prestressed concrete. The upward force produced by the prestressed tendons is of concern when the self weight of the bridge is reduced due to hydrostatic buoyancy forces. The combination of the reduction in self weight, upward storm wave forces and prestressing effect may ultimately cause the girders to fail in negative bending. If the girders fail, then so will the entire bridge.

The negative bending capacity of a girder is calculated under two conditions. The first case considered is when the bridge is not submerged and the full self weight of the bridge is present. The second case is when the bridge is submerged to the top of the deck. In the latter situation, buoyancy forces are included and a non-conservative 50% air pocket volume is assumed.

To determine the negative bending capacities of the prestressed girders, the same process developed in Section 4.4.6.3 is followed.

To determine the negative bending capacity of the reinforced concrete girders, the negative moment capacity of each girder is calculated and then converted into an equivalent distributed load. The self weight of the tributary area of each girder is then added to the negative moment capacity load.

Table 4.5-4 summarizes the girder properties and Table 4.5-5 summarizes the computed negative bending capacities.
<table>
<thead>
<tr>
<th>Bridge Description</th>
<th>Concrete $f'c$ (psi)</th>
<th>Girder Length (ft)</th>
<th>Number of Girders</th>
<th>Prestressing Strands</th>
<th>Pe (lbs)</th>
<th>Total Dead Load (unsubmerged) (lb/in)</th>
<th>Total Dead Load (submerged) (lb/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kuliouou Bridge: (widening)</td>
<td>3000</td>
<td>48.39</td>
<td>8</td>
<td>24 - 7/16&quot; dia</td>
<td>317520.00</td>
<td>75.78</td>
<td>11.98</td>
</tr>
<tr>
<td>Kahaluu Stream Bridge:</td>
<td>4000</td>
<td>106.00</td>
<td>24</td>
<td>40 - 1/2&quot; dia</td>
<td>1156680.00</td>
<td>113.54</td>
<td>18.06</td>
</tr>
<tr>
<td>Ukoa Pond Bridge: (span 1) Keahi Type IV Girder</td>
<td>4500</td>
<td>90.00</td>
<td>7</td>
<td>1/2&quot; dia</td>
<td>793400</td>
<td>121.10</td>
<td>22.55</td>
</tr>
<tr>
<td>Ukoa Pond Bridge: (span 2, 3, &amp; 4) AASHTO Type III Girder</td>
<td>4000</td>
<td>60.00</td>
<td>21</td>
<td>1/2&quot; dia</td>
<td>636200</td>
<td>109.96</td>
<td>16.16</td>
</tr>
<tr>
<td>Maipalaoa Bridge: (widening)</td>
<td>6000</td>
<td>50.00</td>
<td>14</td>
<td>12 - 1/2&quot; dia</td>
<td>211680.00</td>
<td>62.29</td>
<td>35.71</td>
</tr>
<tr>
<td>Maipalaoa Bridge: (existing)</td>
<td>6000</td>
<td>50.00</td>
<td>18</td>
<td>12 - 1/2&quot; dia</td>
<td>211680.00</td>
<td>62.29</td>
<td>35.71</td>
</tr>
<tr>
<td>Moanalua Bridge:</td>
<td>3000</td>
<td>8.00</td>
<td>9</td>
<td>none</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Kalihi Bridge:</td>
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<td>8.00</td>
<td>13</td>
<td>none</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2:</td>
<td>3500</td>
<td>60.00</td>
<td>11</td>
<td>none</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3:</td>
<td>3500</td>
<td>32.50</td>
<td>13</td>
<td>none</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Bridge Description</td>
<td>Negative Bending (unsubmerged)</td>
<td>Negative Bending (submerged)</td>
<td>Loss in capacity once submerged</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>------------------------------------------</td>
<td>--------------------------------</td>
<td>-------------------------------</td>
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<td>Compression</td>
<td>Tensile</td>
<td>Compression</td>
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</tr>
<tr>
<td>Kuliouou Bridge:</td>
<td>63.49</td>
<td>194.71</td>
<td>26.45</td>
<td>157.66</td>
<td>58.34%</td>
<td>19.03%</td>
<td></td>
</tr>
<tr>
<td>Kahaluu Stream Bridge:</td>
<td>131.13</td>
<td>236.24</td>
<td>9.67</td>
<td>114.78</td>
<td>92.62%</td>
<td>51.41%</td>
<td></td>
</tr>
<tr>
<td>Ukoa Pond Bridge: (span 1)</td>
<td>136.20</td>
<td>230.82</td>
<td>11.25</td>
<td>105.87</td>
<td>91.74%</td>
<td>54.13%</td>
<td></td>
</tr>
<tr>
<td>Keehi Type IV Girder</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ukoa Pond Bridge: (span 2, 3, &amp; 4)</td>
<td>66.46</td>
<td>112.81</td>
<td>-1.07</td>
<td>45.28</td>
<td>101.61%</td>
<td>59.86%</td>
<td></td>
</tr>
<tr>
<td>AASHTO Type III Girder</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maipalaoa Bridge: (widening)</td>
<td>80.70</td>
<td>147.75</td>
<td>64.76</td>
<td>131.80</td>
<td>19.76%</td>
<td>10.80%</td>
<td></td>
</tr>
<tr>
<td>Maipalaoa Bridge: (existing)</td>
<td>80.70</td>
<td>147.75</td>
<td>64.76</td>
<td>131.80</td>
<td>19.76%</td>
<td>10.80%</td>
<td></td>
</tr>
<tr>
<td>Moanalua Bridge:</td>
<td>68.43</td>
<td>68.43</td>
<td>39.08</td>
<td>39.08</td>
<td>42.89%</td>
<td>42.89%</td>
<td></td>
</tr>
<tr>
<td>Kalihi Bridge:</td>
<td>65.54</td>
<td>65.54</td>
<td>42.58</td>
<td>42.58</td>
<td>35.04%</td>
<td>35.04%</td>
<td></td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2:</td>
<td>55.86</td>
<td>55.86</td>
<td>47.83</td>
<td>47.83</td>
<td>14.37%</td>
<td>14.37%</td>
<td></td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3:</td>
<td>82.36</td>
<td>82.36</td>
<td>76.41</td>
<td>76.41</td>
<td>7.22%</td>
<td>7.22%</td>
<td></td>
</tr>
</tbody>
</table>
4.5.4 Bridge Negative Bending Capacity: Analysis of Results

Columns 1 and 2 of Table 4.5-5 list the tensile and compressive stress limit negative bending capacities of each girder when not submerged. Columns 3 and 4 list the tensile and compressive stress limit negative bending capacities when the bridge is submerged. Lastly, columns 5 and 6 list the reduction in the negative bending capacities once the bridge becomes submerged. Again it is observed that the tensile stress limit capacity controls the overall negative bending capacity of each prestressed girder.

Through the analysis of the prestressed girders, it is determined that only the AASHTO Type III girder used to support the Ukoa Pond Bridge deck are vulnerable to failure if the bridge is submerged. With a 50% air pocket, the buoyancy force is not large enough to cause the bridge to become fully buoyant. However, the upward buoyancy force is able to reduce the self weight of the bridge enough to cause the prestressing term in Equation 4.4-4 to be greater than the sum of $M_D$ and $S_f^* f_t$. This causes a negative capacity value of -1.07 kips, which indicates that the tensile stress limit will be exceeded if the bridge is submerged. As a result, concrete cracks may begin to develop, which can lead to failure of the girders. However, this type of failure will be resisted by the tension reinforcement in the girders. Therefore, the bridge girders are at risk of failing, but are by no means going to fail immediately under the aforementioned conditions.

As waves impact the underside of the bridge deck, only a portion of the entire wave load will affect a single girder. Therefore, the percentage of the wave load applied to each girder is taken as the tributary area of the girder divided by the plan view cross sectional area of the bridge deck. This results in a distributed load. The resultant of the distributed load is compared to the negative bending capacity of the girders. This comparison is made in Chapter 6.
4.5.5 Bridge Superstructure Capacity: Notes

The Kuliouou Bridge, the Kahaluu Bridge, the Ukoa Pond Bridge, and the Maipalaoa Bridge prestressed girders are connected to the bridge decks using #4 and #5 stirrups. The stirrups are provided along the entire length of the girders. The stirrups cause the bridge deck and girders to behave as a composite. It was determined that the strength of the connection between the bridge deck and girders is not the limiting capacity of the bridge superstructure.

The lateral capacities of the bridge superstructures were also considered. However, it was found that the bridge decks will fail due to an upward wave load before failing due to lateral forces. Therefore, the lateral calculations were not included in this thesis.
4.6 Summary of Structural Capacity Calculations

Table 4.6-1 summarizes the structural capacities calculated in Chapter 4. Columns 1, 2, and 3 summarize the resistance to displacement of each bridge. Column 4 lists the negative moment capacity of the bridge decks. Column 5 lists the total negative bending capacity of all girders for a particular bridge and is labeled as ‘Bridge Negative Bending Capacity.’
<table>
<thead>
<tr>
<th>Bridge Description</th>
<th>Lateral Resistance (kips)</th>
<th>Vertical Resistance (kips)</th>
<th>Overturning Resistance (kip-ft)</th>
<th>Negative Moment Deck Capacity (kips)</th>
<th>Negative Moment Bridge Capacity (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kahaluu Bridge:</td>
<td>518.1</td>
<td>3811.6</td>
<td>725.8</td>
<td>9799.8</td>
<td>333.8</td>
</tr>
<tr>
<td>Kalihi Stream Bridge:</td>
<td>1823.9</td>
<td>87665.7</td>
<td>NA</td>
<td>1263.0</td>
<td>1744.0</td>
</tr>
<tr>
<td>New South Punalu Bridge: (span #1)</td>
<td>672.8</td>
<td>1263.0</td>
<td>NA</td>
<td>2752.1</td>
<td>263.4</td>
</tr>
<tr>
<td>Ukoa Pond Bridge:</td>
<td>NA</td>
<td>3042.3</td>
<td>NA</td>
<td>349.1</td>
<td>1406.7</td>
</tr>
<tr>
<td>New Makaha Stream #3A Bridge:</td>
<td>70.4</td>
<td>1127.1</td>
<td>9799.8</td>
<td>3042.3</td>
<td>1406.7</td>
</tr>
<tr>
<td>Old Makaha Stream #3A Bridge:</td>
<td>333.8</td>
<td>417.3</td>
<td>333.8</td>
<td>5731.6</td>
<td>333.8</td>
</tr>
<tr>
<td>Maipalaoa Bridge:</td>
<td>263.4</td>
<td>1406.7</td>
<td>263.4</td>
<td>45248.9</td>
<td>263.4</td>
</tr>
<tr>
<td>Maunalua Bridge: (single span)</td>
<td>333.8</td>
<td>417.3</td>
<td>333.8</td>
<td>45248.9</td>
<td>333.8</td>
</tr>
<tr>
<td>Kalii Bridge:</td>
<td>452.1</td>
<td>565.1</td>
<td>452.1</td>
<td>24958.1</td>
<td>452.1</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2:</td>
<td>1655.6</td>
<td>206.9</td>
<td>1655.6</td>
<td>12390.0</td>
<td>1655.6</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3:</td>
<td>1744.0</td>
<td>181.7</td>
<td>1744.0</td>
<td>7448.3</td>
<td>1744.0</td>
</tr>
<tr>
<td>New Makaha Stream #3A Bridge: (span #2)</td>
<td>333.8</td>
<td>417.3</td>
<td>333.8</td>
<td>5731.6</td>
<td>333.8</td>
</tr>
<tr>
<td>Kahaluu Bridge:</td>
<td>518.1</td>
<td>3811.6</td>
<td>725.8</td>
<td>9799.8</td>
<td>333.8</td>
</tr>
<tr>
<td>Kalihi Stream Bridge:</td>
<td>1823.9</td>
<td>87665.7</td>
<td>NA</td>
<td>1263.0</td>
<td>1744.0</td>
</tr>
<tr>
<td>New South Punalu Bridge: (span #2)</td>
<td>672.8</td>
<td>1263.0</td>
<td>NA</td>
<td>2752.1</td>
<td>263.4</td>
</tr>
<tr>
<td>Ukoa Pond Bridge:</td>
<td>NA</td>
<td>3042.3</td>
<td>NA</td>
<td>349.1</td>
<td>1406.7</td>
</tr>
<tr>
<td>New Makaha Stream #3A Bridge:</td>
<td>70.4</td>
<td>1127.1</td>
<td>9799.8</td>
<td>3042.3</td>
<td>1406.7</td>
</tr>
<tr>
<td>Old Makaha Stream #3A Bridge:</td>
<td>333.8</td>
<td>417.3</td>
<td>333.8</td>
<td>5731.6</td>
<td>333.8</td>
</tr>
<tr>
<td>Maipalaoa Bridge:</td>
<td>263.4</td>
<td>1406.7</td>
<td>263.4</td>
<td>45248.9</td>
<td>263.4</td>
</tr>
<tr>
<td>Maunalua Bridge: (single span)</td>
<td>333.8</td>
<td>417.3</td>
<td>333.8</td>
<td>45248.9</td>
<td>333.8</td>
</tr>
<tr>
<td>Kalii Bridge:</td>
<td>452.1</td>
<td>565.1</td>
<td>452.1</td>
<td>24958.1</td>
<td>452.1</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2:</td>
<td>1655.6</td>
<td>206.9</td>
<td>1655.6</td>
<td>12390.0</td>
<td>1655.6</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3:</td>
<td>1744.0</td>
<td>181.7</td>
<td>1744.0</td>
<td>7448.3</td>
<td>1744.0</td>
</tr>
</tbody>
</table>
4.7 Limitation of Structural Capacity Calculations:

In the preceding structural analyses, the bridges were assumed to be horizontal. In reality most of the bridge decks have a slight drainage slope. Due to this slope, a small portion of the horizontal wave component will likely impact the top of the deck. If this occurs, a downward force will develop on the bridge, which may counteract a portion of the upward wave force.

Also, the increase in concrete strength due to concrete curing over time was not considered. As concrete ages, the strength of the concrete will continuously increase. For example, some bridges built in the 1940’s have concrete strengthened over an approximately 70 year period. However, the capacity calculations completed in this thesis uses the concrete strength listed on the as-built plans provided by the HDOT. If considered, the calculated capacities will increase. The increase in concrete strength will vary from bridge to bridge.

However, by not including the two abovementioned limitations, the resulting calculations in Chapter 4 are conservative and therefore provide a lower bound for the wave forces needed to cause deck failure, girder failure, or bridge superstructure displacement.
5 Wave Force Estimation

5.1 Determination of Still Water Level Including Storm Surge

The still water level including the storm surge for a 100-year storm were determined from flood insurance studies conducted by the Federal Emergency Management Agency (FEMA) on the island of Oahu [20]. The flood elevations ascertained from the FEMA reports are known as the “base flood elevations,” which is comprised of both the still water level, which includes the storm surge, and the wave setup. The base flood elevations are measured from the ground elevation to the highest point on the waves (see Figure 5-1).

In order to separate the still water level from the wave amplitude, the base flood elevation values are used to back calculate the needed information. Based on the AASHTO Guide Specifications [9], the wave height is determined by taking 65% of the still water level ($d_s$). Only 70% of the resulting wave height is above the still water level, which is typically denoted as the “crest” of the wave ($\eta$). The remaining 30% is the “trough” of the wave and is below the still water level (see Figure 5-1). Based on these wave height characteristics, the following equations were developed to determine the wave heights and still water level from the base flood elevations:

$$d_s = \frac{\text{base flood elevation}}{(1+0.65*0.70)} \quad (5.1-1)$$

$$\eta = \text{base flood elevation} - d_s \quad (5.1-2)$$

in which

- $d_s$ = still water level (including storm surge)
- $\eta$ = wave amplitude above the storm surge depth

The FEMA data are summarized in Table 5.1-1. Table 5.1-2 compares the elevations of each bridge to the calculated still water levels (SWL). If the SWL is greater than the elevation of the top of the deck, then the bridge will be submerged during a 100-year storm event.
Figure 5-1: Wave Profile
### Table 5.1-1: Data from FEMA Flood Insurance Study for a 100-year Storm

<table>
<thead>
<tr>
<th>Bridge</th>
<th>Base Flood Elevation (ft)</th>
<th>Still Water Level with Storm Surge: (ft)</th>
<th>Wave height above surge elevation: (ft)</th>
<th>Elevation to bottom of bridge deck (ft)</th>
<th>Elevation to top of bridge deck (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kaliouou Bridge</td>
<td>8.00</td>
<td>5.50</td>
<td>2.50</td>
<td>7.94</td>
<td>8.61</td>
</tr>
<tr>
<td>Kahaluu Stream Bridge</td>
<td>16.00</td>
<td>11.00</td>
<td>5.01</td>
<td>15.25</td>
<td>15.75</td>
</tr>
<tr>
<td>New South Punaluu Bridge</td>
<td>12.00</td>
<td>8.25</td>
<td>3.75</td>
<td>6.67</td>
<td>7.55</td>
</tr>
<tr>
<td>Ukoa Pond Bridge</td>
<td>No data*</td>
<td>No data*</td>
<td>No data*</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>New Makaha Stream #3A Bridge</td>
<td>13.00</td>
<td>8.93</td>
<td>4.06</td>
<td>10.59</td>
<td>11.05</td>
</tr>
<tr>
<td>Old Makaha Stream #3A Bridge</td>
<td>13.00</td>
<td>8.93</td>
<td>4.06</td>
<td>12.74</td>
<td>13.24</td>
</tr>
<tr>
<td>Maipalaoa Bridge</td>
<td>12.00</td>
<td>8.25</td>
<td>3.75</td>
<td>9.50</td>
<td>10.00</td>
</tr>
<tr>
<td>Moanalua Bridge</td>
<td>10.00</td>
<td>6.87</td>
<td>3.13</td>
<td>8.33</td>
<td>9.00</td>
</tr>
<tr>
<td>Kalihi Bridge</td>
<td>10.00</td>
<td>6.87</td>
<td>3.13</td>
<td>8.33</td>
<td>9.00</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2</td>
<td>8.00</td>
<td>5.50</td>
<td>2.50</td>
<td>12.33</td>
<td>13.63</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3</td>
<td>8.00</td>
<td>5.50</td>
<td>2.50</td>
<td>12.33</td>
<td>13.63</td>
</tr>
</tbody>
</table>

### Table 5.1-2: Comparison of Still Water Level to Bridge Elevations

<table>
<thead>
<tr>
<th>Bridge</th>
<th>Still Water Level with Storm Surge: (ft)</th>
<th>Elevation to bottom of bridge deck (ft)</th>
<th>Elevation to top of bridge deck (ft)</th>
<th>Submerged?</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kaliouou Bridge</td>
<td>5.50</td>
<td>7.94</td>
<td>8.61</td>
<td>No</td>
</tr>
<tr>
<td>Kahaluu Stream Bridge</td>
<td>11.00</td>
<td>15.25</td>
<td>15.75</td>
<td>No</td>
</tr>
<tr>
<td>New South Punaluu Bridge</td>
<td>8.25</td>
<td>6.67</td>
<td>7.55</td>
<td>Yes</td>
</tr>
<tr>
<td>Ukoa Pond Bridge</td>
<td>No Data*</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>New Makaha Stream #3A Bridge</td>
<td>8.93</td>
<td>10.59</td>
<td>11.05</td>
<td>No</td>
</tr>
<tr>
<td>Old Makaha Stream #3A Bridge</td>
<td>8.93</td>
<td>12.74</td>
<td>13.24</td>
<td>No</td>
</tr>
<tr>
<td>Maipalaoa Bridge</td>
<td>8.25</td>
<td>9.50</td>
<td>10.00</td>
<td>No</td>
</tr>
<tr>
<td>Moanalua Bridge</td>
<td>6.87</td>
<td>8.33</td>
<td>9.00</td>
<td>No</td>
</tr>
<tr>
<td>Kalihi Bridge</td>
<td>6.87</td>
<td>8.33</td>
<td>9.00</td>
<td>No</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2</td>
<td>5.50</td>
<td>12.33</td>
<td>13.63</td>
<td>No</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3</td>
<td>5.50</td>
<td>12.33</td>
<td>13.63</td>
<td>No</td>
</tr>
</tbody>
</table>

* Outside of FEMA Flood Area
It should be noted that the still water level during a 100-year storm is above the deck of the New South Punaluu Bridge. Therefore, the methods used to estimate the wave forces in the following sections are not directly applicable. It is likely that the wave forces on the bridge deck will be smaller than the values calculated.

For the Nimitz Highway Slip Covers #2 and #3, a 4.33 foot clearance exists between the bottoms of the bridge decks to the highest point of the storm waves, indicating that storm waves will not directly impact the slip covers during a 100-year storm. However, as waves impact the existing sea wall at the far edge of the slip covers, the waves will be redirected upward. This will cause the wave heights to increase by 2.0 to 2.3 times of the original wave height, causing an upward force at the far edge of the slip covers. The wave estimation methods used in this thesis are not applicable for this situation and therefore, the estimated wave forces on the slip covers calculated in the subsequent sections were determined to be zero.
5.2 Douglass [10]

The Douglass’ wave estimation method was described in Chapter 2 and is as follows:

The wave forces are given by

\[ F_v = c_{v-va} * F_v^* \]  
\[ F_h = [1 + C_r * (N - 1)] * c_{h-va} * F_h^* \]  
\[ F_v^* = \gamma * (\Delta z_v) * A_v \]  
\[ F_h^* = \gamma * (\Delta z_h) * A_h \]

in which

- \( F_v \) = vertical wave-induced load
- \( F_h \) = horizontal wave-induced load
- \( F_v^* \) = a “reference” vertical load
- \( F_h^* \) = a “reference” horizontal load
- \( c_{v-va} \) = an empirical coefficient for the vertical “varying” load  
  (recommended value is \( c_{v-va} = 1 \) for non conservative, 2 for conservative design)
- \( c_{h-va} \) = an empirical coefficient for the horizontal “varying” load  
  (recommended value is \( c_{h-va} = 1 \) for non conservative, 2 for conservative design)
- \( C_r \) = a reduction coefficient for reduced horizontal load on the internal girders  
  (recommended value is \( C_r = 0.4 \))
- \( N \) = the number of girders supporting the bridge span deck
- \( \gamma \) = unit weight of salt water (10.06 kN/m\(^3\) or 64 lb/ft\(^3\))
\[
\Delta z_y = \text{difference between the elevation of the crest of the maximum wave and the elevation of the underside of the bridge deck (magnitude of inundation)}
\]

\[
A_y = \text{the area of the bridge contributing to vertical uplift, i.e. the projection of the bridge deck onto the horizontal plane}
\]

\[
\Delta z_h = \text{difference between the elevation of the crest of the maximum wave and the elevation of the centroid of } A_h
\]

\[
A_h = \text{the area of the projection of the bridge deck onto the vertical plane}
\]

*Figure 5-2: Douglass Wave Estimation Method Diagram [10]*

The necessary bridge dimensions needed for Douglass’ method are summarized in Table 5.2-1 and Table 5.2-2. The resulting wave forces are summarized in Table 5.2-3.
<table>
<thead>
<tr>
<th>Bridge Name</th>
<th>Number of Girders</th>
<th>Bridge Width (ft)</th>
<th>Bridge Length (ft)</th>
<th>Bridge Area Impacted by Vertical Force (sq ft)</th>
<th>Bridge Area Impacted by Lateral Force (sq ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kaliouou Bridge</td>
<td>12</td>
<td>68.75</td>
<td>48.40</td>
<td>3327.21</td>
<td>308.52</td>
</tr>
<tr>
<td>Kahalu Stream Bridge</td>
<td>8</td>
<td>46.00</td>
<td>318.00</td>
<td>14628.00</td>
<td>2265.75</td>
</tr>
<tr>
<td>New South Punalu Bridge: (span)</td>
<td>30</td>
<td>50.00</td>
<td>66.00</td>
<td>3300.00</td>
<td>371.25</td>
</tr>
<tr>
<td>Ukoa Pond Bridge</td>
<td>7</td>
<td>48.00</td>
<td>270</td>
<td>12960.00</td>
<td>2396.25</td>
</tr>
<tr>
<td>New Makalu Stream #3A Bridge</td>
<td>1</td>
<td>46.83</td>
<td>70</td>
<td>3278.33</td>
<td>277.08</td>
</tr>
<tr>
<td>Old Makaha Stream #3A Bridge</td>
<td>12</td>
<td>32.88</td>
<td>78.83</td>
<td>2591.65</td>
<td>157.67</td>
</tr>
<tr>
<td>Maipalaoa Bridge</td>
<td>16</td>
<td>64.33</td>
<td>100.67</td>
<td>6476.22</td>
<td>553.67</td>
</tr>
<tr>
<td>Moanalua Bridge: (single span)</td>
<td>9</td>
<td>64.33</td>
<td>27.00</td>
<td>1737.00</td>
<td>168.19</td>
</tr>
<tr>
<td>Kalihi Bridge: (single span)</td>
<td>13</td>
<td>88.33</td>
<td>27.00</td>
<td>2385.00</td>
<td>168.19</td>
</tr>
<tr>
<td>Nimtz Hwy. Slip Cover #2</td>
<td>11</td>
<td>Variable</td>
<td>Variable</td>
<td>10704.60</td>
<td>973.82</td>
</tr>
<tr>
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<td>13</td>
<td>Variable</td>
<td>Variable</td>
<td>8536.50</td>
<td>1310.00</td>
</tr>
</tbody>
</table>
Table 5.2-2: Bridge Dimensions Continued

<table>
<thead>
<tr>
<th>Bridge Name</th>
<th>Height of girders (ft)</th>
<th>Height of Deck (ft)</th>
<th>Height of Railing (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kuliouou Bridge:</td>
<td>3.00</td>
<td>0.67</td>
<td>2.71</td>
</tr>
<tr>
<td>Kahaluu Stream Bridge:</td>
<td>4.50</td>
<td>0.50</td>
<td>2.13</td>
</tr>
<tr>
<td>New South Punaluu Bridge:</td>
<td>1.75</td>
<td>0.88</td>
<td>3.00</td>
</tr>
<tr>
<td>Ukoa Pond Bridge:</td>
<td>4.83</td>
<td>0.54</td>
<td>3.50</td>
</tr>
<tr>
<td>New Makaha Stream #3A Bridge:</td>
<td>2.33</td>
<td>0.46</td>
<td>1.17</td>
</tr>
<tr>
<td>Old Makaha Stream #3A Bridge:</td>
<td>1.50</td>
<td>0.50</td>
<td>0.00</td>
</tr>
<tr>
<td>Maipalaoa Bridge:</td>
<td>3.00</td>
<td>0.50</td>
<td>2.00</td>
</tr>
<tr>
<td>Moanalua Bridge: (single span)</td>
<td>1.83</td>
<td>0.67</td>
<td>3.73</td>
</tr>
<tr>
<td>Kalihi Bridge: (single span)</td>
<td>1.83</td>
<td>0.67</td>
<td>3.73</td>
</tr>
<tr>
<td>(typical span)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2:</td>
<td>2.50</td>
<td>1.29</td>
<td>1.67</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3:</td>
<td>2.50</td>
<td>1.29</td>
<td>1.67</td>
</tr>
</tbody>
</table>
Table 5.2-3: Douglass Estimated Wave Forces

<table>
<thead>
<tr>
<th></th>
<th>Vertical Wave Force (kips)</th>
<th>Horizontal Wave Force (kips)</th>
<th>Associated Overturning Moment (kip-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kuliouou Bridge:</td>
<td>332.19</td>
<td>52.34</td>
<td>11585.83</td>
</tr>
<tr>
<td>Kahaluu Stream Bridge:</td>
<td>2808.58</td>
<td>929.86</td>
<td>67909.89</td>
</tr>
<tr>
<td>New South Punaluu Bridge:</td>
<td>1125.70</td>
<td>1277.59</td>
<td>31735.61</td>
</tr>
<tr>
<td>Ukoa Pond Bridge:</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>New Makaha Stream #3A Bridge:</td>
<td>505.65</td>
<td>7.64</td>
<td>11855.76</td>
</tr>
<tr>
<td>Old Makaha Stream #3A Bridge:</td>
<td>43.07</td>
<td>41.41</td>
<td>749.38</td>
</tr>
<tr>
<td>Maipalaoa Bridge:</td>
<td>1036.20</td>
<td>682.12</td>
<td>35206.78</td>
</tr>
<tr>
<td>Moanalua Bridge: (single span)</td>
<td>185.28</td>
<td>17.42</td>
<td>6014.11</td>
</tr>
<tr>
<td>Kalihi Bridge: (single span)</td>
<td>254.40</td>
<td>24.06</td>
<td>11310.94</td>
</tr>
<tr>
<td>Kalihi Bridge: (typical span)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2:</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3:</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Douglass’s method assumes that the wave load components act through their respective area centroids of the bridge superstructure (Figure 5-3). To estimate the largest overturning moment produced by the horizontal and vertical wave forces, the moment is computed at the far edge of the bridge superstructure. The results of this calculation are summarized in column 3 of Table 5.2-3.
A limitation of Douglass’ method was observed during the calculation of the wave forces. The equations used to compute the horizontal and vertical wave forces each have a term that takes the difference between the centroid of the bridge and the highest point on the wave. If the highest point on the wave crest is not above the elevation of the centroid then a negative force will result from the equations.

To adjust for the negative forces in the horizontal direction, the height from the still water level to the center elevation of the wave crest was taken as $\Delta z_h$, as shown in Figure 5-4.

To adjust for the negative forces in the vertical direction, the vertical area centroid was taken at the mid elevation of the bridge girders. This elevation corresponds to a 50% air pocket. This was done because the entrapped air will transfer the force of the waves to the bottom of the bridge deck [11]. See Figure 5-4 for the adjusted centroids.

The horizontal modification was only necessary for the Kuliouou Bridge. The vertical modification was done for the Kuliouou Bridge and the Kahaluu Bridge.
Figure 5-4: Adjusted Bridge Centroids for Douglass’ Method
5.3 McPherson [11]

McPherson’s wave estimation method was described in Chapter 2 and is as follows:

*To estimate the vertical wave force*

The vertical wave force is estimated by

\[
F_{Total} = F_{Hydrostatic} + F_{Bridge} + F_{AirEntrapment} \tag{2.4-1}
\]

\[
F_{Hydrostatic} = \gamma \delta z A - F_w \tag{2.4-1}
\]

\[
F_{Bridge} = \gamma Vol_{Bridge} \tag{2.4-2}
\]

\[
F_{AirEntrapment} = (n - 1)0.5\gamma \delta G A_G \tag{2.4-3}
\]

If \( h \leq h_{model} \),

\[
F_w = \frac{1}{2} \gamma \delta A \tag{2.4-4}
\]

And if \( h > h_{model} \),

\[
F_w = \frac{1}{2} \gamma \delta A + \gamma (h - h_{model}) A \tag{2.4-5}
\]

The horizontal wave force is estimated by

\[
F_{Total} = F_{Hydrostatic_{Front}} - F_{Hydrostatic_{Back}} \tag{2.4-6}
\]

If \( \eta_{max} < h_{deck} \),

\[
F_{Hydrostatic_{Front}} = 0.5 * (\eta_{max} + h - h_{girders}) H_{bridge} L_{bridge} \gamma \tag{2.4-7}
\]

And if \( \eta_{max} > h_{deck} \),

\[
F_{Hydrostatic_{Front}} = 0.5 * [(\eta_{max} + h - h_{girders}) + (\eta_{max} - h_{deck})] H_{bridge} L_{bridge} \gamma \tag{2.4-8}
\]
if \( SWL < h_{girders} \),

\[
F_{\text{Hydrostatic, back}} = 0
\]  \hspace{1cm} (2.4-9)

and if \( SWL > h_{girders} \),

\[
F_{\text{Hydrostatic, back}} = 0.5 (h - h_{girders})^2 L_{\text{bridge}} \gamma
\]  \hspace{1cm} (2.4-10)

In the above equations,

\( \gamma \) = unit weight of salt water [64 lb/ft\(^3\)]

\( \delta_z \) = distance from the top of the bridge deck to the top of the wave [ft]

\( \delta_G \) = height of the bridge girders [ft]

\( \delta \) = height of wave overtopping the bridge deck [ft]

\( A \) = area of bridge impacted by vertical wave force [ft\(^2\)]

\( A_G \) = cross sectional area of trapped air between girders [ft\(^2\)]

\( n \) = number of girders

\( h_{\text{model}} \) = distance from ground elevation to top of deck [ft]

\( h \) = height from the ground elevation to the top of the still water level [ft]

\( \eta_{\text{max}} \) = height of wave above the still water level [ft]

\( h_{girders} \) = height from the ground elevation to the bottom of the bridge girders [ft]

\( H_{\text{bridge}} \) = height of bridge impacted by lateral wave forces [ft]
\[ L_{\text{bridge}} = \text{length of bridge impacted by lateral wave forces [ft]} \]
\[ SWL = \text{still water level including storm surge [ft]} \]

In McPherson’s method, the resultants of the wave forces are also assumed to act through the area centroids of the bridge superstructure. Again, the associated overturning moment was computed at the far edge of each bridge. The moment caused by the horizontal and vertical wave forces are listed in column 3 of Table 5.3-1.
Table 5.3-1: McPherson Wave Force Estimation

<table>
<thead>
<tr>
<th>Bridge Description</th>
<th>Vertical Wave Force (kips)</th>
<th>Horizontal Wave Force (kips)</th>
<th>Associated Overturning Moment (kip-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kuliouou Bridge:</td>
<td>531.23</td>
<td>29.73</td>
<td>18355.91</td>
</tr>
<tr>
<td>Kahaluu Stream Bridge:</td>
<td>2876.79</td>
<td>380.03</td>
<td>67519.95</td>
</tr>
<tr>
<td>New South Punaluu Bridge:</td>
<td>1119.70</td>
<td>54.14</td>
<td>28144.71</td>
</tr>
<tr>
<td>Ukoa Pond Bridge:</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>New Makaha Stream #3A Bridge:</td>
<td>495.69</td>
<td>21.37</td>
<td>11649.65</td>
</tr>
<tr>
<td>Old Makaha Stream #3A Bridge:</td>
<td>518.46</td>
<td>8.88</td>
<td>8531.07</td>
</tr>
<tr>
<td>Maipalaoa Bridge:</td>
<td>1309.70</td>
<td>87.61</td>
<td>42369.57</td>
</tr>
<tr>
<td>Moanalua Bridge: (single span)</td>
<td>319.76</td>
<td>18.72</td>
<td>10343.86</td>
</tr>
<tr>
<td>Kalihi Bridge: (single span)</td>
<td>374.83</td>
<td>24.63</td>
<td>16631.87</td>
</tr>
<tr>
<td>(typical span)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2:</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3:</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>
5.4 AASHTO [9]

To properly use the AASHTO Guide Specifications, a design wave must be generated. To do so, Section 6.2.2.4 of the AASHTO guide is followed. To determine the design wave: the 100-year storm wind speed, wave development fetch length, and average water depth over the fetch length are needed.

The 100-year storm wind speed value is adjusted from a 50 year storm wind speed. The 50 year storm wind speed is determined from the AASHTO specifications figure 6.2.2.2-1b; for Hawaii the value is specified to be 105 miles per hour (154 ft/sec). The wind speed is adjusted to a 100-year wind speed by multiplying the value by 1.07 [9]. It should be noted that the resulting 100-year wind speed is approximately equal to the wind speed of a category 3 hurricane (112 mph).

The fetch length is determined using fundamental hind casting methods [21]. The following equations are used to compute the fetch length.

\[ H_{mo} = \frac{0.21 W^2}{g} \] (5.4-1: Significant Wave Height)

\[ W_A = 0.71 \times W^{1.23} \] (5.4-2: Adjusted Wind Speed)

\[ F = \frac{g \times H_{mo}}{W_A^2} \times \frac{3.281}{(2.56 \times 10^{-6})} \] (5.4-3: Fetch Length)

in which

- \( g \) = gravity [ft/sec²]
- \( W \) = wind speed for a 100-year storm [ft/sec]
- \( W_A \) = adjusted wind speed [ft/sec]
- \( F \) = fetch length [ft]
Using Equations 5.4-1, 5.4-2, and 5.4-3, the fetch length is computed to be 903 miles for each bridge location. For bridges that are sheltered in a harbored area, the fetch length was originally taken as the length of the harbor [8]. However, upon further analysis, the fetch lengths were increased to the computed 903 mile length. It was determined that during a storm event, the water levels will rise enough to submerge low lying areas that normally protect the region from wave action. Therefore, waves that develop in the open ocean will be able to impact the normally sheltered bridge locations.

To compute the average water depth over the fetch length Google Earth was used. The water depths provided by Google Earth are sufficiently accurate for the purposes of this thesis, where wave forces are needed in order to compare against the structural capacities computed in Chapter 4.

Oahu is a unique land mass where the continental shelf is relatively close to the island shores. After the continental shelf, the water depths increase rapidly. It was found that if the water depths after the continental shelf drop-off are used, unreasonably large waves are calculated. Therefore, the AASHTO method was adjusted for Hawaii’s ocean terrain. Instead of averaging the water depth over the entire fetch length, the distance from the continental shelf to the inland bridge location was used. Eight water depths were taken within the adjusted length and averaged.

The resulting wave parameters for each bridge location are summarized in Table 5.4-1.
<table>
<thead>
<tr>
<th>Bridge Description</th>
<th>Fetch Length (miles)</th>
<th>Average Water Depth over Fetch Length (ft)</th>
<th>Surge Depth at Bridge: (ft)</th>
<th>Wave Length (ft)</th>
<th>Hmax (ft)</th>
<th>η max (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kaliouou Bridge:</td>
<td>902.96</td>
<td>25.63</td>
<td>5.50</td>
<td>132.28</td>
<td>3.58</td>
<td>2.50</td>
</tr>
<tr>
<td>Kahaluu Stream Bridge:</td>
<td>902.96</td>
<td>16.38</td>
<td>11.00</td>
<td>156.93</td>
<td>7.15</td>
<td>5.01</td>
</tr>
<tr>
<td>New South Punaluu Bridge:</td>
<td>902.96</td>
<td>61.22</td>
<td>8.25</td>
<td>222.87</td>
<td>5.36</td>
<td>3.75</td>
</tr>
<tr>
<td>Ukoa Pond Bridge:</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>New Makaha Stream #3A Bridge:</td>
<td>902.96</td>
<td>27.10</td>
<td>8.93</td>
<td>171.95</td>
<td>5.80</td>
<td>4.06</td>
</tr>
<tr>
<td>Old Makaha Stream #3A Bridge:</td>
<td>902.96</td>
<td>27.10</td>
<td>8.93</td>
<td>171.95</td>
<td>5.80</td>
<td>4.06</td>
</tr>
<tr>
<td>Maipalaoa Bridge:</td>
<td>902.96</td>
<td>20.59</td>
<td>8.25</td>
<td>148.88</td>
<td>5.36</td>
<td>3.75</td>
</tr>
<tr>
<td>Moanalua Bridge: (single span)</td>
<td>902.96</td>
<td>7.42</td>
<td>6.87</td>
<td>80.54</td>
<td>4.47</td>
<td>3.13</td>
</tr>
<tr>
<td>(spans 3, 4, 5, &amp; 6)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kalihi Bridge: (single span)</td>
<td>902.96</td>
<td>7.42</td>
<td>6.87</td>
<td>80.54</td>
<td>4.47</td>
<td>3.13</td>
</tr>
<tr>
<td>(typical span)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2:</td>
<td>902.96</td>
<td>6.88</td>
<td>5.50</td>
<td>70.34</td>
<td>3.58</td>
<td>2.50</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3:</td>
<td>902.96</td>
<td>6.88</td>
<td>5.50</td>
<td>70.34</td>
<td>3.58</td>
<td>2.50</td>
</tr>
</tbody>
</table>
The AASHTO Guide Specification computes two sets of wave forces. The first set is the maximum quasi-static vertical force and associated forces and moments (AASHTO Section 6.1.2.2). The second set of wave forces is the maximum horizontal wave force and associated forces and moments (AASHTO Section 6.1.2.3). In each situation both horizontal and vertical wave forces are generated. The maximum of two cases is taken as the resulting wave forces. The AASHTO generated wave forces and associated overturning moments are summarized in Table 5.4-2.

Table 5.4-2: AASHTO Estimated Wave Forces

<table>
<thead>
<tr>
<th>Location</th>
<th>Vertical Wave Force (kips)</th>
<th>Horizontal Wave Force (kips)</th>
<th>Maximum Associated Overturning Moment (kip-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kuliouou Bridge:</td>
<td>288.66</td>
<td>27.61</td>
<td>15,296.27</td>
</tr>
<tr>
<td>Kahaluu Stream Bridge:</td>
<td>3737.39</td>
<td>647.40</td>
<td>112,176.33</td>
</tr>
<tr>
<td>New South Punaluu Bridge:</td>
<td>1054.21</td>
<td>75.47</td>
<td>14,407.44</td>
</tr>
<tr>
<td>Ukoa Pond Bridge:</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>New Makaha Stream #3A Bridge:</td>
<td>402.79</td>
<td>53.80</td>
<td>13,197.16</td>
</tr>
<tr>
<td>Old Makaha Stream #3A Bridge:</td>
<td>175.75</td>
<td>34.95</td>
<td>4,371.66</td>
</tr>
<tr>
<td>Maipalaoa Bridge:</td>
<td>1312.27</td>
<td>128.30</td>
<td>38,712.95</td>
</tr>
<tr>
<td>Moanalua Bridge: (single span)</td>
<td>129.35</td>
<td>20.59</td>
<td>3,348.62</td>
</tr>
<tr>
<td>Kalihi Bridge: (single span)</td>
<td>127.50</td>
<td>20.59</td>
<td>4,505.59</td>
</tr>
<tr>
<td>(typical span)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2:</td>
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<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3:</td>
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<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>
6 Comparison of Bridge Structural Capacities to Estimated Wave Forces

6.1 Comparison of Results

The computed structural capacities from Chapter 4 are compared to the estimated wave forces from Chapter 5. If the structural capacities are found to be greater than the estimated wave loads then it can be concluded that the bridge will survive a 100-year storm event. The following tables are comparisons of the three wave estimation methods and the respective structural capacities.

Table 6.1-1: Vertical Wave Force Comparison

<table>
<thead>
<tr>
<th></th>
<th>Douglass Vertical Force (kips)</th>
<th>McPherson Vertical Force (kips)</th>
<th>AASHTO Vertical Force (kips)</th>
<th>Vertical Bridge Capacity (kips)</th>
<th>Bridge Deck Capacity (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kalanikoa Bridge:</td>
<td>342.19</td>
<td>531.23</td>
<td>288.66</td>
<td>1157.23</td>
<td>740.23</td>
</tr>
<tr>
<td>Kahaluu Stream Bridge:</td>
<td>2808.58</td>
<td>2876.79</td>
<td>3737.39</td>
<td>3811.55</td>
<td>3819.40</td>
</tr>
<tr>
<td>New South Punaluu Bridge: (span #2)</td>
<td>1125.70</td>
<td>1119.70</td>
<td>1054.21</td>
<td>NA</td>
<td>1262.99</td>
</tr>
<tr>
<td>Ukea Pond Bridge:</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>2752.11</td>
</tr>
<tr>
<td>New Makaha Stream #3A Bridge:</td>
<td>505.65</td>
<td>495.69</td>
<td>402.79</td>
<td>NA</td>
<td>1127.13</td>
</tr>
<tr>
<td>Old Makaha Stream #3A Bridge:</td>
<td>43.07</td>
<td>518.46</td>
<td>175.75</td>
<td>349.13</td>
<td>303.37</td>
</tr>
<tr>
<td>Maipalaoa Bridge:</td>
<td>1036.20</td>
<td>1309.70</td>
<td>1312.27</td>
<td>1406.69</td>
<td>1449.29</td>
</tr>
<tr>
<td>Moanalua Bridge: (single span)</td>
<td>185.28</td>
<td>319.76</td>
<td>129.35</td>
<td>417.27</td>
<td>453.39</td>
</tr>
<tr>
<td>Kalihi Bridge: (single span) (typical span)</td>
<td>254.40</td>
<td>374.83</td>
<td>127.50</td>
<td>565.087</td>
<td>619.13</td>
</tr>
<tr>
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<td>0.00</td>
<td>206.95</td>
<td>221.01</td>
</tr>
<tr>
<td>Nineteenth Hwy. Slip Cover #3:</td>
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<td>0.00</td>
<td>0.00</td>
<td>181.67</td>
<td>245.26</td>
</tr>
</tbody>
</table>
### Table 6.1-2: Horizontal Wave Comparison

<table>
<thead>
<tr>
<th>Bridge</th>
<th>Douglass Horizontal Force (kips)</th>
<th>McPherson Horizontal Force (kips)</th>
<th>AASHTO Horizontal Force (kips)</th>
<th>Horizontal Bridge Capacity (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kuliouou Bridge:</td>
<td>52.34</td>
<td>29.73</td>
<td>27.61</td>
<td>518.10</td>
</tr>
<tr>
<td>Kahaluu Stream Bridge:</td>
<td>929.86</td>
<td>380.03</td>
<td>647.40</td>
<td>1823.88</td>
</tr>
<tr>
<td>New South Punalu Bridge: (span #2)</td>
<td>1277.59</td>
<td>54.14</td>
<td>75.47</td>
<td>725.84</td>
</tr>
<tr>
<td>Ukoa Pond Bridge:</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>New Makaha Stream #3A Bridge:</td>
<td>7.64</td>
<td>21.37</td>
<td>53.80</td>
<td>9799.82</td>
</tr>
<tr>
<td>Old Makaha Stream #3A Bridge:</td>
<td>41.41</td>
<td>8.88</td>
<td>34.95</td>
<td>70.37</td>
</tr>
<tr>
<td>Maipalaoa Bridge:</td>
<td>682.12</td>
<td>87.61</td>
<td>128.30</td>
<td>263.35</td>
</tr>
<tr>
<td>Moanalua Bridge: (single span)</td>
<td>17.42</td>
<td>18.72</td>
<td>20.59</td>
<td>333.82</td>
</tr>
<tr>
<td>Kalihi Bridge: (single span)</td>
<td>24.06</td>
<td>24.63</td>
<td>20.59</td>
<td>452.07</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2:</td>
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<td>0.00</td>
<td>0.00</td>
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</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3:</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>1744.02</td>
</tr>
</tbody>
</table>

### Table 6.1-3: Overturning Moment Comparison

<table>
<thead>
<tr>
<th>Bridge</th>
<th>Douglass Overturning Moment (kip-ft)</th>
<th>McPherson Overturning Moment (kip-ft)</th>
<th>AASHTO Overturning Moment (kip-ft)</th>
<th>Overturning Moment Capacity (kip-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kuliouou Bridge:</td>
<td>11585.83</td>
<td>18355.91</td>
<td>15296.27</td>
<td>24879.90</td>
</tr>
<tr>
<td>Kahaluu Stream Bridge:</td>
<td>67909.89</td>
<td>67519.95</td>
<td>112176.33</td>
<td>87665.70</td>
</tr>
<tr>
<td>New South Punalu Bridge: (span #2)</td>
<td>31735.61</td>
<td>28144.71</td>
<td>14407.44</td>
<td>NA</td>
</tr>
<tr>
<td>Ukoa Pond Bridge:</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>New Makaha Stream #3A Bridge:</td>
<td>11855.76</td>
<td>11649.65</td>
<td>13197.16</td>
<td>NA</td>
</tr>
<tr>
<td>Old Makaha Stream #3A Bridge:</td>
<td>749.38</td>
<td>8531.07</td>
<td>4371.66</td>
<td>5731.55</td>
</tr>
<tr>
<td>Maipalaoa Bridge:</td>
<td>35206.78</td>
<td>42369.57</td>
<td>38712.95</td>
<td>45248.85</td>
</tr>
<tr>
<td>Moanalua Bridge: (single span)</td>
<td>6014.11</td>
<td>10343.86</td>
<td>3348.62</td>
<td>13422.19</td>
</tr>
<tr>
<td>Kalihi Bridge: (single span)</td>
<td>11310.94</td>
<td>16631.87</td>
<td>4505.59</td>
<td>24958.14</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2:</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>12390.00</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3:</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>7448.30</td>
</tr>
</tbody>
</table>
Table 6.1-4: Girder Negative Bending Capacity Comparison

<table>
<thead>
<tr>
<th>Bridge Name</th>
<th>Douglass Vertical Force (kips)</th>
<th>McPherson Vertical Force (kips)</th>
<th>AASHTO Vertical Force (kips)</th>
<th>Girder Negative Bending Capacity (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kaliouou Bridge</td>
<td>27.68</td>
<td>44.27</td>
<td>24.06</td>
<td>63.49</td>
</tr>
<tr>
<td>Kahaluu Stream Bridge:</td>
<td>117.02</td>
<td>119.87</td>
<td>155.72</td>
<td>131.13</td>
</tr>
<tr>
<td>New South Punalu Bridge: (span #2)</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Ukoa Pond Bridge</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>New Makaha Stream #3A Bridge:</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Old Makaha Stream #3A Bridge:</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Maipalaoa Bridge</td>
<td>32.38</td>
<td>40.93</td>
<td>41.01</td>
<td>80.70</td>
</tr>
<tr>
<td>Moanalua Bridge: (single span)</td>
<td>20.59</td>
<td>35.53</td>
<td>14.37</td>
<td>68.43</td>
</tr>
<tr>
<td>Kalihi Bridge: (single span)</td>
<td>19.57</td>
<td>28.83</td>
<td>9.81</td>
<td>65.54</td>
</tr>
<tr>
<td>(typical span)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2:</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>55.86</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3:</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>82.36</td>
</tr>
</tbody>
</table>

Table 6.1-4 compares the negative bending capacity of the bridge girders to the portion of the vertical wave force that will impact the tributary area of a single girder. The total wave magnitudes that were computed in Chapter 5 were divided by the area of the bridge contributing to vertical uplift (i.e. the projection of the bridge deck onto the horizontal plane). The resulting value was then multiplied by the area of the tributary area of the girder. The calculated values are summarized in columns 1, 2, and 3 of Table 6-4.

The Old Makaha #3A Bridge is neglected from this calculation because the bridge girders are composed of wooden planks. It is more likely that the bridge will be displaced before the wooden girders fail due to a negative moment. The Ukoa Pond Bridge girders were determined to fail once the bridge becomes submerged, and therefore no further calculations were necessary for these girders.

Figures 6-1, 6-2, 6-3, 6-4, 6-5, and 6-6 are graphical representations of the values found in Table 6.1-1, Table 6.1-2, and Table 6.1-3.
Figure 6.1: Graphical Comparison of Vertical Wave Forces
Figure 6-2: Graphical Comparison of Vertical Wave Forces cont.

<table>
<thead>
<tr>
<th>Bridge Type</th>
<th>Wave Force (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maipalaa Bridge: (single span)</td>
<td>1036.20</td>
</tr>
<tr>
<td>Moanalua Bridge: (single span)</td>
<td>185.28</td>
</tr>
<tr>
<td>Kalihi Bridge: (single span)</td>
<td>254.40</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2:</td>
<td>0.00</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3:</td>
<td>0.00</td>
</tr>
<tr>
<td>Douglass</td>
<td>1309.70</td>
</tr>
<tr>
<td>McPherson</td>
<td>319.76</td>
</tr>
<tr>
<td>AASHTO</td>
<td>1312.27</td>
</tr>
<tr>
<td>Bridge Vertical Capacity</td>
<td>1406.69</td>
</tr>
<tr>
<td>Bridge Deck Capacity</td>
<td>1449.29</td>
</tr>
<tr>
<td>horizontal</td>
<td>417.27</td>
</tr>
<tr>
<td>vertical</td>
<td>565.087</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2:</td>
<td>206.95</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3:</td>
<td>181.67</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3:</td>
<td>221.01</td>
</tr>
<tr>
<td>Maipalaa Bridge:</td>
<td>245.26</td>
</tr>
</tbody>
</table>
Figure 6-3: Graphical Comparison of Horizontal Wave Forces

<table>
<thead>
<tr>
<th>Bridge Type</th>
<th>Wave Force (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kulouui Bridge</td>
<td>52.34</td>
</tr>
<tr>
<td>Kahaluu Stream Bridge</td>
<td>929.86</td>
</tr>
<tr>
<td>New South Punaluu Bridge</td>
<td>0.00</td>
</tr>
<tr>
<td>New Makaha Stream #3A Bridge</td>
<td>7.64</td>
</tr>
<tr>
<td>Old Makaha Stream #3A Bridge</td>
<td>41.41</td>
</tr>
</tbody>
</table>

- Douglass: 52.34
- McPherson: 29.73
- AASHTO: 27.61
- Bridge Horizontal Capacity: 518.10

Graph showing wave forces for different bridge types and scenarios.
Figure 6-4: Graphical Comparison of Horizontal Wave Forces cont.

<table>
<thead>
<tr>
<th>Bridge Name</th>
<th>Wave Force (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maipalaoa Bridge:</td>
<td></td>
</tr>
<tr>
<td>(single span)</td>
<td></td>
</tr>
<tr>
<td>Douglass</td>
<td>0.00</td>
</tr>
<tr>
<td>McPherson</td>
<td>87.61</td>
</tr>
<tr>
<td>AASHTO</td>
<td>128.30</td>
</tr>
<tr>
<td>Bridge Horizontal Capacity</td>
<td>263.35</td>
</tr>
<tr>
<td>Moanalua Bridge:</td>
<td></td>
</tr>
<tr>
<td>(single span)</td>
<td></td>
</tr>
<tr>
<td>Douglass</td>
<td>17.42</td>
</tr>
<tr>
<td>McPherson</td>
<td>18.72</td>
</tr>
<tr>
<td>AASHTO</td>
<td>20.59</td>
</tr>
<tr>
<td>Bridge Horizontal Capacity</td>
<td>333.82</td>
</tr>
<tr>
<td>Kalihi Bridge:</td>
<td></td>
</tr>
<tr>
<td>(single span)</td>
<td></td>
</tr>
<tr>
<td>Douglass</td>
<td>24.06</td>
</tr>
<tr>
<td>McPherson</td>
<td>24.63</td>
</tr>
<tr>
<td>AASHTO</td>
<td>20.59</td>
</tr>
<tr>
<td>Bridge Horizontal Capacity</td>
<td>452.07</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2:</td>
<td></td>
</tr>
<tr>
<td>Douglass</td>
<td>0.00</td>
</tr>
<tr>
<td>McPherson</td>
<td>0.00</td>
</tr>
<tr>
<td>AASHTO</td>
<td>0.00</td>
</tr>
<tr>
<td>Bridge Horizontal Capacity</td>
<td>1655.56</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3:</td>
<td></td>
</tr>
<tr>
<td>Douglass</td>
<td>0.00</td>
</tr>
<tr>
<td>McPherson</td>
<td>0.00</td>
</tr>
<tr>
<td>AASHTO</td>
<td>0.00</td>
</tr>
<tr>
<td>Bridge Horizontal Capacity</td>
<td>1744.02</td>
</tr>
</tbody>
</table>
Figure 6-5: Graphical Comparison of Overturning Moments
Figure 6-6: Graphical Comparison of Overturning Moments cont.

<table>
<thead>
<tr>
<th>Bridge/Slip Cover</th>
<th>Overturning Moment (kip-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moanalua Bridge: (single span)</td>
<td>6014.11</td>
</tr>
<tr>
<td>Kalihi Bridge: (single span)</td>
<td>11310.94</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2:</td>
<td>0.00</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3:</td>
<td>0.00</td>
</tr>
<tr>
<td>Douglass</td>
<td>10343.86</td>
</tr>
<tr>
<td>McPherson</td>
<td>16631.87</td>
</tr>
<tr>
<td>AASHTO</td>
<td>3348.62</td>
</tr>
<tr>
<td>Bridge Overtwisting Moment Capacity</td>
<td>13422.19</td>
</tr>
<tr>
<td></td>
<td>24958.14</td>
</tr>
<tr>
<td></td>
<td>12390.00</td>
</tr>
<tr>
<td></td>
<td>7448.30</td>
</tr>
</tbody>
</table>
6.2 Comparison of Bridge Structural Capacities to Estimated Wave Forces: Discussion of Results

The maximum wave forces produced by the three wave estimation methods are compared against the computed structural capacities as a preliminary check of the bridge’s vulnerability to failure. However, if a wave estimation method is not fully applicable or has been found to overestimate forces for the conditions of a particular bridge, then the wave forces estimated by the method are ignored. The bridge’s structural capacity must be greater than the vertical, horizontal and overturning moment in order to survive a 100-year storm event. If a bridge does not meet these criteria, then the bridge is at risk of failing during a storm event. If the bridge structural capacities are above the estimated wave forces then a factor of safety is computed. The purpose of the factor of safety is to illustrate the structural capacity of the bridge superstructure beyond the expected wave loads.

It should be noted that McPherson’s research found that the Douglass method increasingly overpredicts the horizontal force as the water depth increases [11]. This overestimation is observed in the Maipalaoa Bridge and the New South Punaluu Bridge, where the base flood elevations are above the bridge decks. For this reason, the horizontal forces generated from Douglass’ method for these particular bridges have been ignored and were not included in Figure 6-3 and Figure 6-4.

In addition, each of the methods used to estimate the wave forces are not fully applicable when the bridges are completely submerged. Each method computes continuously increasing wave forces as water depths increase. In reality, the forces on the bridges should reach a maximum limit, and then decrease as water levels continue to rise. The New South Punaluu Bridge is the only locations where this condition is relevant, as the storm surge elevation is above the deck of the bridge. Therefore, the wave forces experienced by the bridge during a 100-year storm event will likely be smaller in magnitude than those computed in this thesis.
6.2.1 Bridge Negative Bending Capacity

It was determined that the upward wave force on the Kahaluu Bridge estimated by AASHTO exceeds the negative bending capacity of the girders (see Table 6.1-4). Therefore, the Kahaluu Bridge is at risk of failing due to negative bending of the bridge girders.

The bridge girders of the remaining bridges have a great enough negative bending capacity to resist the upward wave loads. Therefore, the girders of the Kuliouou Stream Bridge, the Maipalaoa Bridge, the Moanalua Bridge, the Kalihi Bridge, and the Nimitz Highway Slip Covers #2 and #3 will not fail during a 100-year storm event.

6.2.2 Kuliouou Stream Bridge

The Kuliouou Stream Bridge is determined to survive a 100-year storm event. In the vertical direction, the factor of safety above the largest estimated wave force is computed to be 2.18. The factor of safety against horizontal wave forces and the associated overturning moments are 9.90 and 1.36, respectively. Even though the factor of safety to overturning moment is low, it is not likely that the combination of the horizontal and vertical wave loads will be great enough to completely flip the bridge deck over.

Additionally, the bridge deck and girders are not at risk of failing. Nor is it likely that the bridge will be buoyant if submerged by tsunami inundation.

6.2.3 Kahaluu Bridge

The Kahaluu Bridge is at risk of failing during a 100-year storm event. Recall from Section 4.4.3, that the only source of vertical resistance to wave loads is provided by the self weight of the bridge. As a result, the factor of safety against vertical wave loads is computed to be 1.02. Therefore, with repeated wave impacts the bridge will be lifted out of the pot type bearing pads. In addition, the overturning moment estimated by the
AASHTO specifications is 28% above the overturning moment resistance of the bridge. Ultimately the bridge may be flipped over by wave forces, making the bridge unusable after a storm event.

The Kahaluu Bridge is also susceptible to negative bending failure. The upward wave force estimated by AASHTO for the Kahaluu Bridge exceeds the negative bending capacity of the girders. Therefore, the Kahaluu Bridge is also at risk of failing due to negative bending of the bridge girders.

6.2.4 Ukoa Pond Bridge

The Ukoa Pond Bridge is at risk of failing if it is submerged by tsunami inundation. It was found that the Ukoa Pond Bridge is located outside of the FEEMA Flood Study. Therefore, if the bridge is submerged, it will be caused by a tsunami. Wave forces were not estimated for the Ukoa Pond Bridge as it is sheltered from direct wave impacts by thick brush. However, the bridge will likely fail due to buoyancy forces. It was determined that the bridge will be fully buoyant if 84% of the volume under the bridge is filled with air. If this condition is met, then the bridge will break the vertical hinge restrainers connecting the bridge superstructure to its foundation. This will allow the bridge to be displaced by current and wind loads, making the bridge unusable during a storm event.

6.2.5 Old Makaha #3A Bridge

The Old Makaha #3A Bridge is determined to fail during a 100-year storm event. The vertical wave force and overturning moment estimated by McPherson’s method were both computed to be greater than the bridge’s resistance. Therefore the bridge is at risk of failing during a 100-year storm event.

In addition, the bridge is fully buoyant if submerged by tsunami inundation.
6.2.6 New Makaha #3A Bridge

The New Makaha #3A Bridge is determined to survive a 100-year storm event. Bridge displacement is not of concern, as the bridge superstructure is securely attached to its reinforced concrete abutments.

The prestressed hollow core planks used in the construction of the bridge deck were determined to have a high enough capacity to resist the estimated vertical wave forces, and are therefore are not in danger of failing due to negative bending. It is calculated that the bridge deck has a 2.23 factor of safety against the largest estimated vertical wave force. Thus, it is likely that the New Makaha #3A Bridge will survive a 100-year storm event.

Additionally, the New Makaha #3A Bridge will not be buoyant if submerged by tsunami inundation.

6.2.7 New South Punaluu Bridge

The New South Punaluu Bridge is not at risk of failing during a 100-year storm event. The safety factor against the maximum vertical wave load estimated by Douglass method is calculated to be 1.12. Also, the New South Punaluu Bridge will not be buoyant if submerged by storm surge or tsunami inundation.

It should be noted that the still water level during a 100-year storm is above the deck of the bridge. Therefore, the methods used to estimate the wave forces are not directly applicable. It is likely that the wave forces on the bridge deck will be smaller than the values calculated in Chapter 5 of this thesis.

6.2.8 Maipalaoa Bridge

The Maipalaoa Bridge is at risk of failing during a 100-year storm event. The factor of safety against vertical wave loads and the associated overturning moment are
computed to be 1.07 and 1.07, respectively. The low factor of safety indicates that there exists a high chance that repeated wave impacts will be able to exceed the bridge’s resistance to wave loads. With no wing walls to stop lateral displacement, wave loads will be able to progressively “bump” the bridge until it is displaced off of the abutments.

6.2.9 Moanalua Bridge

The Moanalua Bridge is determined to survive a 100-year storm event. The factors of safety against vertical wave loads, horizontal wave loads and the associated overturning moments are computed to be 1.30, 16.21, and 1.30 respectively.

The Moanalua Bridge is also not buoyant if it becomes submerged by tsunami inundation.

6.2.10 Kalihi Bridge

The Kalihi Bridge is determined to survive a 100-year storm event. The factors of safety against vertical wave loads, horizontal wave loads and the associated overturning moments are computed to be 1.51, 18.36, and 1.50 respectively.

The Kalihi Bridge is also not buoyant if it becomes submerged by tsunami inundation.

6.2.11 Nimitz Highway Slip Covers #2 and #3

Based on the data from the FEMA studies, the highest wave crest elevation during a 100-year storm is 4.33 ft below the deck of the bridges. Therefore, based on the analysis completed in this study, it is determined that the slip covers are sufficiently high enough, such that waves will not directly impact the bridge decks. Thus the slip covers are not in immediate danger of failing during a 100-year storm event.
6.3 AASHTO Guide Specifications

The AASHTO Guide Specifications for Bridges Vulnerable to Coastal Storms is the only method that is a specified guide used to estimate wave forces on bridge structures. Therefore, if the wave forces generated by the AASHTO guide are compared to the structural capacities, then only the Kahaluu Bridge will potentially fail due to an overturning moment. The forces computed to impact the other bridges are below the calculated structural capacities. Thus, based on the AASHTO guide, all of the analyzed bridges, except the Kahaluu Bridge, are not at risk of failing during a 100-year storm event.

The calculated factors of safety above the AASHTO wave forces are summarized in Table 6.3-1. If any of the factors of safety are below 1.0, then the wave force estimated by AASHTO exceeds the capacity of the bridge.
<table>
<thead>
<tr>
<th>Bridge Name</th>
<th>Vertical Factor of Safety</th>
<th>Horizontal Factor of Safety</th>
<th>Overturing Moment FS</th>
<th>Bridge Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kuliouou Bridge:</td>
<td>4.01</td>
<td>18.76</td>
<td>1.63</td>
<td>OK</td>
</tr>
<tr>
<td>Kahaluu Stream Bridge:</td>
<td>1.02</td>
<td>2.82</td>
<td>0.78</td>
<td>NG</td>
</tr>
<tr>
<td>New South Punaluu Bridge:</td>
<td>1.20</td>
<td>9.62</td>
<td>NA</td>
<td>OK</td>
</tr>
<tr>
<td>Ukoa Pond Bridge:</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>OK</td>
</tr>
<tr>
<td>New Makaha Stream #3A Bridge:</td>
<td>2.80</td>
<td>182.16</td>
<td>NA</td>
<td>OK</td>
</tr>
<tr>
<td>Old Makaha Stream #3A Bridge:</td>
<td>1.99</td>
<td>2.01</td>
<td>1.31</td>
<td>OK</td>
</tr>
<tr>
<td>Maipalaoa Bridge:</td>
<td>1.07</td>
<td>1.81</td>
<td>1.17</td>
<td>OK</td>
</tr>
<tr>
<td>Moanalua Bridge: (single span)</td>
<td>3.23</td>
<td>16.21</td>
<td>4.01</td>
<td>OK</td>
</tr>
<tr>
<td>(spans 3, 4, 5, &amp; 6)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kalihi Bridge: (single span)</td>
<td>4.43</td>
<td>21.96</td>
<td>5.54</td>
<td>OK</td>
</tr>
<tr>
<td>(typical span)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2:</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>OK</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3:</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>OK</td>
</tr>
</tbody>
</table>
6.4 Summary of Bridges

Table 6.4-1 summarizes the wave estimation method that estimates the largest wave force for a particular bridge. Table 6.4-2 summarizes the calculate factors of safety for each bridge. The factors of safety were calculated by dividing the bridge’s capacity by the largest estimated wave load. The cells in Table 6.4-2 highlighted in pink are factors of safety that are less than one, which results if the bridge’s capacity less than the estimated wave load. This indicates that the bridge is at risk of failing.

Table 6.4-3 summarizes each of the bridges and their vulnerability to failure during a 100-year storm event or during tsunami inundation. Column 1 of Table 6.4-3 summarizes the potential failure of each bridge. Column 2 provides notes on each bridge, and if necessary gives a short description of the cause of the potential bridge failure. Column 3 provides the vulnerability of the bridge. If the bridge is determined to survive a 100-year storm event, then the vulnerability is listed as ‘OK.’ If the wave loads are greater than the estimated bridge capacities then the vulnerability is listed as ‘At Risk.’
<table>
<thead>
<tr>
<th>Bridge Name</th>
<th>Largest Overturning Moment Estimated By</th>
<th>Largest Horizontal Force Estimated By</th>
<th>Largest Vertical Force Estimated By</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kuliouou Bridge</td>
<td>AASHTO</td>
<td>Douglass</td>
<td>McPherson</td>
</tr>
<tr>
<td>Kahaluu Stream Bridge</td>
<td>Douglass</td>
<td>AASHTO</td>
<td>McPherson</td>
</tr>
<tr>
<td>New South Punaluu Bridge</td>
<td>Douglass</td>
<td>AASHTO</td>
<td>McPherson</td>
</tr>
<tr>
<td>Ukoa Pond Bridge</td>
<td>Douglass</td>
<td>AASHTO</td>
<td>McPherson</td>
</tr>
<tr>
<td>New Makaha Stream #3A Bridge</td>
<td>McPherson</td>
<td>AASHTO</td>
<td>McPherson</td>
</tr>
<tr>
<td>Old Makaha Stream #3A Bridge</td>
<td>McPherson</td>
<td>AASHTO</td>
<td>McPherson</td>
</tr>
<tr>
<td>Maipalaoa Bridge</td>
<td>McPherson</td>
<td>AASHTO</td>
<td>McPherson</td>
</tr>
<tr>
<td>Moanalua Bridge</td>
<td>McPherson</td>
<td>AASHTO</td>
<td>McPherson</td>
</tr>
<tr>
<td>Kalihi Bridge</td>
<td>McPherson</td>
<td>AASHTO</td>
<td>McPherson</td>
</tr>
<tr>
<td>Niniliz Hwy. Slip Cover #2</td>
<td>McPherson</td>
<td>AASHTO</td>
<td>McPherson</td>
</tr>
<tr>
<td>Niniliz Hwy. Slip Cover #3</td>
<td>McPherson</td>
<td>AASHTO</td>
<td>McPherson</td>
</tr>
<tr>
<td>Bridge Name</td>
<td>Vertical</td>
<td>Lateral</td>
<td>Overturning</td>
</tr>
<tr>
<td>-----------------------------------</td>
<td>----------</td>
<td>---------</td>
<td>-------------</td>
</tr>
<tr>
<td>Kuliouou Bridge</td>
<td>2.18</td>
<td>9.90</td>
<td>1.36</td>
</tr>
<tr>
<td>Kahaluu Stream Bridge</td>
<td>1.02</td>
<td>1.96</td>
<td>0.78</td>
</tr>
<tr>
<td>New South Punaluu Bridge</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Ukoa Pond Bridge</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>New Makaha Stream #3A Bridge</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Old Makaha Stream #3A Bridge</td>
<td>0.67</td>
<td>1.70</td>
<td>0.67</td>
</tr>
<tr>
<td>Maipalaoa Bridge</td>
<td>1.07</td>
<td>2.05</td>
<td>1.07</td>
</tr>
<tr>
<td>Moanalua Bridge</td>
<td>1.30</td>
<td>16.21</td>
<td>1.30</td>
</tr>
<tr>
<td>Kalihi Bridge</td>
<td>1.51</td>
<td>18.36</td>
<td>1.50</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
</tbody>
</table>

Table 6.4.2: Calculated Factors of Safety
<table>
<thead>
<tr>
<th>Estimated Potential Failure Mode</th>
<th>Notes</th>
<th>Bridge Vulnerability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kuliouou Bridge:</td>
<td>None</td>
<td>The estimated wave forces are less than the calculated capacities.</td>
</tr>
<tr>
<td>Kahaluu Stream Bridge:</td>
<td>Displacement - Overturning, Bridge Negative Bending Failure</td>
<td>The estimated wave forces by McPherson’s method are greater than the overturning resistance and the bridge negative bending capacity.</td>
</tr>
<tr>
<td>New South Punaluu Bridge:</td>
<td>None</td>
<td>The estimated vertical wave loads are below the negative bending capacity of the bridge deck.</td>
</tr>
<tr>
<td>Ukoa Pond Bridge:</td>
<td>Buoyancy</td>
<td>An 84% air pocket will cause the buoyancy force to be greater than the total vertical capacity of the bridge.</td>
</tr>
<tr>
<td>New Makaha Stream #3A Bridge:</td>
<td>None</td>
<td>The estimated vertical wave forces are less than the calculated negative bending capacity of the bridge deck.</td>
</tr>
<tr>
<td>Old Makaha Stream #3A Bridge:</td>
<td>Displacement - Vertical, Displacement - Overturning</td>
<td>The vertical wave load and associated overturning moment estimated by McPherson’s method exceeds the vertical capacity of the bridge. In addition, if the bridge is submerged the bridge will be buoyant.</td>
</tr>
<tr>
<td>Maipalaoa Bridge:</td>
<td>Displacement - Vertical, Displacement - Overturning</td>
<td>The estimated vertical wave force by AASHTO and overturning moment estimated by McPherson nearly exceed the bridge capacities. Repeated wave impacts may cause failure.</td>
</tr>
<tr>
<td>Moanalua Bridge:</td>
<td>None</td>
<td>The estimated wave forces are less than the calculated capacities.</td>
</tr>
<tr>
<td>Kalihi Bridge:</td>
<td>None</td>
<td>The estimated wave forces are less than the calculated capacities.</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #2:</td>
<td>None</td>
<td>During a 100-year storm event, the highest wave crest elevation is 4.33 ft below the deck of the bridge. However, an existing sea wall at the far edge of the bridge will redirect the waves upward, increasing the wave height 2.0 to 2.3 times its original height. Therefore, there will be a force on the slip covers. Based on the analysis performed in this thesis the slip cover is not in immediate danger of failing during a 100-year storm event.</td>
</tr>
<tr>
<td>Nimitz Hwy. Slip Cover #3:</td>
<td>None</td>
<td>During a 100-year storm event, the highest wave crest elevation is 4.33 ft below the deck of the bridge. However, an existing sea wall at the far edge of the bridge will redirect the waves upward, increasing the wave height 2.0 to 2.3 times its original height. Therefore, there will be a force on the slip covers. Based on the analysis performed in this thesis the slip cover is not in immediate danger of failing during a 100-year storm event.</td>
</tr>
</tbody>
</table>
Eleven bridges around the island of Oahu were structurally evaluated to determine their vulnerability to failure when impacted by storm waves or when submerged by tsunami inundation. As a preliminary check of the bridges vulnerability to failure, the structural capacities of each bridge were compared to wave forces generated by three different wave estimation methods. The storm waves were computed based on a 100-year storm. Through the structural analyses it was determined that the Kahaluu Bridge, Old Makaha #3A Bridge, and the Maipalaoa Bridge are all at risk of failing during a 100-year storm event. If submerged by tsunami inundation, the Ukoa Pond Bridge was determined to be at risk of failing due to buoyancy and negative bending of the bridge girders.

The Kuliouou Stream Bridge, the New Makaha #3A Bridge, the New South Punaluu Bridge, the Moanalua Bridge, the Kalihi Bridge, and the Nimitz Highway Slip Covers #2 and #3 are all determined to have sufficient factors of safety against wave loads and are not at risk of becoming buoyant if submerged by tsunami inundation, which indicates that the bridges will likely survive a 100-year storm event.

Based on the results obtained, the following recommendations are made:

- Many of the analyzed bridges are gravity type structures, where the main source of vertical and lateral resistance to wave loads is provided by the self weight of the bridge. It is recommended that connection retrofits should be added to secure the bridge girders to the bridge’s foundation and abutments. If possible, thru bolts passing through the bulb of the girders should be used to attach steel angle connections on either side of the girder. Stiffeners should be added to the connections to prevent angle bending failure [16].
- Lehrman [17] performed extensive research on common anchors used in the construction of coastal bridges. Lehrman found that the anchors did not have sufficient strength to resist wave loads predicted by the AASHTO Guide
specifications if there is entrapped air under the bridge deck. Therefore, it is suggested that low lying bridges be designed with bulkheads that will allow air to escape from below the bridge decks, which will reduce the volume of trapped air [15]. This should be done in addition to providing proper anchorage.

- It was determined that buoyancy forces can significantly reduce the self weight of a bridge superstructure, which in turn decreases the gravity induced horizontal frictional resistance. For this reason, it is recommended that shear keys be provided on low lying bridges to resist all anticipated lateral loads. The contributing resistance provided by gravity induced friction should be ignored when designing the shear keys [15].

- It was determined that prestressed members are most likely to fail due to negative bending. Thus, it is recommended that prestressed systems be designed to withstand the negative bending and shear caused by upward hydrodynamic and hydrostatic forces [11]. Possible design modifications include: increasing the strength of concrete or reducing the distance from the centroid of the prestressing tendons to the centroid of cross sectional area of the concrete.

- If it is economically feasible and structurally possible, bridges should be raised to an elevation that will result in a one foot minimal clearance above the highest estimated storm wave crest elevation. If the bridge is sufficiently high, then waves will not be able to impact the bridge.

- If a full hydrodynamic study is not possible, then it is suggested that McPherson’s [11] method be used to estimate wave forces on bridge structures. When compared to the AASHTO Guide Specifications [9], both methods produce similar wave loads for the majority of cases. The advantage of McPherson’s method is that it requires less computational effort.
8 References


16. Robertson, Ian N.; Yim, Solomon; Tran, T. (n.d.). Case study of Concrete Bridge Subjected to Hurricane Storm Surge and Wave Action.


Appendices
Appendix A: Hand Calculations
**Bridge Deck Calculations:** (New Bridge Deck)

- Sheet 31

\[ A_1 = \frac{1}{2} \left( \frac{1}{2} \cdot \frac{1}{2} + \frac{1}{2} \cdot \frac{1}{4} \right) + 6.25 + 5.25 + 5.25 + 2.0 \] = 12.56 ft²

\[ A_2 = (5)(1.675) \] = 4.375 ft²

\[ A_3 = \left( \frac{1}{2} \right) \left( \frac{1}{2} \right) \] = 0.6875 ft²

\[ A_4 = \left( \frac{1}{4} \right) \left( \frac{1}{2} \right) \] = 0.1675 ft²

\[ A_T = A_1 + A_2 = 17.935 \text{ ft}^2 \]

\[ A_{guardrails} = A_3 + A_4 = 1.604 \text{ ft}^2 \]

\[ A_p = 1.604 \text{ ft}^2 \text{ Guard Rails} \]

\[ A_p = 7.99 \text{ ft}^2 \text{ New Deck no guardrails} \]
Concrete Diaphragm (New Section)

- Sheet 31 for geometry
- Sheet 32 for Diaphragm

\[ A_1 = (4 + \frac{3}{12})(\frac{3}{12}) = 2.125 \text{ ft}^2 \]

\[ A_2 = 2 \left[ \frac{1}{2} \times \frac{3}{12} \times \frac{3}{12} \right] = 0.0625 \text{ ft}^3 \]

\[ A_3 = (4 + \frac{3}{12})(\frac{11}{12}) = 6.333 \text{ ft}^3 \]

\[ A_T = A_1 - A_2 + A_3 = 8.39583 \text{ ft}^2 \]

- \[ A_T = 8.396 \text{ ft}^2 \] Concrete Diaphragm
  New Bridge Section

Thickness = 1.26 ft

Amount = 6
Original Bridge Section:

- **Side Girders:**
  \[ A_1 = \left(\frac{1}{2}\pi \times \frac{1}{2}\pi \right) = 4.5 \text{ ft}^2 \]
  \[ A_2 = \frac{1}{2} \left(\frac{1}{2}\pi \times \frac{1}{2}\pi \right) = 0.125 \text{ ft}^2 \]
  \[ A_T = A_1 + A_2 = 4.625 \text{ ft}^2 \]
  \[ A_T = 4.625 \text{ ft}^2 \] Side Girders

- **Inner Girders:**
  \[ A_1 = \left(\frac{1}{2}\pi \times \frac{1}{2}\pi \times \frac{1}{2}\pi \right) = 4.5 \]
  \[ A_2 = 2 \left[ \frac{1}{2} \left(\frac{1}{2}\pi \times \frac{1}{2}\pi \right) \right] = 0.25 \text{ ft}^2 \]
  \[ A_T = A_1 + A_2 = 4.75 \text{ ft}^2 \]
  \[ A_T = 4.75 \text{ ft}^2 \] Inner Girders

- **Deck:**
  \[ \text{Width} = (8 + \frac{1}{2}\pi) + (8 + \frac{1}{2}\pi) + (8 + \frac{1}{2}\pi) + (\frac{1}{2}\pi) + (\frac{1}{2}\pi) = 28 \text{ ft} \]
  \[ A_T = (28) \times \frac{1}{2}\pi = 18.67 \text{ ft}^2 \]
  \[ A_T = 18.67 \text{ ft}^2 \] Deck

- **Total Area:**
  \[ A_T = (2) \text{Side} + (2) \text{Inner} + \text{Deck} \]
  \[ = (2)(4.625) + (2)(4.75) + 18.67 \]
  \[ A_T = 37.4167 \text{ ft}^2 \]
  \[ A_T = 37.4 \text{ ft}^2 \] Total Area Original Section
Concrete Diaphragm (Original Section)

\[ A_1 = \left( \frac{15}{12} \right) \left( \frac{10}{12} \right) = 18.333 \, \text{ft}^2 \]

\[ A_2 = 2 \left[ \frac{1}{2} \left( \frac{5}{12} \right)^2 \right] = 0.25 \, \text{ft}^2 \quad (-) \]

\[ A_T = A_1 - A_2 = 18.083 \, \text{ft}^2 \]

Concrete Diaphragm (Original Section)
Thickness = 1.25'
Amount = 3
Kulouou Bridge  Structural Analysis  Pg. 5

* Air Pocket Calculations: (New Bridge Section)

* Sheet 31:
  0 Between Original and New Section:
  A. Note: Air is assumed to
     all the entire area
     between old and new
     section girders.

\[
\begin{align*}
A_1 &= \left( \frac{25.5}{2} \times \frac{5}{2} \right) = 1.0625 \text{ ft}^2 \\
A_2 &= \frac{1}{2} \left( \frac{3}{2} \right) \left( \frac{5}{2} \right) = 0.3125 \text{ ft}^2 (-) \\
A_3 &= \frac{1}{2} \left( \frac{3}{2} \right) \left( \frac{5}{2} \right) = 0.125 \text{ ft}^2 (-) \\
A_4 &= \left( \frac{25.5}{2} \times \frac{1}{2} \right) = 3.5625 \text{ ft}^2 \\
A_5 &= \left( \frac{25.5}{2} \times \frac{1}{2} \right) = 1.1875 \text{ ft}^2 \\
A_6 &= \left( \frac{25.5}{2} \times \frac{1}{2} \right) = 0.9375 \text{ ft}^2
\end{align*}
\]

0 Between Abutment and Girder:

* Sheet 30 Section E

\[
\begin{align*}
A_1 &= \left( \frac{3}{2} \times \frac{5}{2} \right) = 0.2083 \text{ ft}^2 \\
A_2 &= \frac{1}{2} \left( \frac{3}{2} \right) \left( \frac{5}{2} \right) = 0.0315 \text{ ft}^2 (-) \\
A_3 &= \frac{1}{2} \left( \frac{3}{2} \right) \left( \frac{5}{2} \right) = 0.125 \text{ ft}^2 (-) \\
A_4 &= \left( \frac{1}{2} \times \frac{5}{2} \right) = 1.0 \text{ ft}^2 \\
A_5 &= \left( \frac{1}{2} \times \frac{5}{2} \right) = 0.333 \text{ ft}^2 \\
A_6 &= \left( \frac{3}{2} \times \frac{5}{2} \right) = 0.833 \text{ ft}^2
\end{align*}
\]

\[
A_T = A_1 - A_2 - A_3 + A_4 + A_5 + A_6 = 0.2083 - 0.0315 - 0.125 + 1.0 + 0.333 + 0.833 = 1.4687 \text{ ft}^2
\]

\[
A_T = 1.47 \text{ ft}^2 \quad \text{Air pocket between} \quad \text{abut. & girder}
\]
Air Pocket Calculations: (New Bridge Section)

- Between Girders: (no creep block)

\[ A_1 = (4 + \frac{3}{12})(\frac{3}{12}) = 2.125 \text{ ft}^2 \]
\[ A_2 = 2 \left( \frac{3}{12} \times \frac{3}{12} \times \frac{3}{12} \right) = 0.0625 \text{ ft}^2 \]
\[ A_3 = (\frac{5}{12})^2 = 0.625 \text{ ft}^2 \]
\[ A_4 = 2 \left[ \frac{1}{12} \times \frac{3}{12} \times \frac{3}{12} \right] = 0.25 \text{ ft}^2 \]
\[ A_5 = (\frac{4}{12})^2 = 0.1875 \text{ ft}^2 \]

\[ A_T = A_1 - A_2 + A_3 - A_4 + A_5 = 13.1875 \text{ ft}^2 \]

\[ A_T = 13.19 \text{ ft}^2 \] Air Pocket

- Between Girders: (creep block)

\[ A_T = A_1 - A_2 + A_3 - A_4 = 11.313 \text{ ft}^2 \]

\[ A_T = 11.31 \text{ ft}^2 \]
Air Pocket Calculations: (Original Section)

- Between Girder:

\[ A_1 = \left( \frac{88\%}{2} \right) \left( \frac{35\%}{2} \right) = 2.2 \text{ ft}^2 \]

\[ A_2 = 2 \left[ \frac{1}{2} \left( \frac{9.75\%}{2} \right) \right] = 2.25 \text{ ft}^2 \] (−)

\[ A_T = A_1 - A_2 = 21.75 \text{ ft}^2 \]

* Note: Air fills entire area between girder

Reduction of air pocket area:

- For all air Pocket:

  \[ h = \left( \frac{9.75\%}{2} \right) + \left( \frac{9.75\%}{2} \right) = 3.667 \text{ ft} \]

  \[ P_2 = 14.7 + h \left( \frac{9.75\%}{2} \right) = 14.7 + (3.667) \left( \frac{9.75\%}{2} \right) \]

  \[ P_2 = 16.3296 \text{ psi} \]

- Between Original and New Section:

  \[ A_2 = \frac{P_1 A_1}{P_2} = \frac{(14.7)(4.594)}{16.3296} \]

  \[ A_2 = 5.9356 \text{ ft}^2 \]

\[ A_T = 5.936 \text{ ft}^2 \]

- Between Abutment and Girder:

  \[ A_2 = \frac{P_1 A_1}{P_2} = \frac{(14.7)(4.587)}{16.3296} \]

  \[ A_2 = 1.3221 \text{ ft}^2 \]

\[ A_T = 1.322 \text{ ft}^2 \]

- Between Girder: (New Section) - Creep Block

  \[ A_2 = \frac{P_1 A_1}{P_2} = \frac{(14.5)(11.325)}{16.3296} \]

  \[ A_2 = 10.1836 \text{ ft}^2 \]

\[ A_T = 10.184 \text{ ft}^2 \]

- Between Girder: (New Section) - No Creep Block

  \[ A_2 = \frac{P_1 A_1}{P_2} = \frac{(14.7)(3.14)}{16.3296} \]

  \[ A_2 = 11.8714 \text{ ft}^2 \]

\[ A_T = 11.872 \text{ ft}^2 \]

- Between Girder: (Original Section)

  \[ A_2 = \frac{P_1 A_1}{P_2} = \frac{(14.7)(21.75)}{16.3296} \]

  \[ A_2 = 19.5744 \text{ ft}^2 \]

\[ A_T = 19.58 \text{ ft}^2 \]
**Collection of Data:**

- **Concrete:** $(y_{RC} = 150 \text{lbf/ft}^2)$
  - New Section Girder:
    - Area = 2.563 ft$^2$
    - Amount = 8
    - Length = 48.39'
  - New Section Deck:
    - Area = 17.94 ft$^2$
    - Amount = 2
    - Length = 48.39'
  - Railing:
    - Area = 1.604 ft$^2$
    - Amount = 2
    - Length = 48.39'

- **Diaphragm:**
  - New Section:
    - Area = 8.396 ft$^2$
    - Amount = 6
    - Length = 1.25'
  - (Original Section)
    - Area = 8.08 ft$^2$
    - Amount = 3
    - Length = 1.25'

- **Air Pocket:** (Compressed) $(y_{Vacuum} = 64 \text{lbf/ft}^2)$
  - Between Original and New Section:
    - Area = 5.936 ft$^2$
    - Amount = 2
    - Length = 48.39'
  - Between Abutment and Girder:
    - Area = 1.332 ft$^2$
    - Amount = 2
    - Length = 48.39'

- **Buoyancy Calculations:**

  - **Self Weight:**
    \[
    \text{Self Weight} = y_{RC} \left\{ 8 \text{(New Girder)} + 2 \text{Deck} + 2 \text{(Railing)} + 2 \text{(Original Section)} \right\} (48.39') + \left\{ \left( \text{New Dia}(1.25') + 3 \text{Original Dia}(1.25') \right) y_{RC} \right\} \\
    = (150)(48.39') \left\{ 8(2.563) + 2(17.94) + 2(1.604) + 37.42 \right\} + (150)(1.25') \left\{ 6(8.396) + 3(18.08) \right\} \\
    = 723277 \text{ lbs}
    \]

  - **Buoyant Force:**
    \[
    \text{Submerged Volume} = \text{Submerged Concrete} + \text{Air Pocket} \\
    = [(8(2.563) + 2(17.94) + 3(18.08) + 2(37.42) + 2(5.936) \\
    + 2(1.332) + 4(11.871) + 3(19.58)](48.39') \\
    + 2(10.184)(12.018) + 2(11.871)(36.393) \\
    = 11469.9 \text{ ft}^3
    \]
Buoyancy Calculations: cont.

\[ \text{Buoyant Force: } BF = SW \times \text{Fou} \]
\[ BF = (1489.9 \times 64) \]
\[ BF = 73535.34 \text{ lbs} \]

\[ \text{Residual Weight: } RW = SW - BF \]
\[ RW = (73535.35 - 73535.34) \]
\[ RW = -115.74 \text{ lbs} \]

% Retained Weight = -1.6%

Summary of Results:
- Self Weight = 723.8 kips
- Buoyant Force = 735.4 kips
- Residual Weight = -11.57 kips
- % Retained Weight = -1.6%

\[ \therefore \text{Bridge is Buoyant} \]
**Deck Capacity: (Widened Section)**

- **Positive Bending:**
  1. \( A_0 = 6 \times 0.20 = 1.2 \text{ in}^2 \)
  2. \( a = \frac{A_0 f_y}{0.85 t_b} = \frac{(1.2)(40)}{0.85(2)(5)} \)
    \[ a = 0.569 \text{ in.} \]
  3. \( d = 8 - 1.5 - (\frac{a}{2})(0.9) = 6.5 \text{ in.} \)
  4. \( E_b = 0.0419 > E_y = 0.004 \text{ (ok) } \)

- **Negative Bending:**
  1. \( A_1 = 3 \times 0.20 = 0.6 \text{ in}^2 \)
  2. \( a = \frac{A_1 f_y}{0.85 t_b} = \frac{(0.6)(40)}{0.85(2)(5)} \)
    \[ a = 1.845 \text{ in.} \]
  3. \( d = 8 - 1.5 - (\frac{a}{2})(0.9) = 6.25 \text{ in.} \)
  4. \( E_b > E_y \text{ (ok) } \)

- **Negative Shear:**
  1. \( V_c = 2.3 \sqrt{V_c \cdot bcd} \)
    \[ = 2.3\sqrt{(3000)(3)(6.25)} \]
    \[ V_c = 34.917\text{ kips} \]
  2. \( \phi V_c = 0.75(34.917) = 26.178 \text{ kips} \)

**Top:**
- \( M_0 = 3000 \text{ kips-in} \)
- \( V_0 = 410,000 \text{ kips} \)
- Rot: 5 - \#4
- Post: 6 - \#4

**Negative Bending:**
- \( M_n = 25.26 \text{ k-ft} \)
- \( \phi M_n = 22.74 \text{ k-ft} \)

**Positive Bending:**
- \( M_n = 25.26 \text{ k-ft} \)
- \( \phi M_n = 22.74 \text{ k-ft} \)

**Negative Shear:**
- \( V_c = 34.92 \text{ kips} \)
- \( \phi V_n = 26.19 \text{ kips} \)
<p>Deck Capacity: (Existing Section)</p>

- **Positive Bending:**
  - \( A_s = (10)(0.30) = 3.0 \text{ in}^2 \)
  - \( a = \frac{A_s f_y}{0.85} = 0.30(40) \)
    \[ a = 0.85 \times b = 0.85 \times (3)(83) \]
    \[ a = 0.85 \times 6.5 = 5.56 \text{ in} \]
  - \( d = 6 - 1.5 - (\%)(6.5) = 6.25 \text{ in} \)
  - \( E_s = 0.0376 \geq E_f = 0.0041 \text{ (o.k.)} \)

- **Negative Bending:**
  - \( A_s = (7)(0.11) = 0.77 \text{ in}^2 \)
  - \( a = \frac{A_s f_y}{0.85} = 0.77(40) \)
    \[ a = 0.85 \times b = 0.85 \times (3)(83) \]
    \[ a = 0.85 \times 6.5 = 5.52 \text{ in} \]
  - \( d = 6.25 \text{ in} \)
  - \( E_s \geq E_f \text{ (o.k.)} \)

- **Negative Shear:**
  - \( V_c = 2.1 J f_d \times b \times d \)
    \[ = 2(10)(3000)(0.30)(6.25) \]
    \[ V_c = 60,249.5 \text{ lbs} = 60,249.5 \text{ kips} \]
  - \( V_s = \frac{A_s f_y}{S} \)
    \[ = \frac{(6)(0.30)(40)(6.25)}{10} \]
    \[ V_s = 10 \text{ kips} \]

- **Moments:**
  - \( M_n = A_s f_y (d - \%d) \)
    \[ = (0.30)(40)(6.25 - 0.30)(6.25) \]
    \[ M_n = 382.745 \text{ k-in} = 44.3954 \text{ k-ft} \]
    \[ \phi M_n = 0.90 (44.3954) = 39.936 \text{ k-ft} \]

- **Positive Bending:**
  - \( M_n = 44.40 \text{ k-ft} \)
  - \( \phi M_n = 39.96 \text{ k-ft} \)

- **Negative Bending:**
  - \( M_n = 15.87 \text{ k-ft} \)
  - \( \phi M_n = 14.28 \text{ k-ft} \)

- **Negative Shear:**
  - \( V_n = V_c + V_s = 70.5475 \text{ kips} \)
  - \( \phi V_n = 0.75 (70.5475) = 52.6871 \text{ kips} \)

- **Positive Shear:**
  - \( V_n = 70.5475 \text{ kips} \)
  - \( \phi V_n = 52.69 \text{ kips} \)
Koke Head Abutment: (5ft x 30)

\[ V_e = 2 \sqrt{f_y} \cdot b \cdot d \]

\[ V_e = 2(3000)(3000)(10)(24) \]

\[ V_e = 31.548.8 \text{ lbs} \]

\[ V_n = 31.55 \text{ kips} \]

*Note: The wing wall will fail due to bottom shear and vertical bending.*

**Shear Horizontal Plane:**
- Reinforcement will not provide shear resistance.

\[ a = A_{fy} \cdot (d - \frac{a}{2}) \]

\[ a = 0.270487 \text{ in} \]

\[ d = 12 - 2 - \frac{a}{2}(0.5) = 9.75 \text{ in} \]

\[ f_{y} = 0.0286 \geq f_y = 0.014 \text{ (ok)} \]

\[ M_n = A_{fy} \cdot (d - \frac{a}{2}) \cdot (1.0)(40)(6.25 - 2.5) \]

\[ M_n = 384.19 \text{ k-in} \]

*Note! Only top reinforcement will provide flexural resistance.*
Koko Head Abutment: cont.

- Vertical Flexural Capacity: cont.

\[
M_n = 384.19 \text{ k-in}
\]

\[
F_R = \frac{M_n}{13.8} = \frac{384.19 \text{ k-in}}{13.8}
\]

\[
F_R = 27.8462 \text{ kips}
\]

\[
\Rightarrow F_R = 27.85 \text{ kips}
\]

- Total Capacity: (Koko head Wing Wall)

- Horizontal Shear = 31.56 kips

- Vertical Flexure = 27.85 kips

\[
\Rightarrow \text{Total} = 59.395 \text{ kips}
\]
Diamond Head Abutment: (slab 30)

- Horizontal Shear Plane:
  - \( V_c = 2.1 \sqrt{f_y} b d \)
    \[ V_c = 2(10)(2000)^{1/2}(12)(12 + 6.4665) \]
    \[ V_c = 24195.6 \text{ kips} \]
  - \( V_c = 24.1956 \text{ kips} \)
  - \( \Rightarrow V_n = 24.20 \text{ kips} \)

- Vertical Flexure Capacity:
  - \( M_n = 384.19 \text{ k-in} \)
    (same as koko head abutment)

- \( F_{fr} = M_n / 9.2 \)
  \[ F_{fr} = 384.19 / 9.2 \]
  \[ F_{fr} = 41.76 \text{ kips} \]
  - \( \Rightarrow F_{fr} = 41.76 \text{ kips} \)
- Diamond Head Abutment: cont.
  - Total Capacity: (Diamond Head Wing Wall)
    - Horizontal Shear = 24.20 kips
    - Vertical Flexture = 41.76 kips
    - Total = 65,956 kips

- Both Wing Walls:
  Total Capacity = Koko + Diamond
  = 59,396 + 65,956
  Total Capacity = 125,352 kips

  - Total Lateral Capacity = 125,352 kips

  Wing Walls
Girder Seat Bearing Pad: (Sheet 30 & 32)

*Note: The stub in the top and bottom of the girder seat have adequate reinforcement preventing concrete failure. Failure will be due to welds, steel, or stud failure.

ACI 318-08 section RD 6.2.9

Properties:
- Studs: $F_y = 40\,\text{ksi}$
- Steel: $F_y = 50\,\text{ksi}$
- Welds: Assume $E_{30}$

Fu = 60 ksi
Bearing Lateral Capacity:

0 Steel Shear Strength: (2 - 3/8")

\[ f_c' = 6000 \text{ psi girders} \]
\[ f_y = 3000 \text{ psi bottom} \]

- For girders:
  1) \[ Q_n = 0.5 \frac{A_{sc} \sqrt{f_c' E_c}}{L} \leq R_y \frac{R_p A_{sc} F_u}{L} \]
  2) \[ A_{sc} = (\frac{3}{4})(\frac{3}{4})^3 = 0.441736 \text{ in}^3 \]
  3) \[ E_c = f_c' \sqrt{f_c'} = (6000)^{1.5} \] in.
  \[ E_c = 4800 \text{ kips} \]

\[ Q_n = 0.5 \frac{A_{sc} \sqrt{f_c' E_c}}{L} \]
\[ = 0.5 \cdot (24,408.7)(6000) \]
\[ Q_n = 24,4087 \text{ kips} \]

- For bottom:
  1) \[ E_c = a_{sc} \sqrt{f_c' E_c} \]
  \[ = (3000)^{1.5} \]
  \[ E_c = 3181.98 \text{ kips} \]

\[ Q_n = 0.9 \frac{A_{sc} \sqrt{f_c' E_c}}{L} \]
\[ = 0.9 \cdot (24,408.7)(3181.98) \]
\[ Q_n = 21,582 \text{ kips} \leq R_y \frac{R_p A_{sc} F_u}{L} \]
\[ \therefore Q_n = 21,582 \text{ kips} \]

- Studs in bottom concrete controls:

\[ Q_n = 43,164 \text{ kips/bearing} \]

8 bearings at Diamond Head Abutment Only
B Bearing Lateral Capacity: Cont.

1. Weld Strength:
   - Weld: 3/16"
   - F 70
   - 18" length total

2. \( \phi R_n = \phi (0.707 \times (0.6 F_{u b})) \)
   \( = 0.76 (0.707 (3/16)(0.6 (70))) \)
   \( \phi R_n = 8.35144 \text{ kips/lin} \)

3. For entire length:
   \( \phi R_n = 8.35144 (18) = 150.326 \text{ kips} \)
   \[ \therefore R_n = 200.435 \text{ kips} \]
   
   Weld Strength

* Base Metal Shear Strength (along weld)

4. \( \phi R_n = \min \left[ 1.0 (0.6 F_{u b}), 0.75 (0.6 F_{u b}) \right] \)

5. \( 1.0 (0.6 F_{u b}) = 1.0 (0.6 (50)) (3/4) \)
   \( = 22.5 \text{ kips/lin} \)

6. \( 0.75 (0.6 F_{u b}) = 0.75 (0.6 (50)) (3/4) \)
   \( = 21.94 \text{ kips/lin} \) \( \therefore \) cont'd

7. \( \phi R_n = 8 (18) = 394 \text{ kips} \) \( \text{(for entire length)} \)
   \[ \therefore R_n = 394 \text{ kips} \]
   
   Base Metal Shear Strength (Along welds)
Bearing Lateral Capacity:

- Bolt Block Shear:

\[ R_n = 0.6 F_{u_n} A_{nv} + U_{bs} F_{u_Ant} \leq 0.6 F_y A_{gvr} + U_{bs} F_{u_Ant} \]

\[ A_{gvr} = (\frac{3}{4})(4.5\text{ in})(1.0) = 12.375 \text{ in}^2 \]

\[ A_{nv} = (\frac{3}{4})(16.5\text{ in}) - (\frac{3}{4})(\frac{1}{8})(1.0) = 11.513 \text{ in}^2 \]

\[ A_{Ant} = (\frac{3}{4})(16.5\text{ in}) - (\frac{3}{4})(\frac{3}{4})(\frac{1}{8}) = 10.0435 \text{ in}^2 \]

\[ R_n = 0.6 F_{u_n} A_{nv} + U_{bs} F_{u_Ant} \leq 0.6 F_y A_{gvr} + U_{bs} F_{u_Ant} \]

\[ R_n = 650 \text{ kips} \]

\[ R_n = 572.34 \text{ kips} \]

Weld Block Shear:

- Bolt Block Shear:

\[ A_{gvr} = A_{nv} = 0 \text{ (No longitudinal welds)} \]

\[ A_{Ant} = t_{member}(18) \]

\[ = (\frac{3}{4})(18) = 13.5 \text{ in}^2 \]

\[ R_n = U_{bs} F_{u_Ant} \]

\[ = (1.0)(65\text{ kips/ft})(18\text{ ft}) \]

\[ R_n = 877.5 \text{ kips} \]

\[ R_n = 877.5 \text{ kips} \]
Vertical Resistance:

Concrete Break-out Strength of Anchor in Tension (ASD Section 8.5.2)

- **Bottom Concrete:**
  \[ f_c' = 3000 \text{ psi} \]

\[ 30" \]

\[ 2.8125" \]

\[ 1.1875" \]

\[ 8.125" \]

\[ 1.1875" \]

\[ 1.6667" \]

\[ \sin(18) = \frac{\sin(7\pi)}{1.1875} \]

\[ x = 5.1935 \text{ in} \]

\[ D \quad A_{nc} = (30)(2.8125 + 9 + 4.5) - 2[(4)(1.6)(5)3] = 490.61 \text{ in}^2 \]

\[ D \quad A_{nto} = 9 \text{ kcf}^2 = 9(6)^2 = 324 \text{ in}^2 \]

\[ D \quad V_{ec, w} = 1.0 \text{ (Not loaded economically)} \]

\[ D \quad C_{x, min} = (4.5 + 2.8125) = 7.3125 \text{ in} \]

\[ \delta \quad C_{x, min} = \text{ hcf (4)} \]

\[ V_{cd, u} = 0.7 + 0.3 \text{ C_min} \]

\[ 1.5 \text{ hcf} \]

\[ 0.7 + 0.3(7.3125/4) \]

\[ V_{cd, u} = 0.94375 \]

\[ D \quad V_{c, w} = 1.25 \text{ (Cost in anchors)} \]

\[ D \quad V_{cp, u} = 1.0 \]

\[ D \quad N_b = 676152 \text{ kips} \]

\[ D \quad N_b = 676152 \text{ kips (Concrete Break-out Strength)} \]

\[ D \quad N_b = 67 \text{ kips (Bottom Concrete)} \]

\[ D \quad N_b = 38.6293 \text{ kips} \]
**Vertical Resistance: Cont.**

- Concrete Breakout Strength of Anchor in Fusion: (ACE Section 6.2)
  - Girders:
    \[ f_c' = 6000 \, \text{psi} \]
    \[ a = 6'' \]
    \[ w = 10'' \]

  1. \( A_e = (18)(18) - \frac{3}{4}(11.5/18)(2.95) = 302.3 \, \text{in}^2 \)
  2. \( A_{nc} = 9 \, \text{in}^2 = 9 \, \text{(c)}^2 = 324 \, \text{in}^2 \)
  3. \( \gamma_{ec} \cdot w = 1.0 \)
  4. \( C_{nc \, \min} = 4.5 \, \text{in} = \text{hcf} = 1.5 \, \text{(c)} = 9 \, \text{in} \)
    \[ \gamma_{cd} = 0.7 + 0.3 \left( \frac{C_{nc \, \min}}{\text{hcf} \cdot (1.5)} \right) \]
    \[ = 0.7 + 0.3 \left( \frac{9}{1.5} \right) \]
    \[ \gamma_{cd} = 0.25 \]
  5. \( \gamma_{w} = 1.25 \) (cast in anchorage)
  6. \( \gamma_{cp} \cdot w = 1.0 \)
  7. \( N_b = K_c \cdot \sqrt{f_c'} \cdot \text{hcf}^{1.5} \)
    \[ = (20)(10)(6000)^{1/2} (4)^{1/6} \]
    \[ N_b = 27,322 \, \text{lb} \]
    \[ N_b = 27,321 \, \text{kips} \rightarrow \text{For two bolts} = 54,642 \, \text{kips} \]
  8. \( N_{cb} = \left( \frac{A_{nc}}{A_{nc}} \right) \cdot \gamma_{ec} \cdot \gamma_{cd} \cdot \gamma_{w} \cdot \gamma_{cp} \cdot N_b \)
    \[ = (302.3)(9)(1.0)(1.25)(1.0)(27,322) \]
    \[ N_{cb} = 54,186 \, \text{kips} \]

\[ \therefore N_{cb} = 54,186 \, \text{kips} \]

Concrete Breakout Strength
Girders

*Note: this value is likely higher due to the reinforcement in the area.*
**Kaliouau Bridge**

**Bearing Capacity**

---

**Vertical Resistance: cont.**

- **Pull Out Strength of Anchor in Tension:** (ACI Section D.5.3)

- **Girder:**
  
  \[ N_{pm} = \gamma_{hp} N_p \]

- **Bottom:**

  \[ N_p = 8 \times A_{br} f'c_{w} \]

  - **Girder:**
    
    \[ N_p = 8 \times 3.534 \times (3 \times 10^2) = 564.6 \text{ kips} \]

  - **Bottom:**
    
    \[ N_p = 84.82 \text{ kips} \]
- Vertical Resistance: cont.

- Steel Strength of Anchor in tension: (ACI Section D.5.1)
  \[
  \begin{align*}
  \text{diameter} &= \frac{3}{4}'' \\
  f_y &= 40 \text{ ksi} \\
  f_u &= 65 \text{ ksi}
  \end{align*}
  \]

- \( N_{ca} = n \cdot A_{se} \cdot N_{fau} \)

- \( A_{se} = \left( \frac{3}{4}'' \right)^2 = 0.441296 \text{ in}^2 \)

- \( N_{fa} = n \cdot A_{se} \cdot N_{fau} = (4) \cdot (0.441296) \cdot (65) = 57.432 \text{ kips} \)

- \( N_{fa} = 57.43 \text{ kips} \quad \text{Tensile Capacity of Studs} \)

- Steel Plate Bending / Flexural Capacity:
  \[
  \begin{align*}
  f_y &= 50 \text{ ksi} \\
  f_u &= 65 \text{ ksi}
  \end{align*}
  \]
  \[
  \begin{array}{|c|c|c|}
  \hline
  \text{Moment Diagram} \\
  \hline
  \text{Fr} / 2 & \text{Fr} / 4 \\
  \hline
  \text{16.6''} \\
  \hline
  \end{array}
  \]

- \( M_{max} = \frac{Fr}{2} \times \left( \frac{16.6}{2} \right) - \frac{Fr}{4} \times (4) \)

- \( M_{max} = 1.125 Fr \)

- From beam theory:
  \[
  M = \frac{Fr}{Y}
  \]

- \( I = \frac{1}{12} \cdot b \cdot h^3 = \frac{1}{12} \cdot (16.5 \times \frac{3}{4})^3 = 0.580098 \text{ in}^4 \)

- \( y = \frac{h}{2} = \left( \frac{3}{4}'' \right) = 0.375 \text{ in} \)

- \( M = \frac{Fr}{Y} \cdot \frac{Fr}{Y} = \frac{65 \times 65}{0.375} \)

- \( M = 100.547 \text{ k-in} \)

- \( Fr = M / 1.125 = 100.547 / 1.125 \quad Fr = 89.376 \text{ kips} \)

- Plate Flexural Capacity (Both plates)

- For both plates:
  \( Fr = (2 \times 89.376) = 178.75 \text{ kips} \)

- \( Fr = 178.75 \text{ kips} \)
Summary Lateral Capacity:

For one bearing: (8 total at Diamond Head Abutment only)
- Stud Shear Strength = 43,164 kips \( \leq \) controls
- Weld Strength = 200,435 kips
- Base Metal Strength (at webs) = 394 kips
- Bolt Block Shear = 572.34 kips
- Weld Block Shear = 872.5 kips

\[ \Rightarrow \text{Total Lateral Capacity Provided:} \]

\[ H_{R} = 8(43,164) = 345,312 \text{ kips} \]
(From Bearing Beds)

\( \ast \) Note: Concrete pryout or punch out was not considered because of proper reinforcement in the area of the studs will provide adequate strength to reduce these types of failures.

Summary Vertical Capacity:

For one bearing:
- Concrete Block Out Tension (Bot Conc.) = 67,624 kips
- Concrete Block Out Tension (Girder) = 54,1816 kips \( \leq \) controls
- Pull Out Strength of Achor:
  - Girder = 169.65 kips
  - Bottom = 89.82 kips
- Steel Strength of Achor in Tension = 574.32 kips
- Steel plates flexure capacity = 178.75 kips

\[ \Rightarrow \text{Total Vertical Capacity Provided:} \]

\[ V_{R} = 8(54,1816) = 433,453 \text{ kips} \]
(From Bearing Bolt)
**Creep Block Capacity:**

- **Shear Friction:** (See Section 11.4.9)
  - \( V_n = A v t f_y \mu \)
  - \( A v t = 2(72) + 4(72) \)
    \[ = 2(240.76) + 4(240.76) \]
    \[ A v t = 2.4 \text{ in}^2 \]
  - \( \mu = 1.4 \mu = 1.4 \) (Creep block placed monolithically)

- **Check:**
  - \( 0.25 f_c' A_c = 0.25 \times 5000 \times 6 + 72 = 496 \text{ kips} > V_n \) (ok)
  - \( f_y + 0.06 f_c' A_c = (480 + 0.06 \times 5000) \times 8 \times 36 = 576 \text{ kips} > V_n \) (ok)
  - \( 1600 A_c = 1600 \times 48 \times 4.44 = 1.375 \times 10^6 \text{ lbs} > V_n \) (ok)
**Beam Capacity:**
*Note: From experience from the Maipulam Stream Bridge the girder web capacity is controlled by flexure.*

![Diagram of beam capacity](image)

*Note: The flexure plane will have the greatest moment at that location while still in the web.*

**Web Cross Section: (A-A)**

![Diagram of web cross section](image)

\[ F_y = 412,000 \text{ psi} \]

**Notes:** only top reinforcement will provide flexure resistance.

---

1. \[ A_s = (4)(0.31) = 2.79 \text{ in}^2 \]

2. \[ a = \frac{A_s f_y}{(2.71)(40)} = \frac{0.856c'}{0.85(6)(19.463)} \]
   \[ a = 0.5653 \text{ in} \]

3. \[ d = 6 - 1 - (\frac{1}{3})(0.625) = 4.6875 \text{ in} \]

4. \[ \beta = 0.85 - 0.05 \left( \frac{15}{4000} \right) \]
   \[ = 0.85 - 0.05 \left( \frac{4000}{15} \right) \]
   \[ \beta_1 = 0.75 \]

5. \[ E_s = 0.003 \left( \frac{d - 9/16}{0.75} \right) \]
   \[ E_s = 0.016 > E_y = 0.0014 \text{ (OK)} \]

6. \[ M_n = A_s f_y (d - 0.75) \]
   \[ = (0.94)(40)(4.6875 - 0.5653) \]
   \[ M_n = 192.139 \text{ k-in} \]

7. \[ M_{max} = 24 \text{ Fr} \]

8. \[ F_r = \frac{M_n}{24} = \frac{492.139}{24} \]
   \[ F_r = 20.506 \text{ kips} \text{ (controls)} \]

**Beam Web Flexural Capacity:**

![Diagram of beam web flexural capacity](image)

2 Creep Blocks at Diamond Head Abutment only

\[ \Rightarrow \text{For Both Creep Blocks:} \]

\[ F_r = 41.01 \text{ kips} \]
**Negative Bending Capacity:**

- Properties:
  - \( f' = 6000 \text{ psi} \)
  - \( P_0 = (k' \times 10^3 \times 0.3)(34) = 31750.1 \text{ kips} \)
  - \( A_{ps} = 24 \times 0.02 = 0.576 \text{ in}^2 \)
  - \( d_b = 929.336 \text{ psi} \)
  - \( f_c = -5100 \text{ psi} \)

- Centroid:
  - \( A_1 = 7.2 \text{ in}^2 \)
  - \( A_2 = 36 \text{ in}^2 \)
  - \( y_1 = 3 \text{ in} \)
  - \( y_4 = 15.83 \text{ in} \)
  - \( A_5 = 108 \text{ in}^2 \)
  - \( y_5 = 18 \text{ in} \)
  - \( y_6 = 18 \text{ in} \)

- Moment of Inertia:
  - \( I_1 = \frac{h^2}{6} b h = \frac{(12)(12)}{6} \)
  - \( I_1 = 216 \text{ in}^4 \)
  - \( I + A d^2 = 216 \times (21/19.19)^2 = 21442.2 \text{ in}^4 \)
  - \( I_2 = \frac{h^3}{6} = (21/6)^2 = 2.85 \text{ in}^4 \)
  - \( I + A d^2 = 2.25 + 4.5(12.17)^2 = 782.77 \text{ in}^4 \)
  - \( I_3 = \frac{h h_1}{6} = \frac{h \times (9)}{6} \)
  - \( I_3 = 6912 \text{ in}^4 \)
  - \( I + A d^2 = 6912 + (144)(2.17)^2 = 7590.08 \text{ in}^4 \)
  - \( I_4 = \frac{b h^3}{6} = \frac{6}{6} \times 36 = 36 \text{ in}^4 \)
  - \( I + A d^2 = 36 + 36(2.85)^2 = 2448.12 \text{ in}^4 \)
  - \( I_5 = \frac{1}{12} b h^3 = \frac{1}{12} (12)(12)^3 = 12 \text{ in}^4 \)
  - \( I + A d^2 = 324 + (108)(12.17)^2 = 18101.8 \text{ in}^4 \)
I. Negative Bending Capacity:

- Geometric Properties
  - $A_c = 369$ in$^2$
  - $I_c = 63,185.9$ in$^4$
  - $C_b = 20.17$ in
  - $S_b = 2636.98$ in$^3$
  - $C_b = 15.83$ in
  - $S_b = 3351.82$ in$^3$
  - $V = 144.14$ in$^2$
  - $C_c = 10.41$ in
  - $L = 580.68$ in

II. **$W_0$:**

- Self weight = $(2.5625)(160) = 384.375$ lb
- Topping = $(3/4)(5 + 3/4)(150) = 525$ lb

\[ W_0 = 384.375 + 525 = 909.375 \text{ lb} \]

\[ W_0 = 75.781 \text{ lb/in} \]

III. Tensile Limit: (unsubmerged)

\[ W_{te} = \frac{8}{D^2} \left( M_0 + S_b F_e - \frac{S_b P_e}{A_c} \left( \frac{C_c}{V^2} - 1 \right) \right) \]

\[ M_0 = \frac{w_0 L^4}{8} \]

\[ M_0 = 2.19408 \times 10^6 \text{ lb-in} \]

\[ W_{te} = \frac{8}{(580.68)^2} \left( 3.19408 \times 10^5 + (2636.98 \times 929.516) - (3351.82 \times 3126.1) \right) \left( \frac{10.41 \times 20.12}{144.14} - 1 \right) \]

\[ W_{te} = 109.347 \text{ lb/in} \]

\[ F_{te} = 63.49 \text{ kips (tensile limit)} \]

IV. Compression Limit: (unsubmerged)

\[ W_{ce} = \frac{8}{D^2} \left( M_0 - S_b F_e - \frac{S_b P_e}{V^2} \left( 1 + \frac{S_b}{C_c} \right) \right) \]

\[ M_0 = \frac{w_0 L^4}{8} \]

\[ M_0 = 2.19408 \times 10^6 \text{ lb-in} \]

\[ W_{ce} = \frac{8}{(580.68)^2} \left( 3.19408 \times 10^6 - (2636.98)(-5100) - (3351.82 \times 3126.1) \right) \left( 1 + \frac{10.41 \times 20.12}{144.14} \right) \]

\[ W_{ce} = 335.308 \text{ lb/in} \]

\[ F_{ce} = 194.71 \text{ kips (compression limit)} \]
**Negative Bending Capacity:**

**IV) Submerged:**
- Assume 50% air pocket
  \[ w_0 = 909.876 - 6.0625 (64) - 6.9(64) = 143.775 \text{ lb} \]
  \[ w_0 = 11.9813 \text{ lb/in} \]
  \[ M_0 = 504,994 \text{ lb-in} \]

**V) Tensile Limit:**
  \[ w_{ut} = \frac{8}{3} \left[ M_0 + 1.41475 \times 10^6 \right] \]
  \[ w_{ut} = 45.5471 \text{ lb/in} \]
  \[ F_{ut} = 26,448 \text{ kips (tensile)} \]

**V) Compression:**
  \[ w_{uc} = \frac{8}{3} \left[ M_0 + 1.09387 \times 10^6 \right] \]
  \[ w_{uc} = 271.508 \text{ lb/in} \]
  \[ F_{uc} = 157.66 \text{ kips (compression)} \]

**Summary of Results:**

- **Unsubmerged:**
  - Tensile: \( F_{ut} = 26,448 \text{ kips} \)
  - Compression: \( F_{uc} = 194.71 \text{ kips} \)

- **Submerged:**
  - Tensile: \( F_{ut} = 26,448 \text{ kips} \)
  - Compression: \( F_{uc} = 157.66 \text{ kips} \)

- **Loss:**
  \[ T = 58.34\% \]
  \[ C = 19.03\% \]
**Kahaluu Bridge Structural Analysis**

- **Buoyancy Calculations:***
  - **Typical Girder Section: (sheet 17)**

  ![Diagram of a girder section]

  - \[ A_1 = (1 + \frac{1}{2})(\frac{3}{2}) = 1.111 \text{ ft}^2 \]
  - \[ A_2 = 2 \left( \frac{1}{2} \times \frac{1}{2} \times \frac{1}{2} \right) = 0.25 \text{ ft}^2 \]
  - \[ A_3 = (\frac{1}{2}) (\frac{1}{2} + 1 + \frac{1}{2} + \frac{3}{4}) = 2.111 \text{ ft}^2 \]
  - \[ A_4 = 2 \left[ \frac{1}{2} \times \frac{1}{2} \times \frac{1}{2} \right] = 0.5625 \text{ ft}^2 \]
  - \[ A_5 = (1 + \frac{1}{2}) \left( \frac{1}{2} \right) = 1.444 \text{ ft}^2 \]
  - \[ A_T = \frac{6}{4} A_1 = 5.47917 \text{ ft}^2 \]
  - \[ A_T = 5.47 \text{ ft}^2 \] **Girder Section**

  - **Deck Section: (sheet 6)**

  ![Diagram of a deck section]

  - **Concrete:**
    - \[ A_1 = (46)(\frac{3}{4}) = 23 \text{ ft}^2 \]
    - \[ A_2 = 2(\frac{3}{4})(1 + \frac{1}{2}) = 4.25 \text{ ft}^2 \]
    - \[ A_3 = 2 \left[ \left( \frac{1}{4} \times \frac{3}{4} \right) \right] = 5 \text{ ft}^2 \]
    - \[ A_T = \frac{3}{4} A_1 = 32.25 \text{ ft}^2 \]
    - \[ A_T = 32.25 \text{ ft}^2 \] **Concrete Deck**

  - **AC Pavement:**
    - \[ A_T = (36)(1.5) = 4.5 \text{ ft}^2 \]
    - \[ A_T = 4.5 \text{ ft}^2 \] **AC Pavement**

- **Calculations By:** Daniel Lunn

---

A-30
Section: Thru Abutment Diaphragm

Length = 1 ft (Thickness)

Area = \((41 + \frac{7}{6} \times \frac{5}{4})\) = 187.125 ft\(^2\)

\[ A_t = 187.13 \text{ ft}^2 \] Thru Abutment Diaphragm

Section: Thru Pier Cap Diaphragm

Length = 1 ft (Thickness)

Area = \((41 + \frac{7}{6} \times \frac{5}{2})\) = 180.199 ft\(^2\)

\[ A_t = 180.19 \text{ ft}^2 \] Thru Pier Cap Diaphragm

Section: Intermediate Diaphragm

Length = 1 ft (Thickness)

* Diaphragm occupies area between girders

Area of girders = \(7 (1.11 + 0.25 + 2.11 + 0.5625) = 28.243 \text{ ft}^2\)

Area = \((40 + \frac{7}{6} \times \frac{1}{4}) - 28.243 = 128.604 \text{ ft}^2\)

\[ A_t = 128.60 \text{ ft}^2 \] Intermediate Diaphragm
Air Packet Calculations:
* Note: Volume of trapped air is dependent on Thru Abutment Diaphragm, as it has the deepest depth.

\[ A_1 = (4 + \frac{1}{2})(\frac{9}{2}) = 2.722 \text{ ft}^3 \]
\[ A_2 = (5 + \frac{1}{2})(3 + \frac{1}{2}) = 16.092 \text{ ft}^2 \]
\[ A_3 = 2 \left[ \frac{1}{2} \left( \frac{9}{2} \right) \left( \frac{9}{2} \right) \right] = 0.25 \text{ ft}^2 \]
\[ A_4 = 2 \left[ \frac{1}{2} \left( \frac{8}{2} \right) \left( \frac{8}{2} \right) \right] = 0.645 \text{ ft} \]
\[ A_5 = (3 + \frac{1}{2})(\frac{9}{2}) = 2.388 \text{ ft}^2 \]

\[ A_T = A_1 + A_2 - A_3 - A_4 + A_5 \]
\[ = (2.72) + (16.09) - (0.25) - (0.645) + (2.388) \]
\[ A_T = 20.3958 \text{ ft}^3 \]

Reduction of Air Packet:
* Assume the bridge is submerged to top of deck

\[ h = (1.5\frac{1}{2}) + (\frac{1}{2}) + (\frac{59}{12}) = 5.125 \text{ ft} \]
\[ \text{At Pave} \quad \text{Deck} \quad \text{to bot of diaphragm} \]

\[ P_2 = 14.7 + h \left( \frac{64}{44} \right) \]
\[ = 14.7 + (5.125) \left( \frac{64}{44} \right) \]
\[ P_2 = 16.978 \text{ psi} \]

\[ A_2 = \frac{P_2 A_1}{P_2} = \frac{(14.7)(30.958)}{16.978} \]
\[ A_2 = 17.6595 \text{ ft}^3 \]

\[ A_T = 17.66 \text{ ft}^2 \] Compressed Air Packet
A Collection of Data:

- Concrete: ($fc = 150 \text{ psi}$)
  - Typical Girder:
    - Area = 5.48 $\text{ft}^2$
    - Amount = 8
    - Length = 318 ft
  - Deck:
    - Area = 32.25 $\text{ft}^2$
    - Amount = 1
    - Length = 318 ft

- AC Pavement: ($Y_{ac} = 152 \text{ kips/ft}^2$)
  - Area = 4.5 $\text{ft}^2$
  - Amount = 1
  - Length = 318 ft

- Air Pocket: ($\text{mass} = 64 \text{ kips}$)
  - Area = 17.66 $\text{ft}^2$
  - Amount = 7
  - Length = 310 ft

A Total self weight:

- Self weight = \[
(7)(\text{girder})(L) + (1)(\text{Deck})(L) + (2)(\text{Ac. Dia.})(L) + (2)(\text{Pier Dia.})(L) + (6)(\text{Int. Dia.})(L)\] \[\times Y_{ac} + (1)(4)(L)\] \[\frac{Y_{ac}}{150}
+ (1)(4.5)(318)(152)
\]

\[\text{Self weight} = 3.81155 \times 10^6 \text{kips}\]

A Buoyant Force:

- Submerged Volume = Air Pocket + Submerged concrete (Assume submerged to top of deck)
  \[\approx \pi (17.66)(310) + (23.96 \times 2 - 9.25(318) + 1431)\]
  \[\approx 50779.8 \text{ ft}^3\]

- Buoyant Force = $SV \times Y_{sw}$
  \[= (50779.8 \times 64)\]
  \[BF = 3.38935 \times 10^6 \text{kips}\]

A Summary:

- Self weight = 3812 kips
- Buoyant Force = 3389 kips
- Residual Weight = -77.3 kips (-2.0% of original self weight)

\[\therefore \text{Bridge is buoyant}\]
Kahaluu Bridge Deck Capacity

Deck Moment Capacity:

- Positive Bending Capacity:
  - \( d = 6 - 1 - 0.75g = 4.425 \text{ in} \)
  - \( A_0 = 3(0.44) = 1.32 \text{ in}^2 \)
  - \( T = A_0 f_y = (1.32)(60) = 79.2 \text{ kips} \)
  - \( T = \frac{A_0 f_y}{\alpha} = \frac{(29.2)}{0.85(4.4)(44)} = 0.4754 \text{ in} \)
  - \( f_y = 40000 \text{ psi} \)
  - \( \beta_1 = 0.85 \)
  - \( C = \frac{\sigma}{\beta_1} = 0.6593 \text{ in} \)
  - \( \varepsilon_0 = \left( \frac{d - C}{C} \right) \varepsilon_{cu} = (4.625 - 0.56)(0.003) = 0.0218 \)
  - \( \varepsilon_y = \frac{f_y}{E_y} = \frac{60}{29.000} = 0.00207 \)
  - \( \varepsilon_y \geq \varepsilon_y \) o.k.

- \( M_n = A_0 f_y (d - \frac{3}{4}) = (79.2)(4.625 - 0.4754) \)
  - \( M_n = 347.474 \text{ k-ft} \)
  - \( M_n = 28.96 \text{ k-ft} \)
  - Flexure Reduction Factor = 0.90
    - \( M_n = 26.06 \text{ k-ft} \)

- Negative Bending Capacity:
  - \( d = 6 - 1.5 - 0.75g = 4.125 \text{ in} \)
  - \( \varepsilon_0 = \left( \frac{d - C}{C} \right) \varepsilon_{cu} = (4.125 - 0.56)(0.003) = 0.0216 \)
  - \( \varepsilon_y = \frac{f_y}{E_y} = \frac{60}{29.000} = 0.00207 \)

Typical Span Positive Bending Capacity

Typical Span Negative Bending Capacity
Deck Shear Capacity:

\[ d = 4.625 \text{ in} \]
\[ b = 49 \text{ in} \]

\[ V_c = 2.1 \sqrt{V_e} b w d \]
\[ = 2(1.0)(4000)^{0.5}(49)(4.625) \]
\[ V_c = 28666 \text{ lbs} \]

\[ \phi V_c = 0.75(28666) \]
\[ = 21499.5 \text{ lbs} \]
\[ \phi V_c = 21,499.5 \text{ kips} \]

\[ \phi V_c = 21.5 \text{ kips} \]
Wing Wall Capacity:

- #5 bars @ 10°
- #7 bars @ 15°

$V_c = 2.5 \sqrt{f_c' b e d}$

$V_c = 2.5 \sqrt{4000 \times 18 \times 26}$

$V_c = 59197.8$ lbs $= 59.1978$ kips

$\phi V_c = 0.75 \times 59197.8 = 44398.4$ lbs

$\Rightarrow \phi V_c = 44398$ kips

- Shear Failure: (Bottom Plane)

$V_c = 2.5 \sqrt{f_c' b e d}$

$V_c = 2.5 \sqrt{4000 \times 18 \times 54}$

$V_c = 122949$ kips

$\phi V_c = 0.75 \times 122949 = 92212$ kips

$\Rightarrow \phi V_c = 92212$ kips
Wing Wall Capacity

- Flexure Capacity:

\[ M = F_r (9.25 + 9.5) \]
\[ M = 17.75 F_r \]

1) Moment at fixed end:

\[ M = F_r (9.25 + 9.5) \]
\[ M = 17.75 F_r \]

1 Method 0:

\[ M_n = F_r (17.75) \]
\[ F_r = \frac{1.63913 \times 10^6 \text{ lb-in}}{17.75} \]
\[ F_r = 92.345 \text{ lb} \]

\[ F_r = 92.345 \text{ kips} \] Flexure Capacity

2 Method 0:

\[ M_n = \omega \frac{F_r}{2} (1 - 0.59 \omega) b d^2 \]
\[ J = A_c = (6 \times 0.21) \]
\[ A_c = (18 \times 54) \]
\[ J = 0.01914 \]

\[ J = 0.01914 \]
\[ \omega = 1 / 4 \]
\[ \omega = 0.02871 \]

\[ J = 0.8 \]
\[ J = 0.8 \times 10^{-6} \]
\[ J = 0.02871 \]

\[ M_n = \omega \frac{F_r}{2} (1 - 0.59 \omega) b d^2 \]
\[ = (0.02871 \times 4 \times (1 - 0.59 \times 0.02871)) \times (64 \times 18^3) \]
\[ M_n = 1975.21 \text{ k-in} \]

\[ F_r = \frac{M_n}{17.75} \]
\[ F_r = 111.279 \text{ kips} \]
Kahaluu Bridge

Wing Wall Capacity

- Flexure Capacity:
  
  1. Method (j): 

  \[
  a = A_{fy} \left( \frac{6 \times 0.31 \times 80}{0.85 \times 6} \right) 
  \]
  \[
  a = 0.85 (4) \times 54 
  \]
  \[
  a = 407.64 \text{ in} 
  \]

  \[d = 18 - 3 - 0.025\%
  \]
  \[d = 14.6875 \text{ in} \]

  \[M_n = A_{fy} (d - 9\%) 
  \]
  \[M_n = 111.6 (14.6875 - 0.607643) 
  \]
  \[M_n = 1605.21 \text{ k-in} \]

  \[F_R = \frac{M_n}{17.75} = \frac{1605.21}{17.75} 
  \]
  \[F_R = 90.43 \text{ kips} \] - Most Conservative

  **Summary of Results (Wing Wall):**

  - Shear Horizontal Plane = 59.2 kips
  - Shear Vertical Plane = 122.9 kips
  - Flexure Capacity = 90.43 kips

  :: Bottom will shear, then will fail in flexure:

  \[\Rightarrow \text{Capacity} = 149.63 \text{ kips} \] per abutment
**Bearing Capacity** (Sheet Bs-9)

*Note:* Each girder sits on a bearing pad at each abutment
- 16 total Bearing pads

*Note:* The bearing pads do not provide vertical resistance.
- Only lateral capacities are computed

---

### Steel Capacities:

1. **Side Steel Restraints:** (.62" x 0.5")
   - Metal Shear Strength
     - \( F_y = 36 \text{ ksf} \)
     - Length = 9 in (ena. side)
     - \( F_u = 98 \text{ ksf} \)

2. \( V_n = 0.6 \frac{E}{F_y} A_C u \)

3. \( 2.24 \left( \frac{E}{F_y} \right)^{1/2} = 2.24 \left( \frac{29,000}{36} \right)^{1/2} \)
   - \( = 63.5764 \)

4. \( b / h = \frac{0.62}{1.5} = 0.414 \)

   \( \therefore \) \( b / h < 2.24 \left( \frac{E}{F_y} \right)^{1/2} \)

   → Strength is governed by shear yielding
   - \( C_V = 1.0 \)

5. \( V_n = 0.6 \frac{E}{F_y} A_C u \)
   - \( = 0.6 \left( \frac{36}{0.5} \right) (0.5 x 9) (1.0) \)
   - \( V_n = 97.2 \text{ kips} \)

   → \( V_n = 97.2 \text{ kips/Bearing Pad} \)
**Kahalu Bridge**

**Bearing Capacity: Lateral Resistance**

**A. Skew Capacities:**

- **Nelson Stud Shear Capacity:**
  - **Assumptions:**
    1. Studs are stainless steel (as formed)
    2. Thread diameter \( \frac{1}{2} \) - 13 UNC \( \Rightarrow A_b = 0.1916 \text{ in}^2 \)

- From Nelson Charts:
  Shear Strength = 7982 lbs = 7982 kips

  \[ V_n = 7,982 \text{ kips/stud} \]

**B. Block Shear:**

- **Block Shear:**
  
  1. **Stud Shear:**
      
      \[ A_{ys} = \frac{t}{b} \text{member (length x shear x bolt lines)} \]
      
      \[ A_{ys} = \left( \frac{1}{3} \right)(12,125 \times 3) \]
      
      \[ A_{ys} = 18.175 \text{ in}^2 \]

  2. **Anchor:**
      
      \[ A_n = \frac{t}{b} \text{member (length x hole diameter x bolt lines)} \]
      
      \[ A_n = \left( \frac{1}{3} \right)(12,125 - (0.6)(0.5) + 0.5)) \times 3 \]
      
      \[ A_n = 18.78 \text{ in}^2 \]

  3. **Anchor:**
      
      \[ A_n = \left( \frac{1}{3} \right)(200 - 30)(0.5 + 0.5) \]
      
      \[ A_n = 11.625 \text{ in}^2 \]

  4. **Rn:**
      
      \[ R_n = 0.6 F_y A_{ys} + U_b E_u A_{us} \]
      
      \[ R_n = 0.6(35)(16.175) + 10(58)(11.625) \]
      
      \[ R_n = 1258.24 \text{ kips} \]

      or

      \[ R_n = 0.6 F_y A_{ys} + U_b E_u A_{us} \]
      
      \[ R_n = 0.6(35)(16.175) + 10(58)(11.625) \]
      
      \[ R_n = 1067.1 \text{ kips} \]

\[ \Rightarrow R_n = 1067.1 \text{ kips} \]

**Block Shear**
Weld Capacity:

- E70 Fillet Weld: (1/4" = 5/6")
  1) \( \phi R_n = 1.39 \times 0.25 \)
      \( = 1.392 \times 0 \)
      \( = 5.664 \text{ kips/lin} \)

2) Weld is 8" long on 4 sides:
   \( \phi R_n = (5.664 \times 8 \times 4) = 178.176 \text{ kips} \)

 Base Metal Capacity:

1) \( \phi R_n = 0.6 F_y t = 0.6 \times 36 \times (1/2) \)
   \( \phi R_n = 10.8 \text{ kips/lin} \)

2) Yield Strength:
   \( \phi R_n = 0.45 F_y t = 0.45 \times 36 \times (1/2) \)
   \( \phi R_n = 13.05 \text{ kips/lin} \)

3) Yield Control: (For 32")
   \( \phi R_n = 10.8 \times (32) = 345.6 \text{ kips} \)
   \( \phi R_n = 345.6 \text{ kips} \)
Concrete Capacity:
- Bolts in Grider: $f_y = 6000 \text{ psi}$

- Using ACI 318-08 Appendix D (Section D.6.2):
  1. $V_{bg} = (\frac{4}{2} f_{vcu}) \frac{h_{a}}{d_{a}} \frac{d_{a}}{h_{a}} = V_{bc}$
  2. $C_{at} = 8.5 \text{ in}$
  3. $A_{bc} = (34.25)(12.75) = 438.688 \text{ in}^2$
  4. $V_{bc} = \frac{4.5}{2} C_{at}^2$
     $= 4.5(8.5)^2$
     $A_{bc} = 32.5 \text{ in}^2$
  5. Section D.6.2.1:
     $L_{e} = \frac{h_{a}}{2} = 8 \text{ in}$
     $d_{a} = \frac{1}{2} \text{ in}$
     $V_{bc} = \left( 7 \left( \frac{5}{2} \right) \frac{V_{bc}}{V_{bc}} \right) \lambda \sqrt{C_{at}} (C_{at})^{1.5}$
     $= (7 \left( \frac{5}{2} \right) \frac{V_{bc}}{V_{bc}}) (10) (2000) \lambda (0.5)^{1.5}$
     $V_{bc} = 16542.9 \text{ lbs}$
  6. Anchors are not loaded eccentrically:
     $V_{bc} = 1.0$
  7. $C_{at} = 12.5 < 1.5 C_{at} = 12.75$
     $V_{bc} = 0.7 + 0.3 C_{at} (1.5 C_{at})$
     $= 0.7 + 0.3 (12.5)$
     $= 9.94$

- Note: The back row will cause concrete break out.
  Front row will affect shear.

- Total Capacity:
  $V_{n} = V_{bc} + 3 \left( \text{stud shear} \right)$
  $= 39040 + 3(7982)$
  $= 62989 \text{ kips}$
**Concrete Capacity:**

- **Both in Abutment:** $f_c = 4000 \text{ psf}$

\[ \begin{align*}
\bar{u} & = 8.5 \text{ in} \\
\bar{h} & = 1.5(8.5) = 12.75 \text{ in} \\
A_{uc} & = (2.6 \times 12.75) = 33.15 \text{ in}^2 \\
A_{u0} & = 4.5 \bar{c}_n = 325.12 \text{ in}^2 \\
L_e & = \bar{h} - \bar{u} = 9 \text{ in} \\
d_o & = \frac{1}{2} \text{ in} \\
V_b & = (\bar{u} \left( \frac{d_o}{2} \right)^{0.3} V_{d0}) \alpha \sqrt{C'} \left( \bar{c}_n \right)^{0.5} \\
& = (2 \times (16)^{0.3} \left( \frac{1}{2} \right)^{0.3})(10) \times (4000) \frac{V}{f_c} \left( \bar{c}_n \right)^{0.5} \\
V_b & = 13507.2 \text{ kips} \\
C_{ai} & = 12.5 \leq 1.5 \bar{c}_n = 12.75 \\
V_{cd} & = 0.944 \\
V_{cu} & = 1.4 \\
h_b & = 8 \text{ in} \leq 1.5 \bar{c}_n \\
V_{h,v} & = 1.26244 \\
V_{cd} = (\bar{u} / \bar{c}_n) V_{c0}, V_{c0}, V_{c}, V_{h}, V_b \\
& = (33.15 \div 12.75)(10) \times 4000 \times 1.4 \times 126244 \times 13507.2 \\
V_{cd} & = 24194.9 \text{ kips} \\
\end{align*} \]

**Total Capacity:**

\[ \begin{align*}
V_n & = V_{cd} + 3 (\text{stud shear}) \\
& = 24194.9 + 3(2982) \\
V_n & = 48,140.9 \text{ kips} \\
\end{align*} \]
Summary:

- Side Steel Capacity = 97.2 kips
- Nelson Studs = 47.892 kips (shear capacity)
- Block Shear (Steel) = 1067.1 kips
- Weld Capacity = 178.2 kips
- Concrete Girder Capacity = 62.94 kips
- Concrete Abutment Capacity = 48.14 kips (likely failure)

* Note: Two bearing pads at far end will fail due to concrete abutment. The remaining 14 will fail due to Nelson Studs.

\[ \text{Total Resistance} = 2(48.14) + 14(47.892) \]

\[ TR = 766.77 \text{ kips} \]

16 Bearing pads (lateral resistance)
**Negative Bending Capacity:**

![Diagram of a beam](image)

- **Properties:**
  \[ f_c' = 6,000 \text{ psi}; \]
  \[ f_{y} = 40,000 \text{ psi}; \]
  \[ A_p = 40 \times 1.153 = 6.12 \text{ in}^2 \]
  \[ f_{pu} = 270,000 \text{ psi}; \]
  \[ P_e = (497,000)(0.3)(6.12) = 1,566,800 \times 10^6 \text{ lbs} \]

- **Centroid:**
  \[ A_1 = 1.11 \text{ ft}^2 \]
  \[ A_2 = 0.25 \text{ ft}^2 \]
  \[ A_3 = 2.11 \text{ ft}^2 \]
  \[ A_4 = 0.5625 \text{ ft}^2 \]
  \[ y_1 = 50 \text{ in} \]
  \[ y_2 = 34 \text{ in} \]
  \[ y_3 = 27 \text{ in} \]

- **Moment of Inertia:**
  \[ I_1 = \frac{1}{6} bh^3 = \frac{1}{6} (40)(8)^3 = 853.3 \text{ in}^4 \]
  \[ I_2 = \frac{1}{12} bh^3 = \frac{1}{12} (40)(6)^3 = 36 \text{ in}^4 \]
  \[ I_3 = \frac{1}{12} bh^3 = \frac{1}{12} (40)(3)^3 = 36581.3 \text{ in}^4 \]
  \[ I_4 = \frac{1}{12} bh^3 = \frac{1}{12} (40)(9)^3 = 3840 \text{ in}^4 \]
  \[ I_5 = \frac{1}{12} bh^3 = \frac{1}{12} (40)(12)^3 = 1109.33 \text{ in}^4 \]
  \[ I_6 = \frac{1}{12} bh^3 = \frac{1}{12} (40)(18)^3 = 90536.9 \text{ in}^4 \]
  \[ I_c = 102979 + (497,000)(0.3)(6.12) + 36140.9 + (2)(270,000) + 90536.9 \]
  \[ I_c = 260,730 \text{ in}^4 \]
Unsubmerged Case:

Geometric Properties:

- $A_c = 787.12 \text{ in}^2$
- $J_c = 260.730 \text{ in}^4$
- $C_6 = 21.265 \text{ in}$
- $C_6 = 24.785 \text{ in}$
- $r^2 = 330.496 \text{ in}^4$
- $f_t = 429.516 \text{ psi}$
- $f_c = -5100 \text{ psi}$
- $P_c = 1.1568 \times 10^6 \text{ lb}$

1) $W_D = \text{Self weight} + \text{Topping}$

\[
W_D = (5.48)(120) + (5+\frac{3}{4})(\frac{6}{12})(150) + (5+\frac{3}{4})(1\frac{1}{2})(152)
\]

\[
W_D = 1362.5 \text{ lb} \text{ ft}
\]

\[
W_D = 111.34 \text{ lb/in}
\]

\[
M_0 = w_D L^2/8 = 2.29632 \times 10^3 \text{ lb-in}
\]

II) Tensile Limit:

\[
\sigma_{wt} = \frac{W_t}{L^2} \left[ \frac{M_0 + s_b f_t - s_b P_c}{A_c} \right] \left( \frac{r^2}{r^4} - 1 \right)
\]

\[
\sigma_{wt} = \frac{9}{8} \left[ \frac{M_0 - 2.1136 \times 10^6}{A_c} \right]
\]

\[
\sigma_{wt} = 108.08 \text{ lb/in}
\]

Fur = 131.13 kips (tension)

III) Compressive Limit:

\[
\sigma_{wc} = \frac{W_c}{L^2} \left[ \frac{M_0 - s_b f_t - s_b P_c}{A_c} \right] \left( 1 + \frac{C_c b^2}{r^2} \right)
\]

\[
\sigma_{wc} = \frac{9}{8} \left[ \frac{M_0 + 1.4586 \times 10^7}{A_c} \right]
\]

\[
\sigma_{wc} = 185.72 \text{ lb/in}
\]

Fur = 236.238 kips (compression)

Submerged Case:

*Assume 50% air pocket

1) $W_D = 13.625 - (9.07375)(64) - 8.83(64) = 216.66 \text{ lb}$

\[
W_D = 18.058 \text{ lb/in}
\]

\[
M_D = 3.65169 \times 10^6 \text{ lb-in}
\]

II) Tensile Limit:

\[
\sigma_{wt} = \frac{W_t}{L^2} \left[ \frac{M_0 - s_b f_t - s_b P_c}{A_c} \right]
\]

\[
\sigma_{wt} = 97.608 \text{ lb/in}
\]

Fur = 9.671 kips (tension)

III) Compressive Limit:

\[
\sigma_{wc} = \frac{W_c}{L^2} \left[ \frac{M_0 - s_b f_t - s_b P_c}{A_c} \right]
\]

\[
\sigma_{wc} = 90.9367 \text{ lb/in}
\]

Fur = 114.978 kips (compression)
Summary of Results:

- Unsubmerged:
  - Tensile: $F_{tu} = 131.13$ kips
  - Compression: $F_{cu} = 236.238$ kips

- Submerged:
  - Tensile: $F_{tu} = 96.710$ kips
  - Compression: $F_{cu} = 114.78$ kips

- Loss:
  - Tensile: $92.62\%$
  - Compression: $51.41\%$
Buoyancy Calculations:

**Typical Girder Section: AASHTO Type III Girder (Spans 2, 3, 4)**

- Sheet T3

\[ A_1 = (1 + 4/5)(3/2) = 0.778 \text{ ft}^2 \]
\[ A_2 = 2 \left[ \frac{1}{2} \left( \frac{4}{5} \times \frac{8}{5} \right) \right] = 0.1406 \text{ ft}^2 \]
\[ A_3 = (\frac{7}{12})(4.5/2 + 9/12 + 7.5/12) = 1.507 \text{ ft}^2 \]
\[ A_4 = 2 \left[ \frac{1}{2} \left( \frac{3.5}{6} \times \frac{2}{6} \right) \right] = 0.3906 \text{ ft}^2 \]
\[ A_5 = (1 + 10/12)(3/12) = 1.0694 \text{ ft}^2 \]
\[ A_T = \sum A_i = 559.5 \text{ in}^2 \]
\[ A_T = 3.885 \text{ ft}^2 \text{ Type III Girder} \]

**Typical Girder Section: Keashi Type IV Girder (Span 1)**

- Sheet T3

\[ A_1 = (z)(4/12) = 0.667 \text{ ft}^2 \]
\[ A_2 = 2 \left[ \frac{1}{2} \left( 3/12 \times 8.3/6 \right) \right] = 0.1823 \text{ ft}^2 \]
\[ A_3 = (6.5/12)(3/12 + 9/12 + 5/12) = 2.0764 \text{ ft}^2 \]
\[ A_4 = 2 \left[ \frac{1}{2} \left( 9.7/12 \times 5/12 \right) \right] = 0.40625 \text{ ft}^2 \]
\[ A_5 = (2 + 9/12)(8/12) = 1.444 \text{ ft}^2 \]
\[ A_T = \sum A_i = 4.776 \text{ ft}^2 \]
\[ A_T = 4.776 \text{ ft}^2 \text{ Type IV Girder} \]
Deck Section: (Span 1)

* Note: The deck is sloped with the high point at the center. Slope is ignored for the purposes of these calculations.

Typical Section: Span 1

\[ \begin{align*}
A_f &= (23 + \frac{3}{12} + 23 + \frac{3}{12})(6\frac{5}{12}) = 26.724 \, \text{ft}^2 \\
A_f &= 26.73 \, \text{ft}^2 \\
\end{align*} \]

Deck Span 1

\[ \begin{align*}
L &= 90.61 \\
\end{align*} \]

* Railings:

\[ \begin{align*}
A_1 &= \left( \frac{1}{12} \right) \left( \frac{3}{12} \right) = 0.067 \, \text{ft}^2 \\
A_2 &= \left( \frac{1}{12} \right) \left( \frac{3}{12} \right) + \frac{1}{12} = 1.938 \, \text{ft}^2 \\
A_3 &= \frac{1}{12} \left( \frac{3}{12} \right) = 0.039 \, \text{ft}^2 \\
A_4 &= \left( \frac{1}{12} \right) \left( \frac{3}{12} \right) = 0.037 \, \text{ft}^2 \\
A_r &= \sum A_c = 3.291 \, \text{ft}^2 \\
A_r &= 3.292 \, \text{ft}^2 \\
\end{align*} \]
Deck Section: (spans 2, 3, 4)

* Note: The width of the bridge varies

* An average width will be used for spans 2, 3, 4

Slope of width = 0.005 ft 3/4

I) Span 2:

- Width = \((23 + \frac{9}{2}) + (0.05247)(90 + 30)\)
  Width = 24.3796 ft

- Thickness \((t) = 6.75\) in (sheet S2)

- Area = \((23 + \frac{9}{2} + 24.3796)(6.75\%2) = 27.0729 ft^2\)
  \[A_r = 27.07 ft^2\]

  Deck Span 2
  \[L = 60 ft\]

II) Span 3:

- Width = \((23 + \frac{9}{2}) + (0.05247)(90 + 60 + 30)\)
  Width = 24.6944 ft

- \(t = 6.75\) in (sheet S2)

- Area = \((23 + \frac{9}{2} + 24.6944)(6.75\%2) = 27.25 ft^2\)
  \[A_r = 27.25 ft^2\]

  Deck Span 3
  \[L = 60 ft\]

III) Span 4:

- Width = \((23 + \frac{9}{2}) + (0.05247)(90 + 60 + 60 + 30)\)
  Width = 25.0093 ft

- \(t = 6.75\) in (sheet S2)

- Area = \((23 + \frac{9}{2} + 25.0093)(6.75\%2) = 27.4271 ft^2\)
  \[A_r = 27.43 ft^2\]

  Deck Span 4
  \[L = 60 ft\]
**Concrete Diaphragms:**

**Beam D (Keel Type III Girder):**

**Span 1**

* Sheet 526

![Diagram of the concrete diaphragms and structural analysis](image)

### Area I:

\[ A_1 = (5)(\frac{9}{12}) = 2.083 \text{ ft}^2 \]

\[ A_2 = 2 \left( \frac{1}{2} \left( \frac{9.2}{12} \right) \left( \frac{9}{12} \right) \right) = 0.1823 \text{ ft}^2 \ (\text{-}) \]

\[ A_3 = \left( \frac{9.2}{12} \right) \left( \frac{4}{12} \right) = 4.759 \text{ ft}^2 \]

\[ A_4 = 2 \left( \frac{1}{2} \left( \frac{9}{12} \right) \left( \frac{9}{12} \right) \right) = 0.4065 \text{ ft}^2 \ (\text{-}) \]

\[ A_T = A_1 - A_2 + A_3 - A_4 \]

\[ = (2.083) - (-0.1823) + 4.759 - 0.4065 \]

\[ A_T = 26.25 \text{ ft}^2 \]

\[ A_T = 26.25 \text{ ft}^2 \] **Area I Concrete Diaphragm**

### Area II:

\[ A_1 = \left( \frac{4}{12} \right) \left( \frac{9}{12} \right) = 0.566 \text{ ft}^2 \ (\text{-}) \]

\[ A_2 = 2 \left( \frac{1}{2} \left( \frac{9}{12} \right) \left( \frac{9}{12} \right) \right) = 0.5625 \text{ ft}^2 \ (\text{-}) \]

\[ A_T = A_{\text{Area II}} - A_1 - A_2 \]

\[ = (26.25) - 2.5 - 0.5625 \]

\[ A_T = 23.1892 \text{ ft}^2 \]

\[ A_T = 23.19 \text{ ft}^2 \] **Area II Concrete Diaphragm**

### Area III:

\[ A_1 = \left( \frac{9}{12} \right) \left( \frac{9}{12} \right) = 0.1389 \text{ ft}^2 \]

\[ A_2 = \left( \frac{9.75}{12} \right) \left( \frac{3}{12} \right) = 0.151 \text{ ft}^2 \ (\text{-}) \]

\[ A_3 = \left( \frac{11.75}{12} \right) \left( \frac{3}{12} \right) = 0.39 \text{ ft}^2 \]

\[ A_4 = \left( \frac{13.75}{12} \right) \left( \frac{3}{12} \right) = 0.46 \text{ ft}^2 \]

\[ A_T = A_1 - A_2 + A_3 - A_4 + A_5 \]

\[ = (0.1389) - (-0.151) + 0.39 - (0.46) \]

\[ A_T = 4.451 \text{ ft}^2 \]

**Air Pocket**

\[ A_T = 4.46 \text{ ft}^2 \]
I) Area I:
\[ A_1 = \left( \frac{w}{2} \right) \left( \frac{h}{2} \right) = 3.78 \text{ ft}^2 \]
\[ A_2 = 2 \left\{ \left( \frac{w}{4} \right) \left( \frac{h}{2} \right) \right\} = 0.1906 \text{ ft}^2 \]
\[ A_3 = \left( \frac{w}{2} \right) \left( \frac{h}{2} \right) = 16.576 \text{ ft}^2 \]
\[ A_4 = 2 \left\{ \left( \frac{w}{4} \right) \left( \frac{h}{2} \right) \right\} = 0.3906 \text{ ft}^2 \]
\[ A_T = A_1 - A_2 + A_3 - A_4 = 19.8229 \text{ ft}^2 \]
\[ A_T = 19.82 \text{ ft}^2 \] Area I Concrete Diaphragm

II) Area II:
\[ A_1 = \left( \frac{w}{2} \right) \left( \frac{h}{2} \right) = 2.75 \text{ ft}^2 \]
\[ A_2 = 2 \left\{ \left( \frac{w}{4} \right) \left( \frac{h}{2} \right) \right\} = 0.6625 \text{ ft}^2 \]
\[ A_T = A_1 - A_2 = 19.8229 - 2.75 - 0.5625 \]
\[ A_T = 16.51 \text{ ft}^2 \] Area II Concrete Diaphragm

III) Area III:
\[ A_1 = \left( \frac{w}{2} \right) \left( \frac{h}{2} \right) = 0.4375 \text{ ft}^2 \]
\[ A_2 = \left( \frac{w}{2} \right) \left( \frac{h}{2} \right) = 0.9703 \text{ ft}^2 \]
\[ A_3 = \left( \frac{w}{4} \right) \left( \frac{h}{2} \right) = 2.90625 \text{ ft}^2 \]
\[ A_4 = \left( \frac{w}{2} \right) \left( \frac{h}{2} \right) = 0.1953 \text{ ft}^2 \]
\[ A_5 = \left( \frac{w}{2} \right) \left( \frac{h}{2} \right) = 0.2917 \text{ ft}^2 \]
\[ A_T = A_1 - A_2 + A_3 - A_4 + A_5 = 3.3749 \text{ ft}^2 \]
\[ A_T = 3.37 \text{ ft}^2 \] Area III Air Pocket
Concrete Diaphragms: Cont.

**Beam B:**

\[ A_T = 19.8229 + \left( \frac{35}{12} \right) \left( \frac{y}{12} \right) = 23.8368 \text{ ft}^2 \]

\[ A_T = 23.84 \text{ ft}^2 \]

**Beam C:**

\[ A_T = 26.2617 + \left( \frac{30}{12} \right) \left( \frac{y}{12} \right) = 29.4739 \text{ ft}^2 \]

\[ A_T = 29.47 \text{ ft}^2 \]

Diaphragm goes to the bottom of the girder.
**Concrete Diaphragms: cont.**

- **Beam D: cont.**

  Total Area = 4 (Area II) + 2 (Area III)
  = 4 (23.14) + 2 (26.28)
  Total Area = 145.26 ft²

  \[ A_T = 145.26 \text{ ft}^2 \]
  Beam D Diaphragm
  Thickness = 1 ft

- **Beam A: cont.**

  Total Area = 4 (Area III) + 2 (Area I)
  = 4 (16.51) + 2 (19.82)
  Total Area = 105.68 ft²

  \[ A_T = 105.68 \text{ ft}^2 \]
  Beam A Diaphragm
  Thickness = 1 ft

- **Beam B: cont.**

  Total Area = 6 (Area)
  = 6 (22.84)
  Total Area = 137.04 ft²

  \[ A_T = 137.04 \text{ ft}^2 \]
  Beam B Diaphragm
  Thickness = 1.167 ft

- **Beam G: cont.**

  Total Area = 6 (Area)
  = 6 (29.47)
  Total Area = 176.82 ft²

  \[ A_T = 176.82 \text{ ft}^2 \]
  Beam G Diaphragm
  Thickness = 2 ft
Air Pocket Calculations:

- AASHTO Type III Girders: (No Creep Block Section)
  \[ A_T = 19.8229 + (\frac{3}{12})(\frac{3}{12}) = 22.8368 \text{ ft}^2 \]
  upper area

  \[ A_T = 22.84 \text{ ft}^2 \] AASHTO Type III Girder (No Creep Block)
  Air Pocket

- Keiki Type IV Girder: (No Creep Block Section)
  \[ A_T = 26.5517 + (\frac{5}{12})(\frac{5}{12}) = 29.4739 \text{ ft}^2 \]
  upper area

  \[ A_T = 29.47 \text{ ft}^2 \] Keiki Type IV Girder (No Creep Block)
  Air Pocket

- AASHTO Type III Girders: (Creep Block Section)
  \[ A_T = 19.82 \text{ ft}^2 \] AASHTO Type III Girder Air Pocket

- Keiki Type IV Girder: (Creep block Section)
  \[ A_T = 26.25 \text{ ft}^2 \] Keiki Type IV Girder Air Pocket
Ukon Pond Bridge

Structural Analysis

Page 9

- Reduction of air pocket:
  - AASHTO Type III Grider:
    - No Creep:  \( h = (6.5\%)+ (5/12+3/12+3/12) = 4.375 \text{ ft} \)
      - P_2 = 14.7 + h(6\% /144)
      \[
      P_2 = 14.7 + (4.375)(6\% /144)
      P_2 = 16.644 \text{ psi}
      \]
      - \( A_2 = \frac{P_1 A_1}{P_2} = \frac{(14.7)(22.44)}{16.644} \)
        \[
        A_2 = 20.1718 \text{ ft}^2
        \]
        \[
        A_T = 20.17 \text{ ft}^2 \text{ Compressed Air}
        \]

  - Creep Block:
    - \( h = (6.5\%)+ (5/12+3/12) = 3.79167 \text{ ft} \)
    - P_2 = 14.7 + h(6\% /144)
      \[
      P_2 = 14.7 + (3.79167)(6\% /144)
      P_2 = 16.3852 \text{ psi}
      \]
      - \( A_2 = \frac{P_1 A_1}{P_2} = \frac{(14.7)(19.42)}{16.3852} \)
        \[
        A_2 = 17.7816 \text{ ft}^2
        \]
        \[
        A_T = 17.78 \text{ ft}^2 \text{ Compressed Air}
        \]

  - Kechi Type III Grider:
    - No Creep:  \( h = (6.5\%)+ (5/12+5/12) = 5.4583 \text{ ft} \)
      - P_2 = 14.7 + h(6\% /144)
        \[
        P_2 = 14.7 + (5.4583)(6\% /144)
        P_2 = 17.1259 \text{ psi}
        \]
      - \( A_2 = \frac{P_1 A_1}{P_2} = \frac{(14.7)(29.47)}{17.1259} \)
        \[
        A_2 = 25.2956 \text{ ft}^2
        \]
        \[
        A_T = 25.30 \text{ ft}^2 \text{ Compressed Air}
        \]

  - Creep Block:
    - \( h = (6.5\%)+ (5/12+4/12) = 4.79167 \text{ ft} \)
      - P_2 = 14.7 + h(6\% /144)
        \[
        P_2 = 14.7 + (4.79167)(6\% /144)
        P_2 = 16.8216 \text{ psi}
        \]
      - \( A_2 = \frac{P_1 A_1}{P_2} = \frac{(14.7)(26.25)}{16.8216} \)
        \[
        A_2 = 22.9283 \text{ ft}^2
        \]
        \[
        A_T = 22.93 \text{ ft}^2 \text{ Compressed Air}
        \]

  - Side Area: (From \( P_0, 5 \))
    - \( h = 4.375 \text{ ft} \)
      - P_2 = 16.644 \text{ psi}
      - \( A_2 = \frac{P_1 A_1}{P_2} = \frac{(14.7)(3.37)}{16.644} \)
        \[
        A_2 = 2.976 \text{ ft}^2
        \]
        \[
        A_T = 2.98 \text{ ft}^2 \text{ Compressed Air}
        \]
Bouyancy Calculations:

- **Span 1:**
  - Concrete:
    \[ V_{s1} = \left[ 7 \times \text{(Keel Girders)} + \text{(Deck)} + 2 \times \text{(Railings)} \right] (90) + (\text{Beam D Diaphragm})(1) \]
    \[ = \left[ 7 \times (4.776) + 25.73 + 2 \times (3.292) \right] (90) + (147.26)(1) \]
    \[ V_{s1} = 6064.4 \text{ ft}^3 \] Concrete
  - Air Pocket:
    \[ V_{a1} = \left[ 6 \times \text{(No Creep Block Area)} \right] (90) \]
    \[ = \left[ 6 \times (25.30) \right] (90) \]
    \[ V_{a1} = 13662.0 \text{ ft}^3 \] Air

- **Span 2:**
  - Concrete:
    \[ V_{s2} = \left[ 7 \times \text{(AASHTO Girders)} + \text{Deck} + 2 \times \text{(Railings)} \right] (60) + (\text{Beam G Diaphragm})(1) \]
    \[ = \left[ 7 \times (3.885) + 27.07 + 2 \times (3.292) \right] (60) + (176.82)(2) \]
    \[ V_{s2} = 4004.6 \text{ ft}^3 \] Concrete
  - Air Pocket:
    \[ V_{a2} = \left[ 6 \times \text{(No Creep Block Area)} \right] (60) \]
    \[ = \left[ 6 \times (20.17) \right] (60) \]
    \[ V_{a2} = 7261.2 \text{ ft}^3 \] Air
Bouyancy Calculations: Cont.

o Span 3:
  ♦ Concrete:
    \[ V_{S3} = (60) \left[ \frac{7}{8} \text{(AASHTO)} + \text{Deck} + 2(\text{Railings}) \right] + \text{Beam B Diaphragm} (1.167) \]
    \[ = (60) \left[ \frac{7}{8} (3.885) + (27.25) + 2(3.292) \right] + (137.04) (1.167) \]
    \[ V_{S3} = 3821.62 \text{ ft}^3 \]
    \[ V_{S3} = 3821.6 \text{ ft}^3 \]

♦ Air Pocket:
    \[ V_{A3} = 7.261.2 \text{ ft}^3 \]

o Span 4:
  ♦ Concrete:
    \[ V_{S4} = \left[ \frac{7}{8} \text{(AASHTO)} + \text{Deck} + 2(\text{Railings}) \right] (60) + \text{(Beam B)} (1.167) \]
    \[ + \text{(Beam A)} (1) \]
    \[ = \left[ \frac{7}{8} (3.885) + 27.43 + 2(3.292) \right] (60) + (137.1) (1.167) + 105.68 (1) \]
    \[ V_{S4} = 3938.1 \text{ ft}^3 \]

♦ Air Pocket:
    \[ V_{A4} = \left\{ \begin{array}{ll} 6 \text{ (No Creep)} & \text{ (60)} \\ 6 (20.17) & \text{ (60)} \end{array} \right\} \]
    \[ V_{A4} = 7261.2 \text{ ft}^3 \]

▲ Self Weight: \( Y_{ac} = 150 \text{ lb/ft}^3 \)

o Total Volume = 6064.4 + 4004.6 + 3821.6 + 3938.1
  \[ TV = 17828.7 \text{ ft}^3 \]

o Self Weight = \( TV \times Y_{ac} \)
  \[ = (17828.7)(150) \]
  \[ SW = 2.67431 \times 10^6 \text{ lbs} \]

▲ Bouyant Force: \( Y_{saw} = 64 \text{ lb/ft}^3 \)

o Total Submerged Volume = (Volume Concrete - Railings) + Air Pockets
  \[ = (17828.7 - \left(2\right)(3.98 \times 2.30) + 1366.2 + 3(7261.2) \]
  \[ TSW = 51665.1 \text{ ft}^3 \]

o Bouyant Force = \( TSW \times Y_{sw} \)
  \[ = (51665.1)(64) \]
  \[ BF = 3.30657 \times 10^6 \text{ lbs} \]
Summary of Buoyancy Calculations:

- Self Weight = 26,74.3 kips
- Buoyant Force = 3306.6 kips
- Residual Weight = -632.3 kips
- % Retained Weight = -23.6%

∴ Bridge is Buoyant
Span Capacity (Moment)

Positive Bending:

- $f_c' = 3,750$ psi
- $f_y = 60,000$ psi
- $b = 60$ in
- $E = 29,000$ psi

\[ d = 6.5 - (1.25) - 0.625\% = 4.9375 \text{ in} \]

\[ A_s = 8 \text{ No. 5} \]
\[ = 8 (0.31) \]
\[ A_s = 2.48 \text{ in}^2 \]

\[ a = A_s f_y = (2.48)(60) \]
\[ = 0.85 \text{ ft} b (0.85 \times 3.75 \times 60) \]
\[ = 0.8778 \text{ in} \]

Check steel has yielded:
\[ C = \frac{f_y}{f_y} = 0.9786 \]
\[ C = 0.915 \]

\[ \varepsilon_s = (\frac{d_c}{d}) \varepsilon_{cy} = 0.0182 \geq \varepsilon_y \text{ ok} \]

Negative Bending:

\[ d = 6.5 - (1.25) - 0.625\% = 4.6875 \text{ in} \]

\[ A_s = 6 \text{ No. 5} + 2 \text{ No. 10} \]
\[ = 6 (0.31) + 2 (1.29) \]
\[ = 4.4 \text{ in}^2 \]

\[ a = A_s f_y = (4.4)(60) \]
\[ = 0.85 \text{ ft} b (0.85 \times 3.75 \times 60) \]
\[ = 1.32 \text{ in} \]

\[ \varepsilon_s \geq \varepsilon_y \text{ ok} \]
Span 2, 3 & 4 Capacity: (Moment)

Same reinforcement as span 1

\[ d = 6.75 \text{ in} \]

- **Positive Bending**:
  \[ f' = 3,750 \text{ psi} \]
  \[ f_y = 60,000 \text{ psi} \]
  \[ b = 60 \text{ in} \]

\( d = 6.75 - 1.25 - 0.625/2 = 5.1875 \text{ in} \)

\( A_0 = 2.48 \text{ in}^2 \)
\( a = 0.778 \text{ in} \)

\( \phi = 0.90 \)

\[ \phi M_n = \phi A_0 f_y (d - a) \]
\[ = 0.90(2.48)(60)(5.1875 - 0.778) \]
\[ \phi M_n = 542.612 \text{ k-ft} \]

\[ \phi M_n = 53.55 \text{ k-ft} \]  \( \text{Positive Bending} \)

- **Negative Bending**:

\( d = 6.75 - 1.5 - 0.625/2 = 4.9375 \text{ in} \)

\( A_0 = 4.4 \text{ in}^2 \)
\( a = 1.38 \text{ in} \)

\( \phi = 0.90 \)

\[ \phi M_n = \phi A_0 f_y (d - a) \]
\[ = 0.90(4.4)(60)(4.9375 - 1.38) \]
\[ \phi M_n = 1009.16 \text{ k-in} \]

\[ \phi M_n = 84.10 \text{ k-ft} \]  \( \text{Negative Bending} \)
**Shear Capacity: (Span 1)**

\[ V_c = 2A \sqrt{f_e} \text{ bar}^d \]
\[ = 2(1.0)(3750)^{1/2}(60)(4.94) \]
\[ V_c = 36301.4 \text{ lbs} \]

\[ \phi V_c = (0.75)(36301.4) \]
\[ = 27226.1 \text{ lbs} \]

\[ \phi V_c = 27.26 \text{ kips} \]

**Shear Capacity: (Spans 2, 3, 4)**

\[ V_c = xA \sqrt{f_e} \text{ bar}^d \]
\[ = 2(1.0)(3750)^{1/2}(60)(5.175) \]
\[ V_c = 38120.2 \text{ lbs} \]

\[ \phi V_c = (0.75)(38120.2) \]
\[ = 28590.1 \text{ lbs} \]

\[ \phi V_c = 28.59 \text{ kips} \]
Vertical Hinge Restrainer:
* (Sheet 18)

7/8" continuous looped galv. cable

(4) at each abutment
Secures bridge to creep block
⇒ 8 total

4 x 19 wire strand
or wire rope

Breaking strength = 23 tons

Total strength = (8)(23) = 184 tons = 368 kips

[Vertical Resistance = 368 kips]
(Vertical Hinge Restrainers)

Buoyancy:

• Self Weight + Vertical Hinge Restrainers = 3674.3 + 368 = 3042.3 kips

• Buoyant Force = 3306.6 kips

• (SW + VHR) - BF = 3042.3 - 3306.6
  = -264.3 kips (ρ = 6)

⇒ Bridge Will Be Buoyant

Buoyant Force > Vertical Resistance
Koshi Type IV Girder: (Span 1)

Properties:
- \( P_e = 793.4 \text{ kips} \)
- \( f' = 4500 \text{ psi} \)
- \( c = 20.46 \text{ in} \)

- **Centroid:**
  - \( A_1 = 0.663 \text{ ft}^2 \)
  - \( A_2 = 1.823 \text{ ft}^2 \)
  - \( A_3 = 2.0749 \text{ ft}^3 \)
  - \( y_1 = 5.61 \text{ in} \)
  - \( y_2 = 5.8 \text{ in} \)
  - \( y_3 = 3.1 \text{ in} \)
  - \( A_0 = 4.0625 \text{ ft}^2 \)
  - \( A_5 = 1.444 \text{ ft}^2 \)
  - \( y_4 = 10 \text{ in} \)
  - \( y_5 = 4 \text{ in} \)
  - \( \bar{y} = \frac{2A_0 y_4}{\Sigma A_i} \)
  - \( \bar{y} = 26.3816 \text{ in} \)
  - \( \gamma_6 = 3.262 \text{ in} \)
  - \( \gamma_7 = 26.38 \text{ in} \)

- **Moment of inertia:**
  - \( I_1 = \frac{1}{12} bh^3 = \frac{1}{12}(6.5 \times 46)^3 \)
  - \( I_1 = 128 \text{ in}^4 \)
  - \( I_1 + A_d y_4^2 = 128 + (96.046 \times 50.62)^2 \)
  - \( = 90181.1 \text{ in}^4 \)
  - \( I_2 = \frac{1}{6} bh^3 = \frac{1}{6}(8.75 \times 3)^3 \)
  - \( I_2 = 6.5625 \text{ in}^4 \)
  - \( I_2 + A_d y_4^2 = (6.5625 - 41.846) \times (27.62)^2 \)
  - \( = 10,019.6 \text{ in}^4 \)
  - \( I_3 = \frac{1}{12} bh^3 = \frac{1}{12}(6.5 \times 46)^3 \)
  - \( I_3 = 52723.7 \text{ in}^4 \)
  - \( I_3 + A_d y_4^2 = (52723.7 - 62) \times (50.62)^2 \)
  - \( = 62167.2 \text{ in}^4 \)
  - \( I_4 = \frac{1}{6} bh^3 = \frac{1}{6}(9.75 \times 6)^3 \)
  - \( I_4 = 58.5 \text{ in}^4 \)
  - \( I_4 + A_d y_4^2 = 58.5 + (29.99 \times 15.36)^2 \)
  - \( = 6747.42 \text{ in}^4 \)
  - \( I_5 = \frac{1}{12} bh^3 = \frac{1}{12}(26 \times 8)^3 \)
  - \( I_5 = 1109.33 \text{ in}^4 \)
  - \( I_5 + A_d y_4^2 = 1109.33 + (207.07 \times 21.38)^2 \)
  - \( = 96,841.1 \text{ in}^4 \)

\[ I_c = 90181.1 + (2)(10019.6) + 62167.2 + (2)(6747.42) + 96841.1 \]
\[ I_c = 2822526 \text{ in}^4 \]
Keehi Type IV Girder: (Span 1)

Geometric Properties:

- $A_e = 687.744 \text{ in}^2$
- $I_z = 282.626 \text{ in}^4$
- $C_t = 32.62 \text{ in}$
- $C_h = 26.38 \text{ in}$
- $r^2 = 410.802 \text{ in}^2$
- $C_e = 20.46 \text{ in}$
- $L = 1080 \text{ in}$

\[ W_0 = \text{Self Weight + Topping + Future wearing surface} \]
\[ \approx (4.775 \times 150) + (3 \times 1.5/8 \times 150) + 24(4) \]
\[ W_0 = 1453.15 \frac{\text{lb}}{\text{ft}} \]
\[ W_D = 121.076 \frac{\text{lb}}{\text{ft}} \]

\[ M_{D0} = W_D L^2/8 = 1.76658 \times 10^7 \text{ lb-in} \]

I. Tensile Limit (Unsubmerged)

\[ W_{ac} = \frac{8}{L^2} (M_0 + S_k f_k - \left( \frac{5k}{A_e} \right) (C_e - 1)) \]

\[ \approx \frac{8}{L^2} \left( M_0 + \left( 866.13 \times 804.984 \right) - \left( 866.13 \times 743.4 \times 10^3 \right) \left( 20.46 \left( 32.62 \right) - 1 \right) \right) \]

\[ = 126.104 \frac{\text{lb}}{\text{ft}} \]

Fur = 136.197 kips (Tension)

II. Compression Limit (Unsubmerged)

\[ W_{ac} = \frac{8}{L^2} \left( M_0 - S_k f_c \right) - \frac{5k}{A_e} \left( 1 + 2C_k \right) \]

\[ \approx \frac{8}{L^2} \left( M_0 + 1.35043 \times 10^2 \right) \]

\[ W_{ac} = 215.719 \frac{\text{lb}}{\text{ft}} \]

Fur = 280.82 kips (Compression)

II. When Submerged:

* Assume 50% air pocket

\[ \approx \text{each girder will take half of the air pocket from each side} \]

\[ W_0 = 1463.15 \frac{\text{lb}}{\text{ft}} - 8.56767 \times 4 \times \left( \frac{13.120(64)}{\text{air pocket}} \right) \]

\[ W_D = 64.8191 \frac{\text{lb}}{\text{ft}} \]

\[ W_D = 6.4016 \frac{\text{lb}}{\text{ft}} \]

\[ M_0 = 787.552 \text{ lb-in} \]

IV. Tensile Limit (Submerged)

\[ W_{ac} = \left( \frac{8}{L^2} \right) \left( 787.552 \times 1.04 + 730.851 \right) \]

\[ W_{ac} = 10.419 \frac{\text{lb}}{\text{ft}} \]

Fur = 11.247 kips (Tension)

IV. Compression Limit (Submerged)

\[ W_{ac} = \left( \frac{8}{L^2} \right) \left( 787.552 \times 1.3504 - 1.5 \times 10^3 \right) \]

\[ W_{ac} = 18.0237 \frac{\text{lb}}{\text{ft}} \]

Fur = 105.866 kips (Compression)
**AASHTO Type III Girder** (Span 3, 3, 4)

Properties:
- $f_c = 4000$ psi
- $P_e = 636.2$ kips
- $e_c = 16.20$ in

**Centroid:**
- $A_1 = 0.978 \text{ ft}^2$
  $y_1 = 4.56 \text{ in}$
- $A_2 = 1.406 \text{ ft}^2$
  $y_2 = 36.5 \text{ in}$
- $A_3 = 1.507 \text{ ft}^2$
  $y_3 = 22.5 \text{ in}$

**Moments of Inertia:**
- $I_1 = \frac{1}{6} bh^3 = \frac{1}{6} (16)(4)^3$
  $I_1 = 467.2 \text{ in}^4$
- $I_2 = \frac{1}{6} bh^3 = \frac{1}{6} (36)(4.5)^3 = 173.8 \text{ in}^4$
- $I_3 = \frac{1}{6} bh^3 = \frac{1}{6} (36)(7.5)^3 = 87.81 \text{ in}^4$
- $I_4 = \frac{1}{6} bh^3 = \frac{1}{6} (36)(7.5)^3 = 87.81 \text{ in}^4$
- $I_5 = \frac{1}{6} bh^3 = \frac{1}{6} (24)(9)^3 = 628.833 \text{ in}^4$

$$\begin{align*}
I_c &= 50927.8 + 2(2676.33) + 18452.4 + 2(390.917) + 43962.7 \\
I_c &= 114777 \text{ in}^4
\end{align*}$$
**AASHTO Type III Girder: cont.**

- **Geometric Properties:**
  \[
  A_c = 559.44 \text{in}^2 \\
  I_c = 119.477 \text{in}^4 \\
  C_c = 24.326 \text{in} \\
  C_b = 20.275 \text{in} \\
  t^2 = 213.566 \text{in}^2 \\
  C_e = 16.20 \text{in} \\
  L = 720 \text{in}
  \]

- **Self Weight + Topping + Weaving Surface**
  \[
  W_0 = (3.885)(160) + (2)(150) + 20(7) \\
  W_0 = 1319.5 \text{lb} \cdot \text{ft} \\
  W_0 = 100.958 \text{lb} \cdot \text{in}
  \]

- **Hoop:**
  \[
  H_0 = \frac{W_0}{L} = 7.1253 \times 10^6 \text{lb} \cdot \text{in}
  \]

- **Tensile Limit: (Unsubmerged)**
  \[
  \omega_{\text{ub}} = \frac{8}{L^2} \left[ M_0 + S_0 \{ \frac{S_0 P_e}{A_c} \left( \frac{Ec}{L^2} - 1 \right) \} \right] \\
  \omega_{\text{ub}} = \frac{8}{L^2} \left[ M_0 + (-11434 x 10^6) \right] \\
  \omega_{\text{ub}} = 62.4613 \text{ kips (tensile)}
  \]

- **Compression Limit: (Unsubmerged)**
  \[
  \omega_{\text{wc}} = \frac{8}{L^2} \left[ M_0 - (S_0 - \{ \frac{S_0 P_e}{A_c} \left( 1 + \frac{L}{C_c} \right) \} \right] \\
  \omega_{\text{wc}} = \frac{8}{L^2} \left[ M_0 + 3.0233 x 10^6 \right] \\
  \omega_{\text{wc}} = 112.818 \text{ kips (compression)}
  \]

- **When Submerged:**
  \[
  W_0 = 1319.5 - (3.6763)(64) - 9.91(64) \times 50\% \text{ air pocket} \\
  W_0 = 193.453 \text{ lb} \cdot \text{ft} \\
  W_0 = 16.1628 \text{ lb} \cdot \text{in} \\
  \Rightarrow M_0 = 1.04735 x 10^6 \text{ lb} \cdot \text{in}
  \]

- **Tensile Limit: (Submerged)**
  \[
  \omega_{\text{ub}} = \frac{8}{L^2} \left[ 1.04735 x 10^6 - 1.14374 x 10^6 \right] \\
  \omega_{\text{ub}} = 1.46355 \text{ lb} \cdot \text{in} \\
  \omega_{\text{ub}} = 1.071 \text{ kips (failure)}
  \]

- **Compression Limit: (Submerged)**
  \[
  \omega_{\text{wc}} = \frac{8}{L^2} \left[ 1.04735 x 10^6 + 3.0233 x 10^6 \right] \\
  \omega_{\text{wc}} = 62.8886 \text{ lb} \cdot \text{in} \\
  \omega_{\text{wc}} = 46.2798 \text{ kips (compression)}
  \]
Summary of Results:

**Ketch Type IV:**
- **Unsubmerged:**
  - Tension: Ftu = 130.147 kips
  - Compression: Fcu = 230.82 kips
- **Submerged:**
  - Tension: Ftu = 11,247 kips
  - Compression: Fcu = 105,866 kips
- **Loss once submerged:**
  - Tension: 91.74%
  - Compression: 54.13%

**AASHTO Type III** (50% air pocket)
- **Unsubmerged:**
  - Tension: Ftu = 66,461 kips
  - Compression: Fcu = 112,218 kips
- **Submerged:**
  - Tension: Ftu = 1.071 kips
  - Compression: Fcu = 45,249 kips
- **Loss:**
  - Tension: 101.61%
  - Compression: 54.13%
**Buoyancy Calculations:**

- **Air Pocket Calculations:**
  - **Plan View (Sheet 2):**

```
\[ A_1 = (19 + \frac{3}{4})(13 + \frac{3}{4}) = 265.281 \text{ ft}^2 \]
\[ A_{beam} = (19 + \frac{3}{4})(\frac{3}{4}) = 9.75 \text{ ft}^2 \]
\[ \text{Air Void Area} = A_1 - 6(A_{beam}) = 265.281 - 6(9.75) \]
\[ A_{AV} = 206.781 \text{ ft}^2 \]
```

\* Note: There are 8 total 19'-6" x 13'-7.25" sections under the Makaha 3A Bridge.

- **Air Pocket Volume:**
  - Total Area = \( 8(A_{AV}) \)
  - Total Area = \( 8(206.781) \)
  - Total Area = 1654.25 \( \text{ft}^2 \)
  - Depth of pocket = 18"
  - Air Pocket Volume = \( \frac{18}{12} \times 1654.25 \)
  - Air Pocket Volume = 2481.38 \( \text{ft}^3 \)

\[ \text{Air Pocket} = 2481.38 \text{ ft}^3 \]

- **Reduction of Air Pocket:**
  - *Note: Slope of bridge is ignored*

\[ h = \left( \frac{5}{12} \right) + \left( \frac{18}{12} \right) + \left( \frac{3}{2} \right) = 2.083 \text{ ft} \]

\[ P_2 = 14.7 + h \left( \frac{64}{144} \right) \]
\[ P_2 = 14.7 + (2.083)(0.444) \]
\[ P_2 = 15.4239 \text{ psi} \]

\[ V_2 = \frac{P_2 V_1}{P_2} = \left( \frac{14.7}{15.4239} \right)(2481.38) \]
\[ V_2 = 2334.34 \text{ ft}^3 \]

\[ V_f = 2334.34 \text{ ft}^3 \text{ Compressed Air Pocket} \]
### Old Makaha No. 3A Bridge

**Self Weight Calculations**

\[
\gamma = 31.2 \text{ lbs/cubic ft} \quad (G = 0.5 \text{ of water - Value for Hawaii Douglas Fir [12% moisture]})
\]

<table>
<thead>
<tr>
<th>Timber:</th>
<th>No. Req.</th>
<th>Width (in)</th>
<th>Height (in)</th>
<th>Length (ft)</th>
<th>Volume (cubic ft)</th>
<th>Total Volume Per Item (cubic ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sill</td>
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<td>12</td>
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<td>Post</td>
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<td>12</td>
<td>18</td>
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<td>12</td>
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<td>Solid Bridging</td>
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<td>4</td>
<td>18</td>
<td>16</td>
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<td>40.00</td>
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<tr>
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<td>26</td>
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<td>12</td>
<td>20</td>
<td>16.67</td>
<td>66.67</td>
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<tr>
<td>Railing</td>
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<td>4</td>
<td>8</td>
<td>20</td>
<td>4.44</td>
<td>35.56</td>
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<tr>
<td>Railing</td>
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<td>20</td>
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<td>8</td>
<td>14</td>
<td>8</td>
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<tr>
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<td>14</td>
<td>12</td>
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<tr>
<td>Guard Rail</td>
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<td>8</td>
<td>18</td>
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<td>3.00</td>
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<td>20.00</td>
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<td>Guard Rail</td>
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<td>15.00</td>
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<td>Guard Rail</td>
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<td>18</td>
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<td>4.00</td>
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<td>4</td>
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<td>0.89</td>
<td>12.44</td>
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<td>8</td>
<td>16</td>
<td>1.78</td>
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<td>18</td>
<td>3</td>
<td>2.25</td>
<td>13.50</td>
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<td>Splice Block</td>
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Railing Post Blocks
Wall Plate

<table>
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<tr>
<th>Member</th>
<th>No. Req.</th>
<th>Width (ft)</th>
<th>Thickness (in.)</th>
<th>Length (ft)</th>
<th>Cross Sectional Area (sq ft)</th>
<th>Volume (cubic ft)</th>
<th>Total Volume (cubic ft)</th>
<th>Total Weight (lbs)</th>
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<tbody>
<tr>
<td>W10x22</td>
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<td>20</td>
<td>8</td>
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<td>1</td>
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<td>215.97</td>
<td>32827.51</td>
<td>32827.51</td>
<td></td>
</tr>
</tbody>
</table>

\[ \gamma = 152 \text{ lbs/cubic ft} \quad \text{(specific weight of asphalt)} \]

**Asphalt:**

<table>
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<tr>
<th>Member</th>
<th>No. Req</th>
<th>Width (ft)</th>
<th>Thickness (in.)</th>
<th>Length (ft)</th>
<th>Volume (cubic ft)</th>
<th>Total Weight (lbs)</th>
<th>Total Volume (cubic ft)</th>
<th>Total Weight (lbs)</th>
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<td>1</td>
<td>78.83</td>
<td>215.97</td>
<td>32827.51</td>
<td>32827.51</td>
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**Bridge Total:**

<table>
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<th>Member</th>
<th>Volume (ft^3)</th>
<th>Weight (lbs)</th>
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<td>Timber</td>
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<td>Steel Repair Beams</td>
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<tr>
<td>Total</td>
<td>6688.23</td>
<td>242999.54</td>
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</table>
\[ A_1 = \left( \frac{9}{12} \right) \left( 2 + 11.75 \right) = 2.23438 \text{ ft}^2 \]

\[ A_2 = \left( 1 + \frac{9}{12} \right) \left( \frac{15}{12} \right) = 1.667 \text{ ft}^2 \]

\[ A_T = A_1 + A_2 = 3.73438 \text{ ft}^2 \]

- **Total Volume:**
  - Length = 32.875 ft
  - Volume = \( 2(AR)(L) \)
  - Volume = \( 2 \times 3.73438 \times 32.875 \)
  - Volume = 245.535 \( \text{ ft}^3 \)

- **Concrete Weight =** \( \frac{1}{4} \times Y_{RC} \)
  - CW = \( (245.535 \times 150) \)
  - CW = 36830.3 \text{ lbs} \]

**Total Self Weight:**

- From Pg. 2:
  - Self Weight = Timber + Steel + Asphalt + Concrete
  - Self Weight = 242999.54 + 36830.3
  - Self Weight = 246682.8 lbs
**Total Self Weight**
- From pg. 34

\[
\text{Self Weight} = 279.83 \text{ kips}
\]

**Buoyant Force:** \(V_{\text{water}} = 41 \text{ ft}^3\)

*Note: Assume bridge is submerged to the top of the deck*

- Submerged Volume = Bridge Volume + Air Pocket + Asphalt + Concrete Abutment

\[
= (\text{Bridge} - \text{Railing} - \text{Sidewalk}) + \text{Air Pocket} + \text{Asphalt} + \text{Concrete Abutment}
\]

\[
= (6454.23 - 247.11) + 2334.34 + 215.97 + 245.535
\]

\[
SV = 9002.97 \text{ ft}^3
\]

- Buoyant Force = \(SV \times y_{\text{sw}}\)

\[
BF = (9002.97)(64)
\]

\[
BF = 576190 \text{ lbs}
\]

**Residual Weight**:

\[
\text{Residual Weight} = SV - BF
\]

\[
= 279830 - 576190
\]

\[
RW = -296360 \text{ lbs}
\]

**Summary of Results**:

- Self Weight = 279.8 kips
- Buoyant Force = 576.2 kips
- Residual Weight = -296.4 kips
- % Retained Weight = -106.0%

\[
\therefore \text{Bridge is Buoyant}
\]
Deck Capacity:

* Bridge deck is made of 2\" x 6\" x 36'-0\" (886)
  Douglas Fir wood, with 12\% moisture content

⇒ From Wood Design Engineering Handbook

Compression (⊥) to grain = 760.0 \text{ lb/in}^2
Shear/Tension (⊥) to grain = 360.0 \text{ lb/in}^2

0 Moment Capacity:

\[ f_b = \frac{M}{z} \]
\[ z = \frac{b h^2}{2} = \frac{(26\times12)(6)}{2} = 1872 \text{ in}^2 \]
\[ M = f_b z = (760 \times 1872) \]
\[ M = 1.422 \times 10^6 \text{ lb-in} \]
\[ M = 118.56 \text{ k-ft} \]

0 Shear Capacity:

\[ f_s = k \left( \frac{V}{A} \right) = \frac{kV}{A} \]
\[ A = (26\times12) = 1872 \text{ in}^2 \]
\[ V = \frac{kV}{A} = \frac{(360)(1872)(3\%)}{1872} \]
\[ V = 436800 \text{ lb} = 436.8 \text{ kips} \]

\[ \phi V_n = 0.75 (436.8) = 327.6 \text{ kips} \]
**Vertical Resistance:** (bolts in bridge beams)

Specific Gravity, Douglas Fir = 0.5
\[ \text{Fr} = 0.5 \times \text{Gr} = 0.5 \text{ lb/ft}^3 \]
\[ G = 0.5 \]

**Ultimate Withdrawal Load:** (Drift Bolt)
\[ d = \frac{3}{8} \text{ in} \]
\[ L = 6 \text{ in} \]

\[ P = 6,600 \times 0.5 \times (\frac{3}{8}) \times (\frac{7}{8}) \times (6) \]
\[ P = 8.662.5 \text{ lbs/bolt} \]

### 4 Bolts Per Bent (8 bolts total)

\[ P = 8 \times 8.662.5 = 69300 \text{ lbs} \]

\[ P = 69.3 \text{ kips} \]

**Lateral Resistance:** (bolts in bridge beams)

*It is assumed that the drift bolts behave similar to wood nails*

\[ P = K \times D^{0.7} \]

\[ P = 8 \times (1800.67) = 14405.4 \text{ lbs} \]

\[ P = 14.4 \text{ kips} \]

---

National Brand 42-182 100 SHEETS
Buoyancy Calculations:

- Hollow Core Planks:

\[
A_1 = (4 + 1\%)(2 + 4\%) = 11.3798 \text{ ft}^2
\]

\[
A_2 = 2 \left[ \frac{3}{4} \left( \frac{1}{4} \right)^2 \right] = 2.7925 \text{ ft}^2
\]

- \( A_T = A_1 - A_2 = 8.4873 \text{ ft}^2 \)

- 9 Hollow core planks
  \( A_T = 9(8.4873) = 76.3837 \text{ ft}^2 \)

\[
A_T = 76.367 \text{ ft}^2
\]

Hollow Core Planks

\( L = 70 \text{ ft} \)
**Self Weight: cont.**

0 **Bridge Deck:**

- **Deck Area**
  - Length = 70 ft
  - $A_T = 28.69 \text{ ft}^2$

0 **Railing:**

- **Window:**
  - $A = \frac{1}{2} (3 \times 1.6) = 2.4 \text{ in}^2 = 0.024 \text{ ft}^2$

- **Posts:**
  - $A = (2.5 \times 4) = 10 \text{ in}^2 = 0.125 \text{ ft}^2$

- **One Set of Railing:**
  - Amount of Windows = $(7 \times 3) = 21 (-)$
  - Amount of Railing = $7 \times 3 = 21$
  - Amount of Posts = 1
  - $A_T = 21 (5.00) - (21) (0.78125) + 9.1667$
  - $A_T = 97.7604 \text{ ft}^2$

- **Volume of all Railing:**
  - $V_T = (97.7604) (4) (1.0)$
  - $V_T = 391.042 \text{ ft}^3$

**Total Volume of Railing**

$V_T = 391.042 \text{ ft}^3$
Buoyancy Calculations: cont.

- Self Weight:

  - Concrete Volume = [Planks + deck] (l) + Railing
    = [76.362 + 38.691] (70) + 391.042
    \[ CV = 7,745.12 \text{ ft}^3 \]

  - Self Weight = \[ CV \times \gamma_{rc} \]
    = \[ (7,745.12)(150) \]
    \[ SW = 1,161,777 \times 10^6 \text{ lb} = 1,161.77 \text{kips} \]

- Buoyant Force:

  - BF = \[ SW \times \rho_{sw} \]
    = \( \text{(Submerged Concrete + Air pocket)} \times \rho_{sw} \)
    = \[ (7,745.12)(64) \]
    \[ BF = 470,661 \text{ lb} = 470.66 \text{kips} \]

  - Residual Weight = 691.11 kips

Summary of Results:

- Self Weight = 1,161.77 kips
- Buoyant Force = 470.66 kips
- Residual Weight = 691.11 kips
- % Retained Weight = 59.49%

- Bridge is not Buoyant
Lateral Resistance:

Lateral resistance will be provided by the abutments, as the deck is tied into the abutment by #6 thimk bars.

\[ f_y = 60,000 \text{ psi} \]
\[ \mu = 0.8 \]

D) Bridge Width = 46' - 10" = 562"

\[ \frac{562}{8} = 70.25 \text{ #6 (Hook Bar)} \]
\[ \frac{562}{4} = 140.5 \text{ #6 (Bent Bar)} \]

Total = 210 #6

E) \[ A_{et} = 210 (0.44) = 92.4 \text{ in}^2 \]

IV) \[ V_n = A_{et} f_y \mu \]
\[ = (92.4)(60)(0.8) \]
\[ V_n = 4435.2 \text{ kips} \]

⇒ For both abutments
\[ V_n = 2(4435.2) \]
\[ V_n = 8870.4 \text{ kips} \]

Lateral Resistance
**Vertical Capacity:**

*Note:* The vertical capacity is assumed to be dependent on the tensile strength of the #6 reinforcement.

The abutments are confined with #5 cross ties on a 12"x12" grid increasing the strength of concrete.

#6 Reinforcement tying the deck to the abutment will control the capacity of the bridge.

---

**Reinforcement Conforms to ASTM A 615**

- **Grade 60**
  - Fy = 60,000 psi
  - Fy = 90,000 psi

*From ASTM A 615:*

- #6 Bars:
  - Yield Strength = 26,400 lbs
  - Tensile Strength = 39,600 lbs

**Vertical Capacity = 210 (Tensile) = 210 (39.6)**

Vertical Cap. = 8316 kips

- For both abutments
  
  \[
  P_n = (8316)(a) = 16632 \text{ kips}
  \]

  \[
  P_n = 16632 \text{ kips}
  \]

Vertical Resistance provided by abutment tie ins.
(tensile strength)
**Deck Capacity:**

* Note: The negative moment caused by wave loads, in addition to the upward camber of the pre stressing may cause the hollow core to fail.

* Note: If the stresses in the concrete exceed:
\[ f_c = 12 \sqrt{f_e} \] (From PCI Handbook Section 1.8.1.2)
\[ f_e = (12000)(0.006)^2 \]
\[ f_e = 1073.31 \text{ psi} \]
The bridge deck will fail.

- \[ f_c' = 8000 \text{ psi} \]
- \[ f_{pu} = 2640,000 \text{ psi} \]
- (53) 2/3% Prestress Stands:
  - \[ A_p = 0.153 \text{ in}^2/\text{strand} \]
  - \[ P_c = 132.5 \text{ kips/strand} \]
  - Stands are not hinged
  - Uncoated - low relaxation strands
- \[ E_c = 53,000 \sqrt{f'_c} = 5,098 \times 10^6 \text{ psi} \]

**Compute Geometric Properties:**
* Assume rectangular section

\[ \Gamma_0 = \frac{5}{12} bh^3 = \frac{5}{12} (48)(24)^3 \]
\[ \Gamma_y = 106.101 \text{ in}^4 \]

\[ \Gamma_{circle} = \frac{\pi}{4} r^4 = \frac{\pi}{4} (8)^4 \]
\[ \Gamma_{circle} = 3216.99 \text{ in}^4 \]

\[ \Gamma_c = \Gamma_y - 2 \Gamma_{circle} \]
\[ = 106.101 - 2(3216.99) \]
\[ \Gamma_c = 9966.74 \text{ in}^4 \]

\[ A_c = (68)(24) = 1634 \text{ in}^2 \]
\[ C = \frac{h}{2} = 14 \text{ in} \]

\[ S = \Gamma / A_c = 7119.1 \text{ in}^3 \]

\[ I = \Gamma / A_c = 61.3715 \text{ in}^3 \]

**Compute Moments:**
* During storm event assume no ice loads

- From Plans:
  - Future wearing surface = 2.5 psf

- For own plant: (self weight)
  \[ \omega_p = \left[ \left(8 \frac{4}{8} + (4 + 4\%)(8.5 \frac{4}{8}) \right) \right] = 1605.1/\text{in} \]

\[ w = (46 + (4 + 1\%)) + 1605.08 = 1225.91 \frac{1}{8} \]
\[ w = 143.5 \frac{1}{8} \text{ in} \]

\[ M_{max} = \frac{wL^2}{8} = \frac{(143.5)(840)^3}{8} \]
\[ M_{max} = 1.26855 \times 10^9 \text{ lb-in} \]
Deck Capacity: cont.

- Wave Force Induced Moment:
  * Note: The deck is small relative to storm surge waves as the period and wave lengths increase during such events.
  - Assume that the wave will produce a load on the entire bridge.

\[ M_{wor} = \frac{w_{wor} L^2}{8} \]

- Compute stresses:
  - I) In negative bending: Tensile force & \( f_b = 12 \sqrt{k_c} = 1073.3 \text{ psi} \) (1)
    - But will be in compression & \( f_c = 0.85 \times 660 = -6800 \text{ psi} \) (2)

  \[ f_b = \frac{P}{A} \left( 1 - \frac{c}{t} \right) - \frac{M}{S} = f_b \]

  \[ = \frac{- (1325 \times 10^3)}{(1624)} \left( 1 - \frac{(48 \times 10^3)}{61.3715} \right) = \frac{M}{7119.1} = 1073.3 \]

  \[ 100800 - \frac{M}{7119.1} = 1073.3 \]

  \[ M = -464379 \]

  \[ IV \]

  \[ M = M_{max} - w_{wor} L^2 = -464379 \text{ lb-in} \]

  \[ w_{wor} (840)^3 = -464379 - 126856 \times 10^6 \]

  \[ w_{wor} = 147.092 \tfrac{\text{ lb-ft}}{\text{ in}} \]

  - A distributed load of 147.09 \tfrac{\text{ lb-ft}}{\text{ in}}
    will not fail a single plank

  \[ \Rightarrow \] Each plank can resist:

  \[ F_w = (147.092 \times 840) \times 1.25 = 135237 \text{ lbs} \]

  For 9 planks:

  \[ F_w = 9 (135237) = 1.2713 \times 10^6 \text{ lbs} \]

  \[ F_w = 112713 \text{ kips} \]

  Capacity in Tension
**Deck Capacity: Cont.**

**Concrete Shores: cont.**

**W**

\[ f_b = \frac{-P_e}{A_e} \left( 1 + \frac{2E}{f_{tu}^2} \right) + \frac{M_b}{s} \]

\[ = \left( \frac{1385 \times 10^3}{1124} \right) \left( 1 + \frac{448}{449} \right) + \frac{M_b}{711.1} = -6800 \]

\[-2631.85 + \frac{M_b}{711.1} = -3800 \]

\[ M_b = -2.96165 \times 10^7 \text{ lb-in} \]

**E**

For \( M_b \):  
Wave moment will be additive to prestressing  
\[ M_b = M_{\text{max}} - \frac{w_{ux}L^2}{8} \]

\[ M_b = (1.26855 \times 10^7) - \frac{w_{ux}L^2}{8} = -2.96165 \times 10^7 \]

\[ -\frac{w_{ux}L^2}{8} = -4.2302 \times 10^7 \]

\[ w_{ux} = \frac{(+4.2302 \times 10^7)(y)}{(640)^2} \]

\[ w_{ux} = 479,615 \text{ lb/ft} \]

\[ w_{ux} \text{ will not fail a single plank} \]

\[ \Rightarrow \text{ Each plank can resist:} \]

\[ F_u = (479,615)(840) = 402,876 \text{ lbs} \]

For 9 planks:

\[ F_u = 9 \times 402,876 = 3,625,884 \text{ lbs} \]

**Capacity in Compression**
Deck capacity: cont.

- Buoyant Force Addition:
  * Note: If the bridge becomes submerged, the self weight will be reduced.
  ⇒ Assume bridge is submerged to top of the deck.

2) \( w_D = [(8.495) + (4 + 10\%) (8.5\%)] (180 - 64) \)
   \( w_D = 920 \text{ in} \times 76.6293 \text{ lb/in} \) (self weight for one plant)

3) \( w = (35 \times (4 + 10\%)) \text{ in} + w_0 = 86.7861 \text{ lb/in} \)

\[
M_{max} = \frac{wL^2}{8} = \frac{(46.7361)(840)^2}{8} \\
M_{max} = 7,650,120 \text{ lb-in}
\]

4) \( f^2 = \frac{P_c}{A_o} (1 - \frac{e^2}{r^2}) - \frac{M_s}{s} = 65 \)
   \[
   = (132.9 \times 10^3) (1 - \frac{9.6 \times 10^2}{61.374}) - \frac{M_s}{719.1} = 1073.31 \\
   1008.08 - \frac{M_s}{719.1} = 1073.31 \\
   M_s = 464,379 \text{ lb-in}
   \]

⇒ \( M_{max} - \frac{w_{or} L^2}{8} = -464,379 \)
   \[
   \frac{w_{or} L^2}{8} = 7,160,630 \times 10^2 + 464,379 \\
   w_{or} = 86,4513 \text{ lb/in}
   \]

4) Force resisted by 9 plants:
   \( F_{or} = 9 \times (86.4513)(840) = 663,572 \text{ lbs} \)
   \[
   F_{or} = 653,572 \text{ kips}
   \]

Capacity in Tension
Deck Capacity: cont.


\[
\begin{align*}
V_f &= -\frac{Pe}{Ac} (1 + \frac{C_s}{h^2}) + \frac{M_b}{S} = f_c \\
M_b &= -2.96165 \times 10^3 \text{ lb-in (same as before)} \\
\Rightarrow M_b &= 7.65012 \times 10^4 - \frac{6 \omega L^2}{8} = -2.96165 \times 10^3 \\
&\quad - \frac{6 \omega L^2}{8} = -3.7815 \times 10^3 \\
&\quad \omega L^2 = 383.391 \text{ lb-in} \\
\Rightarrow \text{Force Resist by 9 planks:} \\
F_w &= (9)(383.391 \times 840) = 3.898 \times 10^6 \text{ lbs} \\
\boxed{F_w = 2.848,444 \text{ kips}} \\
\text{Capacity in compression}
\end{align*}
\]
I. **Deck Capacity:**

- **Deflection:** (caused by estimated wave loads)
  - From plans sheet 5.11.1

\[
\text{Camber (upward)} = \text{Camber at erection} - \Delta d (\text{deck})
\]
\[
= 2.74 - 0.64
\]
\[
\text{Camber (upward)} = 2.10\text{ in}
\]

- To calculate upward deflection due to wave forces use:

\[
S_{w} = \frac{S \cdot w \cdot a \cdot l^4}{384 \cdot E_c \cdot I_c}
\]

*Note: The section is cracked as \( f_u = 7.5 \sqrt{f_e} \) has been exceeded
- Inelastic must be used

\[ d_p = (5.5) + (28.4.2) = 29.3\text{ in} \]

\[ S_p = \text{Aps} = (0.153 \times 53) = .004722 \]

\[ \text{b} \cdot d_p = (50)(29.3) \]

- \[ E_c = 57,000 \sqrt{f_e} = 57,000(800)^{\frac{1}{2}} \]

\[ E_c = 5.0922 \times 10^6 \text{ psi} \]

\[ F_p = 38.5 \times 10^6 \text{ psi} \]

\[ N_p = \frac{F_p}{E_c} = \frac{(38.5 \times 10^6)}{5.0922 \times 10^6} = 7.59017 \]

\[ I_{cr} = N_p \cdot A_p \cdot d_p^4 \cdot (1 - 1.6 \frac{\sqrt{N_p}}{\sqrt{p}}) \]

\[ = (5.59017)(0.109)(29.3)^2(1 - 1.6(5.59017 \times 0.004722)^{\frac{1}{2}}) \]

\[ I_{cr} = 499086.4 \text{ in}^4 \]

- **No live load**
  - \( M_a = 0 \)

\[ I_c = (\frac{M_a}{M_n})^3 I_g + [1 - (\frac{M_a}{M_n})^3] I_{cr} \]

\[ I_c = I_{cr} = 499086.4 \text{ in}^4 \leq I_g \]

- **Deflection:**

<table>
<thead>
<tr>
<th>Buoyant Force</th>
<th>( w_w ) (lbs)</th>
<th>( S_w ) (in)</th>
<th>Existing Camber (in)</th>
<th>Total (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Not Included</td>
<td>2 ( w_w ) = 141,092 lb/in</td>
<td>840</td>
<td>3.86223</td>
<td>2.15</td>
</tr>
<tr>
<td></td>
<td>3 ( w_w ) = 477,615 lb/in</td>
<td>840</td>
<td>12.4244</td>
<td>2.15</td>
</tr>
<tr>
<td>Buoyant Force</td>
<td>3 ( w_w ) = 86,403 lb/in</td>
<td>840</td>
<td>2.23452</td>
<td>2.29</td>
</tr>
<tr>
<td>Included</td>
<td>4 ( w_w ) = 383,341 lb/in</td>
<td>840</td>
<td>9.13176</td>
<td>2.29</td>
</tr>
</tbody>
</table>

\* AASHTO max. Permissible Deflection
Summary of Results:

- Unsubmerged Case: (i.e. no buoyant force)
  \[ F_{ur} = 112.713 \text{ kips (total capacity)} \times \text{Weak in tension} \]
  \[ W_{ur} = 149.09 \text{ kips (Plant distributed load capacity)} \]

- Submerged Case: (i.e. buoyant force included)
  \[ F_{ur} = 653.57 \text{ kips (total capacity)} \times \text{Weak in tension} \]
  \[ W_{ur} = 86.45 \text{ kips (Plant distributed load capacity)} \]

\[ \text{Reduction in capacity once submerged:} \]
  - Tension = 42.01%
  - Compression = 20.06%
**Bouyancy Calculations:**

**Self Weight:**

1. **Tri-deck A:**

   \[ A_1 = (59.625 \times 4.5) / 20 \] ft²

   \[ A_2 = (59.625 \times 0.75) / 20 \] ft²

   \[ A_3 = (59.625 \times 1.5) / 20 \] ft²

   \[ A_4 = 2 \left( \frac{3}{4} \times \frac{3}{16} \right) \] ft²

   \[ A_T = A_1 + A_2 + 3 \times A_3 - 3 \times A_4 - A_5 \]

   \[ A_T = 2.641 + 3.756 + 3 \times 1.6255 - 3 \times 0.1883 - 1.7453 \]

   \[ A_T = 8.759 \] ft³

   \[ \boxed{A_T = 8.759 \text{ ft}^3} \] Tri-deck A Concrete

2. **Air Pocket:**

   \[ A_a = \left( \frac{3}{4} \right) \times \left( \frac{3}{16} \right) + 0.0833 \]

   \[ A_a = 1.4187 \] ft³

   \[ A_T = 2 \times A_a = 2.833 \] ft³

   \[ \boxed{A_T = 2.833 \text{ ft}^3} \] Tri-deck A Air Pocket
**Self Weight: cont.**

- **Trideck B, C, D:**

  \[4' - 11 \frac{3}{8}'' = 59.625''\]

  \[
  A_1 = \left(59.625'' \times \frac{6}{8}\right) = 2.6914 \, \text{ft}^2
  \]

  \[A_2 = \left(59.625'' \times \frac{4}{8}\right) = 1.65625 \, \text{ft}^2\]

  \[A_3 = \left(9.25'' \times \frac{1}{8}\right) = 1.0625 \, \text{ft}^2\]

  \[A_4 = 2 \left(\% \times \frac{1}{8}\right) = 0.14583 \, \text{ft}^2\]

  \[A_T = A_1 + A_2 + 3A_3 - 3A_4 = 2.69 + 1.65625 + 3(1.0625) - 3(0.14583) = 7.08204 \, \text{ft}^2\]

  \[\boxed{A_T = 7.082 \, \text{ft}^2}\]  

- **Concrete:**

  \[A_{\text{Air Pocket}} = \left(\% \times \frac{1}{8}\right) + 2 \left(\% \times \frac{1}{8}\right) = 1.89583 \, \text{ft}^3\]

  \[A_T = 2A_{\text{Air Pocket}} = 3.79166 \, \text{ft}^3\]

  \[\boxed{A_T = 3.792 \, \text{ft}^3}\]  

A-89
Self Weight: cont.

- **Railing**:

  
  ![Diagram](image)

  \*
  
  * The Area of the railing is estimated from sheet 553
  
  \[
  A_1 \approx (13 \times 0.03)(\frac{4}{48}) = 0.825 \text{ ft}^2
  \]
  
  \[
  A_2 \approx (19)(8.0)(\frac{4}{48}) = 0.833 \text{ ft}^2
  \]
  
  \[
  A_3 \approx (12)(10.5)(\frac{4}{48}) = 0.875 \text{ ft}^2
  \]
  
  \[
  A_T = A_1 + A_2 + A_3 = 2.533 \text{ ft}^2
  \]

- **Missing Volume**: (shaded area)

  \[
  V_m = (6)(12+3)(8) = 720 \text{ in}^3 = 0.4167 \text{ ft}^3
  \]

- **Railing Volume**:

  \[
  V_R = (2.533)(170) - (0.0169)(90)
  \]

  \[
  V_R = 401.667 \text{ ft}^3
  \]
**Air Pocket**:
- Between Trideck A & D:

\[
A_t = \left(\frac{25.993}{100}\right)\left(\frac{6}{100}\right) = 1.54167 \text{ ft}^3
\]

\[
A_z = \frac{14}{9}\left(\frac{6}{100}\right)^2 = 0.073717 \text{ ft}^3
\]

\[
A_1 = \frac{14}{9}\left(\frac{6}{100}\right)\left(\frac{6}{100}\right) = 0.046944 \text{ ft}^3
\]

\[
A_2 = \left(\frac{6}{100}\right)\left(\frac{6}{100}\right) = 0.240884 \text{ ft}^3
\]

\[
A_T = 2A_1 = 1.84241 \text{ ft}^3
\]

\[
A_T = 1.842 \text{ ft}^3
\]

- Between Tridecks B, C, & D:

\[
A = (13.830)(x_1)(1/41) + \frac{1}{2}(\frac{1}{4}X_4)(1/4) = 2.1682 \text{ ft}^3
\]

\[
A_T = 2.169 \text{ ft}^3
\]

**Compression of Air Pocket**:
- For all Pockets (Assume submerged to top of deck)
  - \( h = 25 + 6.5 + \frac{1}{2} = 53.5 \) in
  - \( h = 2.39 \) ft
  - \( P_e = P_h + h \times \frac{6}{100} \)
    \[
    = 14.3 + 2.39 \times \frac{6}{100}
    \]
    \[
    = 15.9407 \text{ psi}
    \]
  - \( P_e/P_h = 14.3/15.94 = 0.90216 \)

- Between B, C, & D:

\[
A = (2.167)^2 \times 0.90216
\]

\[
A_T = 1.809 \text{ ft}^3
\]

- Between A & D:

\[
A = (3.833)^2 \times 0.90216
\]

\[
A = 3.49654 \text{ ft}^3
\]

- Between A, D:

\[
A = 1.74185 \text{ ft}^3
\]

- Trideck A:

\[
A = 2.6128 \text{ ft}^3
\]

- Trideck B, C, D:

\[
A = 3.49654 \text{ ft}^3
\]

- Trideck A, B:

\[
A = 1.74185 \text{ ft}^3
\]

A-91
Self Weight: \( y_{rc} = 160 \text{psf} \)

Concrete Volume = \((\text{Tirfck A} + 9 \text{ Tiedck B, C, D})(190) + 3 \text{ Railing} = (8.759 + (9)(7.082))(190) + (3)(401.667) \)

\( CV = 135.9 \times 1.5 \text{ ft}^3 \)

AC Volume = \( (3y)(40)(190) = 1133.33 \text{ ft}^3 \)

Self Weight = \( CV \times y_{rc} + AC \times y_{ac} = (135.9)(160) + (1133.33)(152) \)

Self Weight = 2,33699 \times 10^6 \text{ lbs}

Buoyant Force:

* Assume submerged to top of deck

Submerged Volume = Concrete + Railing + AC Pneumatic + Air Packet

\( = (135.99.5 - (9)(401.667)) + (1133.33) + (0)(2.6118) + (3)(495454) + (1)(1.9114) + 8(2.0)(190) \)

\( SV = 22264.3 \text{ ft}^3 \)

Buoyant Force = \( SV \times y_{sw} = (22264.3)(64) \)

\( BF = 1.42491 \times 10^6 \text{ lbs} \)

Residual Weight:

\( RW = SW - BF = (2.33699 \times 10^6 - 1.4291 \times 10^6) \)

\( RW = 912076 \text{ lbs} \)

\% Retained Weight = 39.0 \%

Summary of Results:

- Self Weight = 2337.0 kips
- Buoyant Force = 1424.9 kips
- Residual Weight = 912.1 kips
- \% Retained = 39.0 \%

Bridge is NOT Buoyant
**Trickle A: Deck Capacity:**

*Note: If the stresses in the concrete exceed:

\[ f_b = 12 \sqrt{f_{ck}} \]
\[ = 12 \left( \frac{1000}{20} \right) \]
\[ f_b = 113.8 \text{ psi (Tension)} \]
\[ f_c = 0.85 f_e \]
\[ = 0.85 \left( \frac{1000}{20} \right) \]
\[ f_c = 76.50 \text{ psi (Compression)} \]

The deck will fail.

\[ f_b = 9000 \text{ psi}; \]
\[ f_p = 2700 \text{ psi}; \]
\[ P_s = 733 \text{ kips} \]

Shovels are handled by aluminum stands:

\[ E_c = 57,000 \sqrt{f_{ck}} = 5.40 \times 49 \times 10^6 \text{ psi}; \]

---

**Compute Geometric Properties:**

- **Moment of Inertia:**

  \[ I_x = \frac{1}{6} bh^3 = \frac{1}{6} \left( 69.625 \right) \left( 15.5 \right)^3 = 18503 \text{ in}^4 \]

  \[ I_y = \frac{1}{6} bh^3 = \frac{1}{6} \left( 175 \right) \left( 15.5 \right)^3 = 298647 \text{ in}^4 \]

  \[ I_z = \frac{1}{2} b h^3 = \frac{1}{2} \left( 69.625 \right) \left( 15.5 \right)^3 = 13564 \text{ in}^4 \]
**Tribalcon A: Capacity:**

- **Compute Moments:**
  
  * During storm event assume no live loads
  
  * From plan: Tire track wearing surface = 25psi = 12.41 ksi (for one deck)
  
  * Dead load = (8.750)(160) = 1401.44 ksi

  \[
  \text{Total } W_D = 1526.66 \text{ ksi} = 127.138 \text{ ksi}
  \]

  \[
  M_{\text{max}} = \frac{W_D l^2}{8} = \frac{(127.138 \times 492)^2}{8} \text{ kip-in}
  \]

  \[
  M_{\text{max}} = 9.946 \times 10^6 \text{ kip-in}
  \]

  - Wave induced moment:
    
    * Assume that wave will produce a distributed load on entire deck of bridge

    \[
    M_{\text{wve}} = W_{\text{wve}} l^2/8 = 78408 \text{ kip-in}
    \]

  - Compute Stresses:
    
    1) In negative bending: Top will be in tension \( f_t = +1138.42 \text{ psi} \)
    
    But will be in compression \( f_c = -7680 \text{ psi} \)

    2) At center:

    \[
    E_c = 1865 - 5.86 = 12.797 \text{ kip-in}
    \]

    \[
    f_t = \frac{P_t}{A_c} (1 - \frac{E_c}{E_s}) - \frac{M_t}{I} \leq f_c
    \]


  \[
  f_t = -7680 \left(1 - \frac{12.797}{12.797}\right) - \frac{1138.42}{95.05} = 1138.42
  \]

  \[
  (1319.06)
  \]

  \[
  + 5.08386 \times 10^6 - M_t = 8.87212 \times 10^6
  \]

  \[
  M_t = -3.39825 \times 10^6
  \]

  \[
  M_{\text{max}} = 78408 \text{ kip-in} = -3.39825 \times 10^6
  \]

  \[
  W_{\text{wve}} = M_{\text{max}} + 3.39825 \times 10^6
  \]

  \[
  W_{\text{wve}} = 78408
  \]

  \[
  W_{\text{wve}} = 175.453 \text{ kips}
  \]

  II: Tribalcon A: Can resist: (in tension)

  \[
  F_{\text{wve}} = 175.453 (942) / 10^6
  \]

  \[
  F_{\text{wve}} = 138.959 \text{ kips}
  \]

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Truss Deck A Deck Capacity: cont.


2. $P_b = \frac{P_e}{ \frac{r}{k} + \frac{M}{M_b} \leq 0.85 \psi'}$

3. $M_{max} = 2.84 \text{ kips}$

4. The truss can resist: (in compression)

   $F_{cr} = \sqrt{f_{pm}^2 + \psi''^2} = 382.51 \text{ kips}$

Truss Deck B & C Deck Capacity:

- 60 in
- 42 in

- 20 in

- 9.25 in

- $y_b = 12.8428 \text{ in}$

- $y_b = 18.6572 \text{ in}$

Compute Geometric Properties:

- $A_1 = (59.605)(10.5) = 626.068 \text{ in}^2$

- $y_1 = 26.49 \text{ in}$

- $A_2 = (3)(52.25\times 21) = 582.75 \text{ in}^2$

- $y_2 = 10.5 \text{ in}$

- $y_c = (626.068)(52.25) + (582.75 \times 10.5)$

- $y_c = 18.6572 \text{ in}$

- $I_1 = \frac{1}{12}bh^3 = \frac{1}{12}(59.605)(10.5)^3$

- $I_1 = 626.119 \text{ in}^4$

- $I_2 = \frac{1}{12}bh^3 = \frac{1}{12}(9.25 \times 21)^3$

- $I_2 = 7.3869 \text{ in}^4$
Trideck B & C Deck Capacity: Cont.

- Compute Moment:
  \[ W_D = (7.082 \times 160) + 124.219 = 1257.34 \text{ lb} \]
  \[ W_D = 104.778 \text{ lb/ft} \]
  \[ M_D = W_D \frac{1}{8} = (104.778 \times 243) \frac{1}{8} \]
  \[ M_D = 8.21543 \times 10^6 \text{ lb-in} \]

- Temple Limit:
  
  (1) At center:
  \[ C_c = 18.6572 - 6.13 = 12.5272 \text{ in} \]
  
  (2) Using equations developed during this Thesis:
  \[ W_{w,1} = \frac{8}{L^2} \left( M_D + \frac{56 \ell_t}{\ell_c} \right) \left( \frac{56 \ell_t}{\ell_c} \right) - \left( \frac{56 \ell_t}{\ell_c} \right) \frac{A_c}{\ell_t} \left( 1 - \frac{C_c}{\ell_t} \right) \]
  \[ W_{w,1} = \frac{8}{(742)^3} \left( 8.21543 \times 10^6 + (5469.04 \times 1138.42) - (5469.04 \times 303.000) \right) \left( 1 + \frac{13.5272 \times 13.5272}{84.41} \right) \]
  \[ W_{w,1} = 157.806 \text{ lb/ft} \]
  \[ F_{w,1} = 124.98 \text{ kips (tension)} \]

- Compress Limit:
  \[ W_{w,2} = \frac{8}{L^2} \left( M_D - \frac{56 \ell_t}{\ell_c} \right) \left( \frac{56 \ell_t}{\ell_c} \right) - \left( \frac{56 \ell_t}{\ell_c} \right) \frac{A_c}{\ell_t} \left( 1 + \frac{C_c}{\ell_t} \right) \]
  \[ W_{w,2} = \frac{8}{(742)^3} \left( 8.21543 \times 10^6 - (5469.04 \times 7660) - (5469.04 \times 303.000) \right) \left( 1 + \frac{13.5272 \times 13.5272}{84.41} \right) \]
  \[ W_{w,2} = 476.825 \text{ lb/ft} \]
  \[ F_{w,2} = 37.74 \text{ kips (compression)} \]

Trideck D Deck Capacity:

- Geometric Properties:
  \[ I_c = 102037 \text{ in}^4 \]
  \[ A_c = 1208.81 \text{ in}^2 \]
  \[ y_a = 12.8428 \text{ in} \]
  \[ y_b = 16.0532 \text{ in} \]
  \[ p_c = 84.411 \text{ in}^2 \]
  \[ P_c = 714,000 \text{ lbs} \]
  \[ G_c = 13.5272 \text{ in} \]

- Bending Moment:
  \[ W_{w,3} = \frac{8}{L^2} \left( M_D + \frac{56 \ell_t}{\ell_c} \right) - \left( \frac{56 \ell_t}{\ell_c} \right) \frac{A_c}{\ell_t} \left( 1 - \frac{C_c}{\ell_t} \right) \]
  \[ W_{w,3} = \frac{8}{(742)^3} \left[ 8.21543 \times 10^6 + (5469.04 \times 1138.42) - (5469.04 \times 303.000) \right] \left( 13.5272 \times 13.5272 \right) \frac{84.411}{1208.81} \]
  \[ W_{w,3} = 156.804 \text{ lb/ft} \]

Tensile Limit:

\[ W_{w,4} = \frac{8}{L^2} \left( M_D - \frac{56 \ell_t}{\ell_c} \right) - \left( \frac{56 \ell_t}{\ell_c} \right) \frac{A_c}{\ell_t} \left( 1 + \frac{C_c}{\ell_t} \right) \]

\[ W_{w,4} = 124.189 \text{ kips} \]
Trideck D Capacity - cont.

Compressive Limit:

\[ W_c = \frac{8}{L^2} \left[ M_0 - (5469.04) - 6\pi \rho C \left( \frac{14.572}{r^2} \right) \right] \]

\[ = \frac{8}{(943)^2} \left\{ (8.21543 \times 10^6) - (5469.04) \times (-7650) \times (\frac{5469.04 \times 10^3 \times 14.000 \times (1 + 13.672 \times 14.572)}{1205.81}) \right\} \]

\[ W_c = 0.242 \times (8.21543 \times 10^6 + 2.8949 \times 10^7) \]

\[ W_c = 473.192 \text{ kips} \]

\[ 	ext{Fur} = 375.402 \text{ kips (compression)} \]

Submerged Case:

* It is assumed bridge is submerged to top of the deck

Buoyant Force must be considered

Trideck A:

D. \( W_D = \text{Dead loads} - \text{Buoyant Force} \)

\[ W_D = (124.219) + (8.759 \times 160) - (5.759 + 2.628 \times 64) \]

\[ W_D = 797.864 \text{ lb/in} \]

\[ W_D = 66.487 \text{ lb/in} \]

\[ M_0 = 5.21324 \times 10^6 \text{ lb-in} \]

D. \( W_{A6} = \frac{8}{L^2} (M_0 + 3.79176 \times 10^6) \)

\[ W_{A6} = 114.803 \text{ lb/in} \]

\[ 	ext{Fur} = 90.92 \text{ kips (tension)} \]

D. \( W_{AE} = \frac{8}{L^2} (M_0 + 2.78497 \times 10^7) \)

\[ W_{AE} = 422.318 \text{ lb/in} \]

\[ 	ext{Fur} = 334.47 \text{ kips (compression)} \]

Trideck B & C:

D. \( M_0 = 48.3574 \text{ lb/in} \)

\[ M_0 = 3.79176 \times 10^6 \text{ lb-in} \]

D. \( W_{AE} = \frac{8}{L^2} (M_0 + 4.13984 \times 10^6) \)

\[ W_{AE} = 101.388 \text{ lb/in} \]

\[ 	ext{Fur} = 80.849 \text{ kips (tension)} \]

D. \( W_{AE} = \frac{8}{L^2} (M_0 + 2.91479 \times 10^7) \)

\[ W_{AE} = 420.106 \text{ lb/in} \]

\[ 	ext{Fur} = 332.724 \text{ kips (compression)} \]

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Submerged Case: cont.

- Trideck D:
  - $M_0 = 3.79176 \times 10^6$ lb-in (same as trideck B & C)
  - $\text{W_at} = \frac{8}{L^2} (M_0 + 4.07927 \times 10^6) = \frac{8}{742^2} (3.79176 \times 10^6 + 4.07927 \times 10^6)$
    - $\text{W_at} = 100.385$ kips
    - $F_{ur} = 79.8$ kips (tension)

- $\text{W_at} = \frac{8}{L^2} (M_0 + 2.89474 \times 10^7)$
  - $\text{W_at} = 417.574$ kips
  - $F_{ur} = 331.7$ kips (compression)

Bridge Deck Capacity: (Span #1)

- Unsubmerged Case:
  - Tension: $F_{ur} = (1.7)A + (1)B + (7)C + (1)D = 139.959 + 124.98 + (7)(14.89) + (14.89)$
    - $F_{ur} = 126.2$ kips
  - Compression: $F_{ur} = 377.7$ kips

- Submerged Case:
  - Tension: $F_{ur} = 812.812$ kips
  - Compression: $F_{ur} = 3326.98$ kips

Summary of Results:

- Unsubmerged:
  - Tension: $F_{ur} = 126.2$ kips
  - Compression: $F_{ur} = 377.7$ kips

- Submerged:
  - Tension: $F_{ur} = 812.812$ kips
  - Compression: $F_{ur} = 3326.98$ kips

- Loss due to Submerged:
  - Tension: $37.64 \%$
  - Compression: $11.92 \%$
**Typical Girder Section: Precast Tee Girder (Widened Section)**

- Sheet 5 of widening

\[
A_1 = (1 + \frac{3}{12})(1.5') = 0.1667 \text{ ft}^2
\]

\[
A_2 = (1 + 1.25)(2.5') = 0.2387 \text{ ft}^2
\]

\[
A_3 = (1 + \frac{4}{12})(1.5') = 0.1667 \text{ ft}^2
\]

\[
A_4 = 2\left[\frac{1}{2}(\frac{1}{12})(\frac{1}{3})\right] = 0.1111 \text{ ft}^2
\]

\[
A_5 = (\frac{1}{3})x^2 + \frac{4}{12} + \frac{3}{6} = 1.861 \text{ ft}^2
\]

\[
A_6 = 2\left[\frac{1}{2}(\frac{3.875}{12})(\frac{3}{2})\right] = 0.0549 \text{ ft}^2
\]

\[
A_T = \sum_{i=1}^{6} A_i = 2.60417 \text{ ft}^2
\]

- **Precast Tee Girder (Widened Section)**

**Typical Girder Section: Precast Tee Girder (Existing Section)**

\[
A_1 = (16.875')(1.5') = 0.1758 \text{ ft}^2
\]

\[
A_2 = \frac{1}{12}x \cdot \frac{5}{12} = 0.2430 \text{ ft}^2
\]

\[
A_3 = (16.875')(1.5') = 0.1758 \text{ ft}^2
\]

\[
A_4 = 2\left[\frac{1}{2}(16.875')(1.5')\right] = 0.1172 \text{ ft}^2
\]

\[
A_5 = (\frac{1}{3})x^2 + \frac{4}{12} + \frac{3}{6} = 1.8611 \text{ ft}^2
\]

\[
A_6 = 2\left[\frac{1}{2}(\frac{3}{2})(\frac{3}{2})\right] = 0.0625 \text{ ft}^2
\]

\[
A_T = \sum_{i=1}^{6} A_i = 2.63542 \text{ ft}^2
\]

- **Precast Tee Girder (Existing Section)**
**Deck Section:**

*Note:

The deck is sloped with the high point at the center.

Slope is ignored for the purposes of these calculations.

\[ \gamma_{AC} = 162 \text{ kN/m}^2 \]
\[ \gamma_{RC} = 150 \text{ kN/m}^2 \]

- **AC Pavement:**
  \[ A_{AC} = (52 + 3\frac{1}{2}) \left( \frac{1}{52} \right) = 6.679 \text{ ft}^2 \]
  \[ A_{AC} = 6.70 \text{ ft}^2 \]  
  AC Pavement

- **Concrete:**
  \[ A_T = \left( [1 + \frac{3}{12}] + 2 \left[ \left( \frac{3}{8} + \frac{3}{12} \right) + 2 \left[ \left( \frac{3}{8} + \frac{3}{12} \right) \left( \frac{3}{12} \right) \right] \right) = 36.4028 \text{ ft}^2 \]
  \[ A_T = 36.40 \text{ ft}^2 \]  
  Concrete (Deck)
  Left railing not included

**Railings (Widened Section):**

\[ A_1 = \left( \frac{13}{12} \right) \left( \frac{1}{2} \right) = 0.662 \text{ ft}^2 \]
\[ A_2 = \left( \frac{3}{4} \right) \left( \frac{1}{2} \right) = 1.0 \text{ ft}^2 \]
\[ A_3 = \frac{1}{2} \left( \frac{1}{2} \right) \left( \frac{3}{12} \right) = 0.09167 \text{ ft}^2 \]
\[ A_4 = \left( \frac{1}{2} \right) \left( \frac{13}{12} \right) = 0.4233 \text{ ft}^2 \]
\[ A_5 = \frac{1}{2} \left( \frac{3}{2} \right) \left( \frac{3}{12} \right) = 0.29167 \text{ ft}^2 \]
\[ A_6 = \left( \frac{3}{12} \right) \left( \frac{3}{12} \right) = 0.11 \text{ ft}^2 \]

- **Total Railing Area:**
  \[ A_T = \frac{6}{5} A_c = 2.19444 \text{ ft}^2 \]
  \[ A_T = 2.19 \text{ ft}^2 \]  
  Railing Widened Section
Concrete Diaphragm:

- **Widened Section**:

![Diaphragm Diagram](image)

\[ A_1 = (3 + \frac{3}{12})(2\frac{4}{12}) = 7.78472 \text{ ft}^2 \]

\[ A_2 = 2 \left[ \frac{1}{2} \left( \frac{3}{12} \right) \left( \frac{3}{12} \right) \right] = 0.039931 \text{ ft}^2 \]

\[ A_3 = 2 \left[ \frac{1}{2} \left( \frac{4}{12} \right) \left( \frac{3}{12} \right) \right] = 0.059896 \text{ ft}^2 \]

\[ A_4 = 2 \left[ \frac{1}{2} \left( \frac{4}{12} \right) \left( \frac{3}{12} \right) \right] = 0.1111 \text{ ft}^2 \]

\[ A_T = A_1 - A_2 - A_3 - A_4 = 7.57378 \text{ ft}^2 \]

**Concrete Diaphragm (Widened Section)**

Thickness \( \approx 1 \text{ ft} \)

- **Existing Section**:

* Spacing center to center = 4' - 0"

\[ A_1 = (3 + 4/12)(2\frac{4}{12}) = 8.1994 \text{ ft}^2 \]

\[ A_2 = 2 \left[ \frac{1}{2} \left( \frac{3}{12} \right) \left( \frac{3}{12} \right) \right] = 0.04167 \text{ ft}^2 \]

\[ A_3 = 2 \left[ \frac{1}{2} \left( \frac{4}{12} \right) \left( \frac{4}{12} \right) \right] = 0.0625 \text{ ft}^2 \]

\[ A_4 = 2 \left[ \frac{1}{2} \left( \frac{4}{12} \right) \left( \frac{3}{12} \right) \right] = 0.1172 \text{ ft}^2 \]

\[ A_T = A_1 - A_2 - A_3 - A_4 = 7.77303 \text{ ft}^2 \]

**Concrete Diaphragm (Existing Section)**

Thickness \( \approx 1 \text{ ft} \)
Air Pocket Calculations:

Widened Section:

\[ A_1 = \left(3 + \frac{3}{15}\right)(\frac{3}{15})^3 = 7.78972 \text{ ft}^2 \]
\[ A_5 = \left(3 + \frac{3}{15}\right)(\frac{3}{15})^2 \times 1.3194 \text{ ft}^3 \]

\[ A_2 = 2 \left[ \left(3.9375\right)\left(\frac{3}{15}\right) \right] = 0.039931 \text{ ft}^2 (-) \]

\[ A_3 = 2 \left[ \frac{1}{2} \left(3.9375\right)\left(\frac{3}{15}\right)^2 \right] = 0.059846 \text{ ft}^2 (-) \]

\[ A_4 = 2 \left[ \frac{1}{2} \left(\frac{1}{15}\right)\left(\frac{3}{15}\right) \right] = 0.1111 \text{ ft}^2 (-) \]

\[ A_T = A_1 - A_2 - A_3 - A_4 + A_5 = 8.89323 \text{ ft}^3 \]

Existing Section:

\[ A_1 = \left(3 + \frac{4}{15}\right)(\frac{3}{15})^3 = 8.1944 \text{ ft}^2 \]
\[ A_5 = \left(3 + \frac{4}{15}\right)(\frac{3}{15})^2 \times 1.3194 \text{ ft}^3 \]

\[ A_2 = 2 \left[ \left(\frac{3}{15}\right)\left(\frac{3}{15}\right) \right] = 0.0404 \text{ ft}^3 (-) \]

\[ A_3 = 2 \left[ \frac{1}{2} \left(\frac{3}{15}\right)\left(\frac{3}{15}\right)^2 \right] = 0.0626 \text{ ft}^3 (-) \]

\[ A_4 = 2 \left[ \frac{1}{2} \left(\frac{1}{15}\right)\left(\frac{3}{15}\right) \right] = 0.1172 \text{ ft}^3 (-) \]

\[ A_T = A_1 - A_2 - A_3 - A_4 + A_5 = 9.3696 \text{ ft}^3 \]
Reduction in air pocket:

* Note: It is assumed that the bridge is submerged to the top of deck.

- **Widened Section:**
  - $h = (1.9\text{ft}) + (5/4\text{ft}) + (3) = 3.625\text{ft}$
  - $A_2 = 14.7 + h (6/44)$
  - $P_x = 14.7 + (3.625)(6/44)$
  - $P_x = 16.311\text{psi}$
  - $A_z = P_x A_1$ = $(14.7)(8.892)$
  - $P_x = 16.311$
  - $A_z = 8.01978\text{ ft}^2$
  - $A_T = 8.015\text{ ft}^2$ Compressed Air (Widened Section)

- **Existing Section:**
  - $h = 2.626\text{ft}$
  - $P_x = 16.311\text{psi}$
  - $A_z = P_x A_1$ = $(14.7)(9.362)$
  - $P_x = 16.311$
  - $A_z = 6.43728\text{ ft}^2$
  - $A_T = 8.437\text{ ft}^2$ Compressed Air (Existing Section)

Collection of Data:

- **AC Pavement:** ($Y_{AC} = 162\text{ lb/ft}^3$)
  - Area = 6.70\text{ ft}^2
  - Amount = 1
  - Length = 100.67\text{ ft}

- **Concrete:** ($Y_{AC} = 150\text{ lb/ft}^3$)
  - **Tee Girder:** (Widened)
    - Area = 2.60\text{ ft}^2
    - Amount = 7
    - Length = 100.67\text{ ft}
  - **Tee Girder:** (Existing)
    - Area = 2.64\text{ ft}^2
    - Amount = 9
    - Length = 100.67\text{ ft}
  - **Deck:**
    - Area = 36.40\text{ ft}^2
    - Length = 100.67\text{ ft}

- **Railway:** (Widened)
  - Area = 2.19\text{ ft}^2
  - Amount = 1
  - Length = 100.67\text{ ft}

- **Diaphragm:** (Existing)
  - Area = 7.97\text{ ft}^2
  - Amount = 9\times 5 = 45
  - Length = 1\text{ ft}
Collection of Data Continued:
- Compressed Air Packet: \( (\text{Air volume} = 64 \frac{1}{2} \text{ft}^3) \)
  - Widen:
    - Area = 8.015 \text{ ft}^2
    - Amount = 6
    - Length = 100.67 \text{ ft}
  - Existing:
    - Area = 8.437 \text{ ft}^2
    - Amount = 9
    - Length = 100.67 \text{ ft}

Self Weight:
\[
\text{Self Weight} = \text{Concrete} + \text{AC Pavement} = (160 \frac{\text{lbs}}{\text{cu ft}}) \left( (2.10 + 0.60) + 36.4 + 2.4 \right) (100.67) + (150 \frac{\text{lbs}}{\text{cu ft}}) (30 \times 7.57) = 1.40669 \times 10^6 \text{ lbs}
\]

Buoyant Force:
\[
\text{Submerged Volume} = \text{Air Packet} + \text{Concrete Volume} + \text{AC Pavement Volume} = (8.015 \times 2.10 \times 100.67) + (8.437 \times 9 \times 100.67)
\]
\[
\text{BF} = 1.31976 \times 10^6 \text{ lbs}
\]

Summary:
- Total Weight = 1406.7 kips
- Buoyant Force = 1319.8 kips
- Residual Weight = 86.9 kips
- % Weight Retained = 6.18%

\textbf{Bridge is NOT Buoyant}
**Deck Capacity (Widened Section)**

![Deck Diagram]

- **Positive Bending:**
  - \( A_3 = (42)(0.20) = 8.4 \text{ in}^2 \)
  - \( a = \frac{Asy}{0.85 t_e' b} = \frac{(4.4)(40)}{0.85(3)(314)} \approx 0.41963 \text{ in} \)
  - \( d = 6 - \frac{3}{4} = 0.6 \text{ in} \)
  - \( f_y = 0.0274 \geq \frac{1}{2} f_y = 0.0014 \) (ok)
  - \( Mn = Asy(d - \frac{3}{4}) = (4.4)(40)(5 - 0.9932) \)
  - \( Mn = 1809.5 \text{ k-in} = 134.125 \text{ k-ft} \)
  - \( \phi Mn = 0.90(134.125) = 120.713 \text{ k-ft} \)

  - **Positive Bending:**
    - \( Mn = 134.13 \text{ k-ft} \)
    - \( \phi Mn = 120.71 \text{ k-ft} \)

- **Negative Bending:**
  - \( A_3 = (12)(0.20) + (14)(0.60) = 10.8 \text{ in}^2 \)
  - \( a = \frac{Asy}{0.85 t_e' b} = \frac{(10.8)(40)}{0.85(3)(314)} \approx 0.53968 \text{ in} \)
  - \( d = 6 - 1.5 - \frac{1}{4} = 4.3 \text{ in} \)
  - \( f_y = 0.172 \geq \frac{1}{2} f_y = 0.014 \) (ok)
  - \( Mn = Asy(d - \frac{1}{4}) = (10.8)(40)(4.3 - 0.9932) \)
  - \( Mn = 1741.06 \text{ k-in} = 145.088 \text{ k-ft} \)
  - \( \phi Mn = 0.90(145.088) = 130.58 \text{ k-ft} \)

- **Negative Bending:**
  - \( Mn = 145.088 \text{ k-ft} \)
  - \( \phi Mn = 130.58 \text{ k-ft} \)

- **Negative Shear:**
  \( V_c = 2.4 \sqrt{f_e} \text{ bur d} \)
  \( = (2)(1.0)(3000)^{1/2}(314)(4.3) \)
  \( V_c = 14790.7 \text{ lbs} = 147.907 \text{ kips} \)
  \( \phi V_c = 0.75(147.907) = 110.93 \text{ kips} \)

  - **Negative Shear:**
    - \( V_c = 147.9 \text{ kips} \)
    - \( \phi V_c = 110.9 \text{ kips} \)
Deck Capacity (Existing Section)

- Positive Bending:
  1. \( A_3 = 38 \text{ in}^2 \)
  2. \( \alpha = \frac{A_3 f_y}{6} = 11.78 \text{ in}^2 \)
  3. \( d = 5.5 - 1.0 - 0.649 = 4.1875 \text{ in} \)
  4. \( e_y = 0.0234 \times 2 = 0.0468 \text{ in} \)

- Negative Bending:
  1. \( A_5 = 30 \text{ in}^2 \)
  2. \( \alpha = \frac{A_5 f_y}{6} = 6.0 \text{ in}^2 \)
  3. \( d = 5.5 - 1.5 - 0.649 = 3.75 \text{ in} \)

- Negative Shear:
  1. \( V_c = 2A \sqrt{f_y} \text{ b o d} \)
    \[ V_c = 2 \times (10)(2000)(4.45)(8.75) = 188,143 \text{ kips} \]
  2. \( \phi V_c = 0.75(188,143) = 141,107 \text{ kips} \)
Creep Block Capacity:

- **Shear Friction Capacity:** (ACI 318-08: see 11.6.4)
  \[ V_n = A_{vf} f_y \mu \]

  \[ A_{vf} = (3)(#6) + (4)(#5) \]
  \[ = (3)(0.31) + (4)(0.31) \]
  \[ A_{vf} = 9.34 \text{ in}^2 \]

- Creep Block was poured monolithically
  \[ \mu = 1.4 \]
  \[ \mu = 1.4 \]

  \[ V_n = A_{vf} f_y \mu \]
  \[ = (4.34)(40,000)(1.4) \]
  \[ V_n = 243,040 \text{ lbs} \]

- **Check:**
  1. \[ 0.2 A_c = 0.2(3000)(40)(3) = 288,000 \text{ lbs} > V_n \text{ (o.k.)} \]
  2. \[ (480 + 0.08 A_c)A_c = (480 + 0.08(3000))(40)(3) = 345,600 \text{ lbs} > V_n \text{ (o.k.)} \]
  3. \[ 1600 A_c = 1600(40)(3) = 768,000 > V_n \text{ (o.k.)} \]

  \[ V_n = 243,040 \text{ lbs} \]

**Creep Block Capacity:**

\[ V_n = 243,040 \text{ kips} \]
Beam Web Capacity:

- Punch Out Failure:
  - The web of the beam is analyzed as a flat slab with prestressing.
Beam Work Capacity:

- Punchout Failure: cont.

  Properties:

  Concrete:
  \[ f_{c'} = 6000 \text{ psi} \]
  \[ f_{c'} = 4000 \text{ psi} \]
  \[ f_{c'} = 0.456 f_{c'} = 2300 \text{ psi} \]
  \[ f_{c'} = 0.6 f_{c'} = 2900 \text{ psi} \]
  \[ f_{c'} = 1 \cdot f_{c'} = 924.5 \text{ psi} \]

- \[ V_c = (b_p \sqrt{f_{c'}} + 0.3 f_{c'}) \text{ bd} + V_p \]

- \[ \beta_p = \frac{a_1 d + 1.5}{b_0} \]
  - For corner column
  - \[ a_1 = 20 \]
  - \[ b_p = (20 \gamma d) + 1.5 \]

- \[ b_0 = (a + 12 + \frac{d f_{p,c}}{M}) (\gamma) + (d + 12 \frac{f_{p,c}}{M}) (\gamma) \]
  - \[ b_0 = 68 \text{ in} \]

- \[ \beta_p = \frac{(20 \gamma d)}{68} + 1.5 = 2.67 < 3.5 \]
  \[ \Rightarrow \beta_p = 2.67 \]

- \[ V_c = A_p \times f_{p,c} = (12 \times 0.153 \times 131265) \]
  - \[ V_c = 241921 \text{ lbs} \]

- \[ f_c = \frac{V_c}{A_c} = \frac{241921}{30.5(8)} = 991.478 \text{ psi} \]

- \[ V_c = (b_p \sqrt{f_{c'}} + 0.3 f_{c'}) \text{ bd} + V_p \]
  - Prestress is not bypassed: \[ \beta = 0 \]

- \[ V_c = (2.67)(6000)^{\frac{1}{2}} + 0.3(991.478)(68)(4) + 0 \]
  - \[ V_c = 137295 \text{ lbs} \]

- \[ V_n = 137.3 \text{ kips} \]

\[ \text{Punching Shear:} \]

\[ V_n = 137.3 \text{ kips} \]
Beam Web Capacity:

1. Shear Failure of Independent Surfaces:
   \[ \begin{align*}
   d_p &= 4 \text{ in} \\
   d_{p/2} &= 2 \text{ in}
   \end{align*} \]

   \[ \text{Horizontal Shear Plane:} \]
   \[ \text{* Not: No prestressing or stirrups provide shear resistance} \]
   \[ V_c = 2.1 \sqrt{f_y} \text{ bend} \]
   \[ = 2 \times (1.0)(6000) \sqrt{2}(8)(20) \]
   \[ V_c = 24,787 \text{ kips} \]

   \[ \text{Vertical Shear Plane:} \]
   \[ \begin{align*}
   A_p &= 6 \times 0.153 = 0.918 \text{ in}^2 \\
   A_c &= (14 \times 8) = 112 \text{ in}^2 \\
   I &= \frac{1}{12}bh^3 = 697.5 \text{ in}^4 \\
   c &= \frac{h}{2} = 4 \text{ in} \\
   s &= \frac{h}{6} = 149.3 \text{ in}^3 \\
   r^2 &= \frac{5h}{6} = 533 \text{ in}^3
   \end{align*} \]
   \[ \text{Mild Steel:} \]
   \[ f_y = 40 \text{ ksf} \]
   \[ A_c = 0.2 \text{ in}^2 \]

   \[ V_c = 0.60 \sqrt{f_y} \text{ bend } d_p + V_i + V_c(M_u) \]
   \[ \text{M}_{\text{max}} \]
   \[ = V_i = 0 \]
   \[ V_i = 2.0 F_k \text{ (Impact load at } d = 2.0) \]
   \[ F_k = \text{force from wave} \]
   \[ M_{\text{max}} = 12 (2.0F_k) = 24F_k \]
   \[ \text{lbf} \]
   \[ V_i = 0.60 \sqrt{f_y} \text{ bend } d_p + V_i(M_u) \]
   \[ \text{M}_{\text{max}} \]
   \[ = 0.60(1.0)(6000) \sqrt{2}(8)(4) + (230632) \]
   \[ 12 \]
   \[ V_c = 2182 \text{ lbs} = 218 \text{ kips} \]

2. Check:
   \[ 1.7 \sqrt{f_y} \text{ bend } d_p = 7372 \text{ lbs} < V_c \text{ (ok)} \]
   \[ 5.0 \sqrt{f_y} \text{ bend } d_p = 21688 \text{ lbs} < V_c \text{ (ok)} \]
   \[ \Rightarrow V_c = 21688 \text{ lbs} \Rightarrow 216 \text{ kips} \]
Beam Web Capacity:
- Vertical Shear Plane: cont.
  - $V_{uw} = (3.5A_{te} + 0.3f_{c}bvd_p + V_p$

  \[ f_c = \frac{P_e}{A_e} = \frac{120,960}{112} = 1080 \text{ psi} \]

  \[ V_{uw} = (3.5 \times 112 + 0.3 \times 1080) bvd_p + V_p \]

  \[ = (3.5 \times (6000)^{1/2} + 0.3 \times (6000)^{1/2}) bvd_p + V_p \]

  \[ V_{uw} = 35,353 \text{ lbs} \]

  \[ V_{uw} > V_c \]

  \[ V_c = V_c = 21,689 \text{ kips} \]

  - Stirrups:
    - #4 stirrups @ 6" o.c.
    - $V_s = \frac{A \times f_y \times d_p}{S} = \frac{0.20 \times 40,000 \times 4}{6}$
    - $V_s = 5333.3 \text{ lbs} = 5.33 \text{ kips}$

  - Total Shear Capacity:
    - $V_n = H_{horiz} + V_{vertical}$
      - $= (24.787) + (21.689) + 5.33$
      - $V_n = 51.8093 \text{ kips}$

    \[ V_n = 51.81 \text{ kips} \]

Web Shear Capacity:
- $V_n = 51.81 \text{ kips}$

Independent failure surfaces.
Beam Web Capacity:

- Independent Failure Surfaces:
  - Vertical Plane Bending: (Flexural Capacity)

Assume $f_{pu} = 230,000$ psi; $f'_c = 6000$ psi;

\[ a = \frac{A_p}{b} \]

\[ d_p = \frac{f_{pu}}{f''} \left(1 - \frac{c_p}{f''} \right) \]

For low-reinforcement strands
\[ \psi_p = 0.28 \]
For 4000 $\leq f'_c \leq 8000$
\[ b_3 = 0.85 - 0.05 \left( \frac{f'_c - 4000}{1000} \right) \]

\[ b_3 = 0.75 \]

\[ \psi = 0 \]
\[ \psi = 0 \]

\[ \Rightarrow d_p = \frac{f_{pu}}{f''} \left(1 - \frac{c_p}{f''} \right) \]

\[ \Rightarrow d_p = 195.64 \text{ kips} \]

\[ a = \frac{A_p}{b} \frac{f''}{c_p} = \frac{(0.98)(195.64)}{0.85 \times 6} \]

\[ a = 2.5154: in \]

\[ M_n = A_p f'' (d_p - \frac{1}{2}) \]

\[ M_n = (0.98)(195.64)(4 - 2.5154) \]

\[ M_n = 422.516 \text{ kips} \]

\[ M_n = 41.043 \text{ kips} \]

Force from creep block will act $(6 + \frac{1}{2}) = 8$ in away from failure surface:

\[ V = \frac{M_n}{(g/\alpha)} = \frac{41.043}{(g/\alpha)} \]

\[ V = 61.56 \text{ kips} \]

Total Capacity = Horizontal Shear + Vertical Flexure

\[ V_n = 86.3514 \text{ kips} \]

Horizontal Shear
Vertical Flexure
**Beam Web Capacity:**

- **For Non-prefeured Slab:** (ACI 11.11.1.1)
  
  *Note: since prestressing will provide reinforcement a distance away from the edge of the beam, the beam web is analyzed as a slab with no prestressing.

  ⇒ Possible capacity

\[ \frac{dp \leq 4.1}{dp/2 = 2.1} \]

**Example Calculation:**

1. \( V_c = \left(2 + \frac{a}{b_d}\right) \lambda \sqrt{f_{cd}} \cdot b_o \cdot d \)

   \( \frac{b_d}{h} = \frac{1}{2} \)

   \( b_o = 2(2+1) + 2(a+1+2) = 68 \text{ in} \)

   \( \frac{V_c = \left(2 + \frac{a}{b_d}\right) \lambda \sqrt{f_{cd}} \cdot b_o \cdot d}{V_c = 12.64 \text{ kips}} \)

2. \( V_c = \left\{ \left( \frac{a}{b_d} \right) + 2 \right\} \lambda \sqrt{f_{cd}} \cdot b_o \cdot d \)

   \( \frac{\alpha_s = 2.0}{(comarw)} \)

   \( \frac{V_c = \left( \frac{a}{b_d} + 2 \right) \lambda \sqrt{f_{cd}} \cdot b_o \cdot d}{V_c = 66.925.2 \text{ kips}} \)

3. \( V_c = 4 \lambda \sqrt{f_{cd}} \cdot b_o \cdot d \)

   \( = 4(1.0)(6000)^{\frac{1}{2}} (68 \times 4) \)

   \( V_c = 84.276.1 \text{ kips} \)

\[ \frac{V_c = 66.93 \text{ kips}}{\text{Comarw}} \]

Nonprefeured Slab Analysis Capacity
Bean Web Capacity:

- Flexure Capacity at Top of Web:

- Euler Length = 36.5 in
  \( \Rightarrow \) 6 \# 4 struts

**Note:** Only the top will provide flexural resistance

- \( f_c = 6000 \) psi
- \( f_y = 40,000 \) psi

\[ A_k = 6(\frac{20}{2}) = 1.2 \text{ in}^3 \]

\[ a = A_k \frac{f_y}{f_c} = (1.2)(40) \]
\[ 0.85 \times 0.85 (a \times 36.5) \]
\[ a = 0.297857 \text{ in} \]

- \( d = q - 2 = 6 \text{ in} \)

- \( \beta_1 = 0.85 - 0.05 \left( \frac{f_c - 4000}{1000} \right) \)
  \( \beta_1 = 0.75 \) for \( 4000 \leq f_c \leq 8000 \) psi

- \( \varepsilon_3 = 0.0494 \geq \varepsilon_2 = 0.001 \) (o.k.)

- \( M_n = A_k \frac{f_y (d - 0.5)}{2} \)
  \( = (1.2)(40)(6 - 0.297857) \)
  \[ M_n = 281.811 \text{ k-in} \]

- Force will act = 6418.5 = 24.5 in away from flexure

\[ V_n = \frac{M_n}{24.5} = \frac{281.811}{24.5} \]
\[ V_n = 11.502 \text{ kips} \]

**Flexure Capacity at Top of Web**
Summary of Results:

- Creep Block (Shear Friction) = 243.04 kips
- Beam Web Prestressed Slab Punchout Failure = 137.3 kips
- Beam Web Independent Shear Surfaces = 50.0 kips
- Beam Web Independent Shear - Flexural Surfaces = 86.3 kips
- Beam Web (Nonprestressed) Shear Failure = 66.93 kips
- Beam Web Flexural Capacity = 11,502 kips 

<table>
<thead>
<tr>
<th>Total Capacity For Bridge = 8 (11,502) = 92,016 kips</th>
</tr>
</thead>
</table>

(8 total creep blocks)
Negative Bending Capacity:

Properties:
- $f_e' = 6000$ psi
- $A_{ps} = (2)(4.875)(3)(0.0367)$ in
- $f_e = 2(22680)$ kips
- $f_e = 12 \sqrt{f_e'} = 129.516$ psi
- $f_e = -0.85 f_e' = -5100$ psi

Compute Geometric Properties:
- Centroid: (see buoyancy calculations for area)
  - $A_1 = 0.1667 \text{ ft}^2$
  - $y_1 = 96.28 \text{ in}$
  - $A_2 = 0.2367 \text{ ft}^2$
  - $y_2 = 34.76 \text{ in}$
  - $A_3 = 1.1667 \text{ ft}^2$
  - $y_3 = 35.25 \text{ in}$
  - $A_4 = 0.111 \text{ ft}^2$
  - $y_4 = 34.166 \text{ in}$
  - $A_5 = 1.8661 \text{ ft}^2$
  - $y_5 = 16.75 \text{ in}$
  - $A_6 = 0.094 \text{ ft}^2$
  - $y_6 = 33.5 \text{ in}$

- Moment of inertia:
  - Iap: $I = \frac{1}{3} bh^3$
  - $I = \frac{1}{6} (45.75)(2.5)^3$
  - $I = 61.5703 \text{ in}^4$
  - $I + A_{ps}^2 = 61.5703 + (45.75)(2.5)(16.1667 - 1.25)^2$
  - $I = 19.07856 \text{ in}^4$

- Center Rectangle:
  - $I = \frac{1}{12} bh^3$
  - $I = \frac{1}{12} (4.875)(3.6)^3$
  - $I = 11916.1 \text{ in}^4$
  - $I = 19.07856 \text{ in}^4$

- Rombi triangles:
  - $I = \frac{1}{2} b^{2} h = \frac{1}{2} (12)(3.6) = 1.494 \text{ in}^4$
  - $I = A_{ps}^2 + (4.875)(3)(10.637)\times(7.528)$
  - $I = 74.139 \text{ in}^4$

- Small Triangles:
  - $I = \frac{1}{2} b^{2} h = \frac{1}{2} (2.575)(3) = 2.1562 \text{ in}^4$
  - $I = A_{ps}^2 + (2.575)(3)(10.637)\times(7.528)$
  - $I = 474.69 \text{ in}^4$

$I_C = 190785.6 + 29585.6$
- $(2)(4.875)(3)(0.0367)$
+ $(2)(11916.1)$
$I_C = 47469.5 \text{ in}^4$
**Negative Bending Capacity**

**Geometric Properties:**
- $A_c = 374.4 \text{ in}^2$
- $I_c = 49469.6 \text{ in}^4$
- $C_c = 14.136 \text{ in}$
- $S_t = 34477.37 \text{ in}^3$
- $C_b = 21.863 \text{ in}$
- $S_b = 22.674 \text{ in}^3$
- $r^2 = 132.13 \text{ in}^2$
- $C_c = 9.363 \text{ in}$
- $L = 50 \text{ ft} = 600 \text{ in}$

- $w_0 = \left( \frac{\text{psf}\text{-topping} + \text{Self weight}}{12} \right) = \left( (7.5\%)(40.93/\ell) + 2.60 \right) \times 150 = 247.42 \text{ lb/ft}$
- $w_0 = 62.285 \text{ lb/ft} \text{ (unsubmarged)}$

- Tensile Limit: (unsubmerged)
  \[ w_{te}=8\left(\frac{40L^5}{EI_c}\right) \]
  \[ M_0 = w_0 L^4/8 = (62.285\times 600)^4/8 = 2.80383\times 10^4 \text{ lb}\cdot\text{in} \]
  \[ w_{te} = \frac{8}{600^5}\left(2.80383\times 10^4 + (34477.37\times 9.1596) - (247.42\times 2.1161) - (9.36\times 14.136) \right) \]
  \[ w_{te} = 134.506 \text{ lb/ft} \]

  \[ F_{te} = 80.703 \text{ kips (tensile limit)} \]

- Compression Limit: (unsubmerged)
  \[ w_{ce}=6\left(\frac{L^4}{2EIC_c}\right) \]
  \[ w_{ce} = 246.25 \text{ lb/ft} \]

  \[ F_{ce} = 147.75 \text{ kips (compression limit)} \]

- Tensile Limit: (submerged)
  \[ w_D = \left( \frac{7.5\%}{12} \right)(45.95/\ell) + 2.60 \times 150 = 428.522 \text{ lb/ft} \]
  \[ w_D = 35.71 \text{ lb/ft} \]

  \[ M_0 = 1.60496\times 10^4 \text{ lb}\cdot\text{in} \]

  \[ w_{te} = 107.93 \text{ lb/ft} \]

  \[ F_{te} = 64.76 \text{ kips (tensile limit)} \]
**Negative Bending Capacity:**

(Compression Limit: (submerged))

\[ \sigma_{wc} = 219.677 \text{ ksi} \]

\( F_{cr} = 131.8 \text{ kips} \) (Compression limit)

**Summary of Results: (12 girders)**

- **Unsubmerged:**
  - Tensile: \( F_{t} = 80.703 \text{ kips} \)
  - Compression: \( F_{c} = 147.75 \text{ kips} \)

- **Submerged:**
  - Tensile: \( F_{t} = 64.96 \text{ kips} \)
  - Compression: \( F_{c} = 131.8 \text{ kips} \)

- **Loss after submerged:**
  - Tensile: 19.76 %
  - Compression: 10.80 %

A-118
Buoyancy Calculations:

- **Self Weight:**

  From Sheet 1

<table>
<thead>
<tr>
<th>Item</th>
<th>Superstructure</th>
<th>Railing &amp; Fast. Post</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>(γ = 146 psf) Class A Concrete</td>
<td>248.01 cu yd</td>
<td>28.90 cu yd</td>
<td>776.11 cu yd</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(20476 cu ft)</td>
</tr>
<tr>
<td>Reinfl. Steel</td>
<td>150,878 lbs</td>
<td>5183 lbs</td>
<td>-NA-</td>
</tr>
<tr>
<td>(γ = 152 psi) 1&quot; AC Pavemnt</td>
<td></td>
<td></td>
<td>156056 lbs</td>
</tr>
</tbody>
</table>

  Total Weight = 3.04161 x 10^6 lbs

  Self Weight = 3.33816 x 10^6 lbs

\[ \text{Self Weight} = 3.338 \text{ kips} \]

- **Air Pockets Calculation:**
  - **Between Girder:**

\[ 
A_1 = \left( \frac{22}{2} \times \frac{22}{2} \times \left( \frac{11}{10} \right) \right) = 11 \text{ ft}^2 \\
A_2 = 2 \left[ \frac{1}{2} \left( \frac{9}{10} \times \frac{9}{10} \right) \right] = 0.25 \text{ ft}^2 \\
A_T = A_1 - A_2 = 10.75 \text{ ft}^2 \\
A_T = 10.75 \text{ ft}^2 \\
\]

Air Pocket Between Girder
Amount = 3
Length = 215 ft
Air Pocket Calculation:

- Between Deck and Pier Cap Reinforcement:

\[ A_1 = 2 \left( \frac{1}{2} \times 20 \times 4 \times \frac{3}{2} \right) = 11.5625 \text{ ft}^2 \]

\[ A_2 = (1 + \frac{3}{8}) \times 4.625 \times \frac{3}{8} = 0.578125 \text{ ft}^2 \]

\[ A_3 = (6.5 \times \frac{3}{8}) = 15.375 \text{ ft}^2 \]

\[ A_T = A_1 + A_2 + A_3 = 27.5156 \text{ ft}^2 \]

Air Between Girder & Pier Cap
Amount = 1
Length = 215 ft

Compression of Air Pocket:

- \[ h = (1 + \frac{3}{8}) + 30 + 1 = 38.625 \text{ in} = 3.21875 \text{ ft} \]

- \[ P_1 = P_1 + h_s \left( \frac{49}{44} \right) = 14.2 + (3.21875) \times \frac{49}{44} \]
  \[ P_e = 16.1306 \text{ psi} \]

- Total Air Pocket = \( 6 \times 10.75 + 27.5156 = 113.516 \text{ ft}^2 \)

- \[ A_3 = \frac{P_1 A_1}{P_e} = \frac{14.2 \times 113.516}{16.1306} \]
  \[ A_2 = 103.449 \text{ ft}^3 \]

Air Pocket
Amount = 1
Length = 215 ft

\[ A_T = 103.456 \text{ ft}^3 \]
**Buoyant Force:** \((W_b = 64 \text{ ft}^3)\)

* Assume the bridge is submerged to the top of the deck

- Submerged Volume = Air Pocket + Concrete + Ac
  \[ V = (102.45 \times 2.15) + 748.01 (27) + (140.50/16) \]
  \[ V = 41362.1 \text{ ft}^3 \]

- Buoyant Force = 43362.1 (64)
  \[ BF = 2.77517 \times 10^6 \text{ lbs} \]

- Residual Weight = 3.35816 \times 10^6 - 2.77517 \times 10^6
  \[ RW = 582987 \text{ lbs} \]
  \[ \% \text{ Retained} = 16.8652 \% \]

**Summary of Results:**

- Self Weight = 3338 kips
- Buoyant Force = 2775 kips
- Residual Weight = 563.0 kips
- \( \% \text{ Retained} = 16.9 \% \)

\[ \therefore \text{Bridge is NOT Buoyant} \]
**Deck Capacity:**

\[ f_y = 40,000 \text{ psi} \]
\[ f_c = 3,000 \text{ psi} \] (Class A concrete)

Top: 3 #4 bars
Bottom: 5 #5 bars

**Positive Bending:**

1. \( A_2 = 3(0.4) = 1.2 \text{ in}^2 \)
2. \( a = A_2 f_y = (1.2)(40) \)
\[ 0.05 \times 6.5 \times 0.85 \times (3)(72) \]
\[ a = 0.33749 \text{ in} \]
3. \( d = 8 - 1.5 - \frac{3}{16}(0.635) = 6.1875 \text{ in} \)
4. \( E_3 = 0.0437 \geq E_f = 0.064 \) (ok)

**Negative Bending:**

1. \( A_2 = 3(0.4) = 0.6 \text{ in}^2 \)
2. \( a = A_2 f_y = (0.6)(40) \)
\[ 0.05 \times 6.5 \times 0.85 \times (3)(72) \]
\[ a = 0.130719 \text{ in} \]
3. \( d = 8 - 2 - (X)(0.6) = 5.75 \text{ in} \)
4. \( E_3 = 0.109 \geq E_f = 0.064 \) (ok)

**Negative Shear:**

1. \( V_c = 2h \sqrt{f_c} \times b \times d \)
\[ = (2)(6)(3000)(0.75)(12) \]
\[ V_c = 45351.4 \text{ lbs} \]
2. \( V_s = A_2 f_y d = 1.2(40)(3000)(0.75) \)
\[ 12 \]
\[ V_s = 11833.3 \text{ lbs} \]
3. \( V_{n1} = V_c + V_s = 57,348 \text{ kips} \)
\[ \phi V_{n1} = 0.75 (57348) = 42,926 \text{ kips} \]
Lateral Resistance:

- **Dowels:**
  - *Note:* 1" of Dowels 4'-0" long (Mark 804)
  - are only provided at fixed abutments/pads only (Piers No. 2, 3, 4, 5, 6)
  - $f_y = 40,000$ psi
  - $f_y' = 3,000$ psi
  - 1" of Dowels 4'-0" long

- **Shear Friction (All Section 11.6.4):**
  - $V_n = \text{Area} \times f_y' \times \lambda$
  - $\lambda = (2)(0.29) = 0.58$ in
  - Concrete placed against hardened concrete (placed monolithically)
  - $\lambda = 1.4 \times 1 = 1.4$
  - $V_n = \text{Area} \times f_y' \times \lambda$
  - $= (1.55)(40,000)(1.4)$
  - $V_n = 88,480$ lbs = 88.48 kips

  - For single span: (span 3, 4, 5 & 6)
    - 9 girders = 18 total shear friction
    - $V_n = (18)(88.48) = 1,592.64$ kips

- **Self Weight:** (Single Span)
  - $SW = (3.3381\times10^4)(\frac{3}{8}) = 417.70$ lbs
  - $SW = 417.70$ kips

- **Total Lateral Resistance:** (Single Span)
  - Span 3, 4, 5 & 6 = 1926.46 kips
  - Span 2 & 7 = 1130.14 kips (More likely to fail first)
- **Vertical Resistance**
  - **Pier Cap Weight:**
    *Note: The dowels will secure the bridge deck to the pier cap.*
    *The weight of the pier cap will add vertical resistance.*
  - See Sheet No. 3 Moanalua Bridge.
    - **Pier Cap Dimensions:**
      - \( h = 3' \)
      - \( \text{width} = 2' - 8' \)
      - \( \text{length} = 66" \)
    - \( V = (3)(x + \frac{9}{12})(66) = 528\text{ ft}^2 \)
    - \( W = V \times Y_{RC} = (528)(180) \text{ lb} = 79200 \text{ lbs} = 79.2 \text{ kips} \)
  - **Total Vertical Resistance:** (spans 3, 4, 5, 6)
    - \( VR = \text{Self Weight} + 2 \text{ (pier caps)} \)
    - \( = (412.27) + (2)(79.2) \)
    - \( VR = 575.67 \text{ kips} \)

- **Summary:** (Single Spans 3, 4, 5, 6)
  - Lateral Resistance = 1926.46 kips
  - Vertical Resistance = 575.67 kips
Buoyancy Calculations:

**Self Weight:**

From Sheet 1

<table>
<thead>
<tr>
<th>Item</th>
<th>Superstructure</th>
<th>Railings &amp; End Fast.</th>
<th>Total</th>
<th>Total Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class A Concrete</td>
<td>885.83 CY</td>
<td>25.63 CY</td>
<td>911.46 CY</td>
<td>3.56337 x 10^8 lbs</td>
</tr>
<tr>
<td>(g= 146 lb CY)</td>
<td></td>
<td></td>
<td>(2460.4 ft^3)</td>
<td></td>
</tr>
<tr>
<td>Rein. Steel</td>
<td>198,043 lbs</td>
<td>4550 lbs</td>
<td>-NA-</td>
<td>202,603 lbs</td>
</tr>
<tr>
<td>1&quot; AC Pavement</td>
<td></td>
<td></td>
<td>-NA-</td>
<td>89.8 tons</td>
</tr>
<tr>
<td>(g= 122 lb CY)</td>
<td></td>
<td></td>
<td></td>
<td>(179,600 lbs)</td>
</tr>
<tr>
<td>AC Roll</td>
<td></td>
<td></td>
<td>-NA-</td>
<td>2.5 tons</td>
</tr>
<tr>
<td>(g= 151 lb CY)</td>
<td></td>
<td></td>
<td></td>
<td>(50,000 lbs)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>Total = 3,95561 x 10^6 lbs</strong></td>
</tr>
</tbody>
</table>

Self Weight = 3,95561 x 10^6 lbs

Self Weight = 3955.6 kips

---

Air Pocket Calculation:

![Diagram of air pocket](attachment:diagram.png)

\[ A_1 = (0.5)(48.5) = 10.83 \text{ ft}^2 \]

\[ A_x = 2 \left[ \frac{1}{2} \left( \frac{1}{2} \times 15 \right) \right] = 0.25 \text{ ft}^2 \]

\[ A_t = A_1 - A_x = 9.83 \text{ ft}^2 \]

Air Pocket Between Girders
Amount = 12
Length = 188 ft

A-125
Air Pocket Calculations: cont.

- Compression of Air Pocket:
  - Assume bridge is submerged to the top of deck
  - \( h = (30) + 1 = 31 \text{ in} = 2.583 \text{ ft} \)
  - \( P_2 = P_1 + h \left( \frac{64}{1000} \right) \)
    \[ = 14.7 + (2.583) \left( \frac{64}{1000} \right) \]
    \[ P_2 = 15.8481 \text{ psi} \]
  - \( A_2 = P_2 A_1 \left( \frac{14.7}{9.83} \right) \)
    \[ P_2 = (15.8481) \]
    \[ A_2 = 9.121 \text{ ft}^2 \]

\[ \boxed{A_2 = 9.121 \text{ ft}^2} \text{ Compressed Air} \]

Buoyant Force: \( \delta_w = 64 \frac{1}{1000} \)

- Submerged Volume = Air + Concrete + Ac
  \[ = (9.121)(8)(100) + (885.83 \times 27) + (174.60 \times 15) \]
  \[ SV = 38816.9 \text{ ft}^3 \]
- Buoyant Force = \( SV \delta_w \times 64 \)
  \[ BF = 2.48428 \times 10^6 \text{ lbs} \]
- Residual Weight = 3.9556 \times 10^6 - 2.48428 \times 10^6
  \[ RW = 1.47133 \times 10^6 \text{ lbs} \]
  \[ \% \text{ Retained} = 37.196 \% \]

Summary of Results:

- Self Weight = 3956 kips
- Buoyant Force = 2484 kips
- Residual Weight = 1471 kips
- \( \% \text{ Retained} = 37.2 \% \)

\[ \therefore \text{ Bridge is NOT Buoyant} \]
**Deck Capacity:**

\[ \frac{1}{8}'' \text{ M4900} \]

\[ 8'' \]

\[ \frac{3}{8}'' \text{ M5600 @ 12'' o.c.} \]

\[ 16'' \]

- **Positive Bending:**
  - \( A_5 = 1.55 \text{ in}^2 \)
  - \( a = A_{fy} = (155)(40) \)
    - 0.47 k"/b
    - 0.85 (3X66)
    - \( a = 0.36839 \text{ in} \)
  - \( d = 6.1875 \text{ in} \)
  - \( E_5 > E_Y \) (ok.)

  \[ M_n = A_{fy}(d - \frac{a}{2}) = (1.55)(40)(0.1875 - 0.36839/2) \]
  \[ M_n = 372.205 \text{ k-in} = 31.0171 \text{ k-ft} \]
  \[ \varphi M_n = 27.954 \text{ k-ft} \]

- **Negative Bending:**
  - \( A_5 = 0.59 \text{ in}^2 \)
  - \( a = A_{fy} = (0.60)(40) \)
    - 0.85 k"/b
    - 0.475 (3X66)
    - \( a = 0.11826 \text{ in} \)
  - \( d = 5.75 \text{ in} \)
  - \( E_5 \geq E_Y \) (OK)

  \[ M_n = A_{fy}(d - \frac{a}{2}) = (0.60)(40)(5.75 - 0.11826/2) \]
  \[ M_n = 186.289 \text{ k-in} = 15.5274 \text{ k-ft} \]
  \[ \varphi M_n = 10.22 \text{ k-ft} \]

- **Negative Shear:**
  - \( V_c = 2.1 \sqrt{w} \text{ kip} \)
    - \( = (2)(10)(3000)(0.75/66)(6.75) \)
    - \( V_c = 41572.1 \text{ lbs} \)
  - \( V_s = 11883.3 \text{ lbs} \) (same as Moonelum Bridge)
  - \( V_n = V_c + V_s = 53455.4 \text{ lbs} = 53.46 \text{ kips} \)
    - \( \varphi V_n = 40.09 \text{ kips} \)

- **Positive Shear:**
  - \( \varphi V_n = 53.46 \text{ kips} \)
    - \( \varphi V_n = 40.09 \text{ kips} \)

\[ \text{A-127} \]
Lateral Resistance

* Note: Piers 3, 4 & 6 have 3 girders secured to the pier caps with doublers. There are 13 girders. The doublers will only provide some lateral resistance but no vertical resistance.

The force will lift causing a moment at the fixed end and pulling the doublers.

Dowels:

\[ 2 \times 1" \text{ of dowels 4' long} \]

 Mk 804

Shear Section:

1. \[ V_n = A_f \cdot f_y \cdot t \]

2. \[ A_f = (2 \times 0.74) = 1.58 \text{ in}^2 \]

3. \[ t = 1.4 \times i = 1.4 \] (concrete placed monolithically)

4. \[ V_n = A_f \cdot f_y \cdot t \]
   \[ = (1.58)(40)(1.4) \]
   \[ V_n = 88.48 \text{ kips} \]

For typical span:

3 girders restrained by doublers \( \Rightarrow \) 6 total

\[ V_n = 6 \times 88.48 = 530.88 \text{ kips} \]

Self Weight: (Single Span)

\[ SW = (3.99561 \times 10^6)/7 = 565087.115 \text{ lbs} = 56.5 \text{ kips} \]

* Girder sit on 4 layers of 30\# roofing paper:
  \[ \mu = 0.5 \]
  Friction = (56.5)(0.5) = 28.2544 kips

Total Lateral Resistance: (Typical Span)

\[ TLR = (530.88) + 282.544 = 813.424 \text{ kips} \]
Vertical Resistance:

Dowels will not provide additional vertical resistance.

Total Vertical Resistance = Self Weight = 565.087 kips.

Summary: (Typical Single Span)

- Lateral Resistance = 813.424 kips
- Vertical Resistance = 565.087 kips
**Negative Bending Capacity**:

\[ \alpha = \frac{A}{b' h} = \frac{0.8 \times 40}{0.85 \times 6'} \times 5 \times (3)(10) \]
\[ \alpha = 1.39459 \text{ in} \]

- \( d = 28 \text{ in} \)
- \( \beta_1 = 0.85 \)
- \( \varepsilon_1 = 0.509 \geq 0.0014 \) (ok)

\[ M_n = \frac{A f_y (d - \beta_1)}{12} = \frac{(0.8 \times 40)(28 - 0.85)}{12} \]
\[ M_n = 893.769 \text{ k-in} \]
\[ M_n = 893769 \text{ lb-in} \]

- **Distributed Load**:

\[ M = \frac{4w \ell^3}{9} \]
\[ \Rightarrow \ w = 68.1123 \text{ lb/in} \]
\[ F_w = 22.0684 \text{ kips} \]

- **Negative Bending Capacity**:

\[ w_{nf} = 68.1123 \text{ lb/in} \]
\[ F_{nf} = 22.02 \text{ kips} \]
Nimitz Highway: Slip #2

**Hydrology Calculations:**

**Self Weight:**

- Deck
- Bent

- Front Elevation (Sheet 10)

- Layout Plan (Sheet 10)

- Concrete: (From Sheet 1)

  Volume superstructure and railing = \(494 + 12 = 511 \text{ yd}^3 = 13747 \text{ ft}^3\)

- Reinforcing Steel: (From Sheet 1)

  Weight = (Deck and Framing) + (Railing and Rail Post)

  = (66,368) + (2,520)

  Weight = 68888 lbs

- Total Self Weight: (Concrete = 145 \(\frac{19}{64}\) )

  Self Weight = Concrete + Steel

  = \(13747 \times 145 + 68888\)

  Self Weight = 2069453 lbs

- Self Weight = 2069.45 kips
**Air Pocket Calculations:** (Sheet No. 12)

- **Typical Cap Reinforcing:**

- **Concrete Cap Volume:** (Intermediate Span)

\[ V = (2 + 1/3)(2 + 1/3)(2 + 5/12) = 20.0614 \text{ ft}^3 \]

\[ V = 20.07 \text{ ft}^3 \]

*Note:* The bent caps are solid beams that run longitudinally along the slip cover. No air will become trapped under the bridge, because the front of the slip cover is open, thus allowing any air to escape. (i.e. Air Pocket = 0)

- **Buoyant Force:** (Use water = 64 lbf/ft³)
  - **Submerged Volume = Submerged Concrete + air pocket**
    \[ SV = 499 \text{ cy} = 13473 \text{ ft}^3 \]
  - **Buoyant Force:**
    \[ BF = SV \times \gamma_w \]
    \[ BF = (13473)(64) \]
    \[ BF = 862,272 \text{ lbs} \]

*Note:* Assume the bridge is submerged to the top of the deck.

- **Residual Weight:** (Once submerged)

\[ \text{Residual Weight} = SW - BF \]

\[ RW = 2069453 - 862272 \]

\[ RW = 1207181 \text{ lbs} \]
Summary of Results:

- Self Weight = 2069.5 kips
- Buoyant Force = 862.3 kips
- Residual Weight = 1207.2 kips
- % Retained Weight = 58.3%

\[ \text{Bridge is NOT Buoyant} \]
Deck Capacity:

\[ f_y = 40,000 \text{ psi} \]
\[ f_{cu} = 3500 \text{ psi} \]

*Note: No. Spans = 10
Max Length = 67 ft
Min Length = 63 ft

14" - 1/2" Ø #4 400 Top
(3# bars)

15 - 3/8" Ø #5 500 Shot
(8# bars)

Positive Bending:

IV. \[ A_0 = (15)(0.31) = 4.65 \text{ in}^2 \]

\[ a = \frac{A_0 f_y}{0.85 f_{cu}} = \frac{4.65}{0.85} = 5.49 \text{ in} \]
\[ a = 0.3035 \text{ in} \]

\[ d = 14 - 2 -(\frac{1}{4})(0.45) = 11.6875 \text{ in} \]
\[ E_s = 0.0952 \geq E_s = 0.0014 \text{ (ok)} \]

\[ M_n = A_0 f_y (d - \frac{a}{2}) \]
\[ = (4.65)(40)(116875 - 0.3035) \]
\[ M_n = 1246.65 \text{ k-in} = 174.8 \text{ k-ft} \]
\[ \phi M_n = 0.9 (174.8) = 167.4 \text{ k-ft} \]

Positive Bending:

\[ M_n = 174.8 \text{ k-ft} \]
\[ \phi M_n = 167.4 \text{ k-ft} \]

Negative Bending:

\[ A_0 = (14)(0.20) = 2.8 \text{ in}^2 \]

\[ a = \frac{A_0 f_y}{0.85 f_{cu}} = \frac{2.8}{0.85} = 3.338 \text{ in} \]
\[ a = 0.1827 \text{ in} \]

\[ d = 14 - 2 -(\frac{1}{2})(0.5) = 11.75 \text{ in} \]
\[ E_s = 0.161 > E_s = 0.0014 \text{ (ok)} \]

\[ M_n = A_0 f_y (d - \frac{a}{2}) \]
\[ = (2.8)(40)(116875 - 0.1827) \]
\[ M_n = 1805.77 \text{ k-in} = 254.5 \text{ k-ft} \]
\[ \phi M_n = 0.90 (254.5) = 229.0 \text{ k-ft} \]

Negative Bending:

\[ M_n = 229.0 \text{ k-ft} \]
\[ \phi M_n = 229.0 \text{ k-ft} \]

Negative Shear:

\[ V_c = 2.5 \sqrt{f_y} \]
\[ = 2.5(40)(3500)^{1/4}(206)(11.75) \]
\[ V_c = 286397 \text{ kips} \]

\[ \phi V_c = 0.78 (286397) = 224.798 \text{ kips} \]

Negative Shear:

\[ V_n = 286.4 \text{ kips} \]
\[ \phi V_n = 214.8 \text{ kips} \]
**Beam Cap Capacity:**

![Beam Diagram]

- **Positive Bending:**
  1. \( A_s = 4 \times (0.74) = 2.96 \text{ in}^2 \)
  2. \( \sigma = \frac{M_s}{I_b} = \frac{3.16 \times 40}{0.85 \times 6^3} \) \( \times \frac{0.85 \times 3.5 \times 34}{3.5^3} \) \( \sigma = 1,249.63 \text{ in} \)
  3. \( d = 30 - 4 - 1.5 = 24.5 \text{ in} \)
  4. \( \varepsilon_s = 0.0472 \) \( \varepsilon_y = 0.001 \) \( (0, k) \)

- **Negative Bending:**
  1. \( A_s = 4 \times (0.74) = 3.16 \text{ in}^2 \)
  2. \( \sigma = 1,249.63 \text{ in} \)
  3. \( d = 30 - 3 = 27 \text{ in} \)
  4. \( \varepsilon_s \geq \varepsilon_y \) \( (0, k) \)

- **Shear:**
  1. \( V_c = 2.11 \sqrt{f_e} \text{ in. burhod} \)
  2. \( V_c = 108.619 \text{ lb} \)
  3. \( V_y = 0.20 \times 40,000 \times 19 \)
  4. \( V_y = 154,286 \text{ lb} \)

- **Moments:**
  1. \( M_n = A_s \varepsilon_y (d - \varepsilon_y) \)
  2. \( M_n = 301.782 \text{ k-in} = 251.385 \text{ k-ft} \)

- **Positive Bending:**
  1. \( M_n = 251.385 \text{ k-ft} \)
  2. \( \phi M_n = 226.337 \text{ k-ft} \)

- **Negative Bending:**
  1. \( M_n = 277.819 \text{ k-ft} \)
  2. \( \phi M_n = 250.037 \text{ k-ft} \)

- **Shear:**
  1. \( V_n = V_y + V_s \)
  2. \( V_n = 124,048 \text{ lb} = 124,048 \text{ kips} \)

- **Shear:**
  1. \( V_n = 93.0357 \text{ kips} \)
  2. \( \phi V_n = 93.04 \text{ kips} \)
Buoyancy Calculations:

**A. Self-Weight:**

- **Concrete:** (From sheet 1)
  
  \[
  \text{Volume} = (\text{Deck} + \text{Framing}) + (\text{Monolithic Sidewalk}) + \text{Railing} \\
  = (321) + (1) + (15) \\
  \text{Volume} = 537 \text{ yd}^3 = 14,499 \text{ ft}^3
  \]

- **Reinforcing Steel:**
  
  \[
  \text{Weight} = (\text{Deck} + \text{Framing}) + \text{Railing} \\
  = 74,631 + 3,037 \\
  \text{Weight} = 77,668 \text{ lbs}
  \]

- **Total Self-Weight:** (Concrete = 145 lb/ft³)
  
  \[
  \text{Self Weight} = \text{Concrete} + \text{Steel} \\
  = (14,499)(145) + (77,668) \\
  \text{Self Weight} = 2,180,023 \text{ lbs}
  \]

  \[
  \text{Total Self Weight} = 2,180.0 \text{ kips}
  \]
**Buoyant Force:** (\(Y_{sea\text{water}} = 64\) lb/ft²)

*Note: No air will be trapped under the bridge deck as the front (facing the harbor) is open to air, allowing air to escape.*

\[ \text{Air Pocket} = 0 \]

- Submerged Volume = Submerged Concrete + Air Pocket
  \[ SV = (521 \times 27) + 0 \]
  \[ SV = 14067 \text{ ft}^3 \]

- Buoyant Force:
  \[ BF = SV \times Y_{sw} \]
  \[ BF = (14067 \times 64) \]
  \[ BF = 900288 \text{ lbs} \]

**Residual Weight:**

\[ \text{Residual Weight} = SW - BF \]
\[ RW = 2180023 - 900288 \]
\[ RW = 1279735 \text{ lbs} \]

**Summary of Results:**

- Self Weight = 2180.0 kips
- Buoyant Force = 900.3 kips
- Residual Weight = 1279.7 kips
- % Retained Weight = 58.7%

*: Bridge is NOT Buoyant*
Deck Capacity:

- $f_y = 40,000$ psi
- $f_c' = 2500$ psi

15 - $1/2\" Nk 422 Top
15 - $3/8\" Nk 611 Bot

$b = 20\,\text{in}$
$h = 14\,\text{in}$
Cover = 2\,\text{in} (top & Bot)

- Positive Bending (same as slip #2)
  - $A_s = 0.65\,\text{in}^2$
  - $a = 0.3075\,\text{in}$
  - $d = 11.6875\,\text{in}$

- Negative Bending:
  - $A_s = (13)(0.20) = 2.6\,\text{in}^2$
  - $a = \frac{A_s f_y}{6} = \frac{(3)(40,000)}{0.85(3) (2500)(206)}
    - $a = 1.19806\,\text{in}$
  - $d = 11.75\,\text{in}$
  - $E_s > E_y$ (ok)

- Shear (same as slip #2)

  - $V_n = 286.4\,\text{kips}$
  - $\phi V_n = 214.8\,\text{kips}$

*Note: No. Spans = 12
Max Length = 41\,\text{ft}
Int. Length = 32.5\,\text{ft}
Min Length = 29\,\text{ft}
Bent Cap Capacity:

No. Bent Caps = 12
Max Length = 41 ft
Int. Length = 32.5 ft
Min Length = 29 ft

Positive Bending:

\[ M_n = 251.49 \, k-ft \]
\[ \varnothing M_n = 226.34 \, k-ft \]

Negative Bending:

\[ M_n = 277.819 \, k-ft \]
\[ \varnothing M_n = 250.04 \, k-ft \]

Negative Shear:

\[ V_n = 124.05 \, kips \]
\[ \varnothing V_n = 93.04 \, kips \]
Appendix B: Douglass Wave Estimation Method

Calculations
Kuliouou Bridge:
Method For Estimating Wave Forces on Bridge Superstructures

Douglass (2006)

**Vertical Force Equation:**

\[ F_v = c_{v-nc} \cdot \Delta \rho \cdot g \cdot A_p \]

\[ F_v = c_{v-c} \cdot (\Delta \rho \cdot g) \cdot A_p \]

**Horizontal Force Equation:**

\[ F_h = (1 + c_{x-nc} \cdot (N - 1)) \cdot c_{x-c} \cdot F_v \]

\[ F_h = (1 + c_{x-c} \cdot (N - 1)) \cdot c_{x-c} \cdot F_v \]

**Constant Coefficients:**

- \( c_{v-nc} = 0.4 \)
- \( c_{v-c} = 0.64 \) lb/cubic ft
- \( c_{x-nc} = 1 \) Nonconserv.
- \( c_{x-c} = 2 \) Conserv.
- \( c_{x-nc} = 1 \) Nonconserv.
- \( c_{x-c} = 2 \) Conserv.

**Vertical Force Calculations:**

Bridge Deck Width = 68.75 ft Water Depth = 3.94 ft
Bridge Deck Length = 48.40 ft Water surface to bot. of girder = 1.00 ft
Height of girder = 3.00 ft

\( A_v = 3327.21 \text{ sq ft} \)
Elevation to bot. of girder = 4.94 ft
N = 12 girders Elevation to bot. of deck = 7.94 ft

Storm Surge Depth + Wave Height = 8.00 ft
(from NFIP Flood Hazard Assessment Tool)

\[ L_{wb} = (0.455 \times \text{storm surge depth}) \]

Wave Height = 2.50 ft (above storm surge elevation)

- **Largest Unbroken Wave:** (0.455*storm surge depth)

**Resulting Vertical Force:**

Fv = 332.19 kips 166.09 tons (Non Conservative Value)

Fv = 664.38 kips 332.19 tons (Conservative Value)

**Vertical Resistance:** (from hand calculations)

- **Sources of Vertical Resistance:**
  - Self Weight = 723,777 lbs 723.777 kips
  - Bearing Plates = 433,453 lbs 433.453 kips

**Total Vertical Resistance:**

Rv = 1,157.23 kips

- **Comparison of Vertical Wave Force to Vertical Resistance:**

<table>
<thead>
<tr>
<th>Fv (kips)</th>
<th>Rv (kips)</th>
<th>Bridge Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>332.19</td>
<td>&lt; 1157.23</td>
<td>O.K.</td>
</tr>
<tr>
<td>664.38</td>
<td>&lt; 1157.23</td>
<td>O.K.</td>
</tr>
</tbody>
</table>

**Horizontal Force Calculations:**

- **Railing Height** = 2.708 ft Water Depth = 3.94 ft
- **Deck Height** = 0.867 ft Water surface to bot. of girder = 1.00 ft
- **Girder Height** = 3.000 ft

Bridge Total Height = 6.375 ft Elevation to bot. of girder = 4.94 ft
Bridge Length = 48.396 ft Elevation of centroid of Ah = 8.13 ft

Ah = 121.07 sq ft
N = 12 girders

- **Largest Unbroken Wave:** (0.455*storm surge depth)
Wave Height = 2.50 ft (above storm surge elevation)

- **Resulting Horizontal Force:**
  
  \[ F_h = 52.34 \text{ kips} \quad 26.17 \text{ tons} \]
  
  \[ F_h = 104.68 \text{ kips} \quad 52.34 \text{ tons} \]

- **Horizontal Resistance: (from hand calculations)**
  
  - **Sources of Horizontal Resistance:**
    
    - Self Weight = 723,777 lbs
    - Girdler Seat Interface = Neoprene Pads
      \[ \mu_s = 0.1 \]
    - Frictional Resistance = 72,377.7 lbs
    - Bearing Plates = 345,312 lbs
    - Beam Web Capacity = 410,116 lbs
    - Koko Head wing wall = 59,400 lbs
  
  - **Total Horizontal Resistance:**
    \[ Rh = 518.10 \text{ kips} \]

- **Comparison of Horizontal Wave Force to Horizontal Resistance:**

<table>
<thead>
<tr>
<th>Fh</th>
<th>Rh</th>
<th>Bridge Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>52.34 kips</td>
<td>&lt; 518.10 kips</td>
<td>O.K.</td>
</tr>
<tr>
<td>104.68 kips</td>
<td>&lt; 518.10 kips</td>
<td>O.K.</td>
</tr>
</tbody>
</table>
**Kahaluu Bridge:**
Method For Estimating Wave Forces on Bridge Superstructures

Douglass (2006)

**Vertical Force Equation:**
\[
F_v = c_{v suspense} \cdot F_v^r \\
F_v^r = y \cdot (\Delta s_y) \cdot A_y
\]

**Horizontal Force Equation:**
\[
F_h = [1 + C_v \cdot (N - 1)] \cdot c_{h suspense} \cdot F_h^r \\
F_h^r = y \cdot (\Delta s_y) \cdot A_y
\]

**Constant Coefficients:**
- \( C_r = 0.4 \)
- \( y = 64 \) lb/cubic ft
- \( c_{v suspense} = 1 \) Nonconserv.
- \( c_{h suspense} = 2 \) Conserv.
- \( c_{v suspense} = 1 \) Nonconserv.
- \( c_{h suspense} = 2 \) Conserv.

* Vertical Force Calculations:

<table>
<thead>
<tr>
<th>Bridge Deck Width</th>
<th>46.00 ft</th>
<th>Water Depth</th>
<th>5.75 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge Deck Length</td>
<td>318.00 ft</td>
<td>Water surface to bot. of girder</td>
<td>5.00 ft</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Height of girder</td>
<td>4.50 ft</td>
</tr>
</tbody>
</table>

\( A_v = 14628.00 \) sq ft  
\( N = 8 \) girders  
\( \text{Elevation to bot. of girder} = 10.75 \) ft  
\( \text{Elevation of centroid of } A_h = 14.31 \) ft  
\( \text{Bridge Total Height} = 7.125 \) ft

**Largest Unbroken Wave:** (0.455*storm surge depth)
- Wave Height = 5.00 ft  
  (above storm surge elevation)

**Resulting Vertical Force:**
- \( F_v = 2808.58 \) kips  
  1404.29 tons  
  (Non Conservative Value)
- \( F_v = 5617.15 \) kips  
  2808.58 tons  
  (Conservative Value)

**Vertical Resistance:** (from hand calculations)
- **Sources of Vertical Resistance:**
  - \( \text{Self Weight} = 3811.55 \) kips
- **Total Vertical Resistance:**
  \( R_v = 3811.55 \) kips

**Comparison of Vertical Wave Force to Vertical Resistance:**
- \( F_v = 2808.58 \) kips  
  \(< \)  
  \( R_v = 3811.55 \) kips  
  O.K.
- \( F_v = 5617.15 \) kips  
  \(>\)  
  \( R_v = 3811.55 \) kips  
  Failure

* Horizontal Force Calculations:

<table>
<thead>
<tr>
<th>Railing Height</th>
<th>2.125 ft</th>
<th>Water Depth</th>
<th>5.75 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck Height</td>
<td>0.500 ft</td>
<td>Water surface to bot. of girder</td>
<td>5.00 ft</td>
</tr>
<tr>
<td>Girder Height</td>
<td>4.500 ft</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bridge Total Height</td>
<td>7.125 ft</td>
<td>Elevation of centroid of ( A_h )</td>
<td>14.31 ft</td>
</tr>
</tbody>
</table>

\( A_h = 2265.75 \) sq ft  
\( N = 8 \) girders

**Largest Unbroken Wave:** (0.455*storm surge depth)
- Wave Height = 5.00 ft  
  (above storm surge elevation)
• Resulting Horizontal Force:
  \[ F_h = 929.86 \text{ kips} \quad 464.93 \text{ tons} \]
  \[ F_h = 1859.73 \text{ kips} \quad 929.86 \text{ tons} \]

• Horizontal Resistance: (from hand calculations)
  • Sources of Horizontal Resistance:
    - Self Weight = 3411350 lbs
    - Girdler Seat Interface = Steel to Concrete \( \mu_s = 0.4 \)
    - Frictional Resistance = 1524620 lbs
    - Wing Walls = 299260 lbs
  • Total Horizontal Resistance:
    \[ R_h = 1823.88 \text{ kips} \]

• Comparison of Horizontal Wave Force to Horizontal Resistance:

<table>
<thead>
<tr>
<th>( F_h )</th>
<th>( R_h )</th>
<th>Bridge Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>929.86 kips</td>
<td>1823.88 kips</td>
<td>O.K.</td>
</tr>
<tr>
<td>1859.73 kips</td>
<td>1823.88 kips</td>
<td>Failure</td>
</tr>
</tbody>
</table>

Bridge Condition:
- O.K.
- Failure
New South Punaluu Bridge: (Span #2)
Method For Estimating Wave Forces on Bridge Superstructures

Douglass (2006)

**Vertical Force Equation:**

\[
F_v = C_r \cdot \gamma \cdot A_p \cdot (\Delta h) \cdot (N - 1) \\
F_v = \gamma \cdot (\Delta h) \cdot A_p 
\]

**Horizontal Force Equation:**

\[
F_h = [1 + C_r \cdot (N - 1)] \cdot C_p \cdot \gamma \cdot (\Delta h) \cdot A_p \\
F_h = \gamma \cdot (\Delta h) \cdot A_p 
\]

**Constant Coefficients:**

- \( C_r = 0.4 \)
- \( \gamma = 64 \) lb/cubic ft
- \( C_p = 1 \) Nonconserv.
- \( C_p = 2 \) Conserv.
- \( C_p = 1 \) Nonconserv.
- \( C_p = 2 \) Conserv.

**Vertical Force Calculations:**

Bridge Deck Width = 50.00 ft
Bridge Deck Length = 66.00 ft

\[
Av = 3300.00 \text{ sq ft} \\
N = 30 \text{ girders} \\
\text{Storm Surge Depth + Wave Height} = 12.00 \text{ ft} \\
\text{Bridge Deck Width} = 50.00 \text{ ft} \\
\text{Water Depth} = 4.92 \text{ ft} \\
\text{Height of girder} = 1.75 \text{ ft} \\
\text{Elevation to bot. of deck} = 6.67 \text{ ft} \\
\text{Elevation to bot. of girders} = 4.92 \text{ ft} \\
\text{Bridge Condition: O.K.} \\
\text{Bridge Length} = 66.00 \text{ ft} \\
\text{Elevation of centroid of Ah} = 7.73 \text{ ft} \\
Ah = 371.25 \text{ sq ft} \\
N = 30 \text{ girders} \\
\text{Storm Surge Depth} = 8.25 \text{ ft} \\
\text{Wave Height} = 3.75 \text{ ft} \\
\text{(above storm surge elevation)}
\]

**Largest Unbroken Wave:** \((0.455 \times \text{storm surge depth})\)

Wave Height = 3.75 ft (above storm surge elevation)

**Resulting Vertical Force:**

- \( F_v = 1125.70 \text{ kips} \) (Non Conservative Value)
- \( F_v = 2251.39 \text{ kips} \) (Conservative Value)

**Vertical Resistance:** (from hand calculations)

- **Sources of Vertical Resistance:**
  - Deck Capacity = 1262990 lbs
  - Rv = 1262.99 kips
- **Total Vertical Resistance:**
  - Rv = 1262.99 kips

**Comparison of Vertical Wave Force to Vertical Resistance:**

<table>
<thead>
<tr>
<th>Fv (kips)</th>
<th>Rv (kips)</th>
<th>Bridge Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>1125.70</td>
<td>1262.99</td>
<td>O.K.</td>
</tr>
<tr>
<td>2251.39</td>
<td>1262.99</td>
<td>Failure</td>
</tr>
</tbody>
</table>

**Horizontal Force Calculations:**

- **Railing Height** = 3.000 ft
- **Deck Height** = 0.875 ft
- **Girder Height** = 1.750 ft
- **Bridge Total Height** = 5.625 ft
- **Bridge Length** = 66.000 ft
- **Water Depth** = 4.92 ft
- **Elevation to bot. of girder** = 4.92 ft
- **Elevation of centroid of Ah** = 7.73 ft

**Largest Unbroken Wave:** \((0.455 \times \text{storm surge depth})\)

Wave Height = 3.75 ft (above storm surge elevation)
• **Resulting Horizontal Force:**

\[
F_h = 1277.59 \text{ kips} \quad 638.79 \text{ tons}
\]

\[
F_h = 2555.17 \text{ kips} \quad 1277.59 \text{ tons}
\]

• **Horizontal Resistance: (from hand calculations)**

  • **Sources of Horizontal Resistance:**
    - Self Weight = 907306 lbs (for span #2)
    - Girder Seat Interface = Concrete to Concrete
    - \( \mu_s = 0.8 \)
    - Frictional Resistance = 725844.8 lbs

• **Total Horizontal Resistance:**

\[
R_h = 725.84 \text{ kips}
\]

• **Comparison of Horizontal Wave Force to Horizontal Resistance:**

<table>
<thead>
<tr>
<th>Force (Fh)</th>
<th>Resistance (Rh)</th>
<th>Bridge Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>1277.59 kips</td>
<td>725.84 kips</td>
<td>Failure</td>
</tr>
<tr>
<td>2555.17 kips</td>
<td>725.84 kips</td>
<td>Failure</td>
</tr>
</tbody>
</table>
New Makaha No. 3A Bridge: Method For Estimating Wave Forces on Bridge Superstructures

Douglass (2006)

**Vertical Force Equation:**

\[ F_v = c_{v,\text{avg}} \cdot F_v^0 \]

\[ F_v^0 = \gamma \cdot (\Delta a) \cdot A_v \]

\[ F_v^h = \gamma \cdot (\Delta a) \cdot A_v \]

**Horizontal Force Equation:**

\[ F_h = [1 + c_r \cdot (N - 1)] \cdot c_b \cdot F_h^0 \]

**Constant Coefficients:**

- \( c_r = 0.4 \)
- \( \gamma = 64 \text{ lb/cubic ft} \)
- \( c_{v,\text{avg}} = 1 \) Nonconserv.
- \( c_b = 2 \) Conserv.
- \( c_b = 1 \) Nonconserv.
- \( c_b = 2 \) Conserv.

**Vertical Force Calculations:**

Bridge Deck Width = 46.83 ft  
Water Depth = 1.00 ft  
Bridge Deck Length = 70.00 ft  
Water surface to bot. of girder = 9.59 ft  
Height of girder = 2.33 ft  
\( A_v = 3278.33 \text{ sq ft} \)  
Elevation to bot. of girder = 10.59 ft  
N = 1 girders  
Elevation to bot. of deck = 10.59 ft  
Storm Surge Depth + Wave Height = 13.00 ft  
Storm Surge Depth = 8.93 ft  
(from NFIP Flood Hazard Assessment Tool)

- **Largest Unbroken Wave:** \((0.455 \times \text{storm surge depth})\)
  
  Wave Height = 4.07 ft  
  (above storm surge elevation)

- **Resulting Vertical Force:**
  
  \( F_v = 505.65 \text{ kips} \)  
  252.83 tons  
  (Non Conservative Value)
  
  \( F_v = 1011.30 \text{ kips} \)  
  505.65 tons  
  (Conservative Value)

- **Vertical Resistance:** (from hand calculations)
  
  - **Sources of Vertical Resistance:**
    
    Deck Capacity = 1127130 lbs  
    1127.13 kips
  
  - **Total Vertical Resistance:**
    
    \( R_v = 1127.13 \text{ kips} \)

- **Comparison of Vertical Wave Force to Vertical Resistance:**
  
  \( F_v = 505.65 \text{ kips} \)  
  < \( R_v = 1127.13 \text{ kips} \)  
  O.K.
  
  \( F_v = 1011.30 \text{ kips} \)  
  < \( R_v = 1127.13 \text{ kips} \)  
  O.K.

**Horizontal Force Calculations:**

- **Largest Unbroken Wave:** \((0.455 \times \text{storm surge depth})\)

  Wave Height = 4.07 ft  
  (above storm surge elevation)
• Resulting Horizontal Force:
  \[ F_h = 7.64 \text{ kips} \quad 3.82 \text{ tons} \]
  \[ F_h = 15.28 \text{ kips} \quad 7.64 \text{ tons} \]

• Horizontal Resistance: (from hand calculations)
  • Sources of Horizontal Resistance:
    Self Weight = 116,277 lbs
    Girder Seat Interface = Concrete to Concrete
    Frictional Resistance = 924,16 lbs
    Shear Friction = 8,870,400 lbs

• Total Horizontal Resistance:
  \[ R_h = 9,799.82 \text{ kips} \]

• Comparison of Horizontal Wave Force to Horizontal Resistance:
  \[
  \begin{array}{c|c|c}
  \hline
  F_h & R_h & \text{Bridge Condition} \\
  \hline
  7.64 \text{ kips} & 9,799.82 \text{ kips} & \text{O.K.} \\
  15.28 \text{ kips} & 9,799.82 \text{ kips} & \text{O.K.} \\
  \hline
  \end{array}
  \]

Bridge Condition:

O.K.
**Old Makaha No. 3A Bridge:**

Method For Estimating Wave Forces on Bridge Superstructures

*Douglass (2006)*

**Vertical Force Equation:**

\[
F_v = c_{v-n} \cdot \gamma \cdot A_y
\]

**Horizontal Force Equation:**

\[
F_h = [1 + c_{v-n} \cdot (N - 1)] \cdot \gamma \cdot \Delta z_y \cdot A_x
\]

**Constant Coefficients:**

- \( c_{v-n} = 0.4 \)
- \( \gamma = 64 \) lb/cubic ft
- \( c_{v-nc} = 1 \) Nonconserv.
- \( c_{v-c} = 2 \) Conserv.
- \( c_{h-nc} = 1 \) Nonconserv.
- \( c_{h-c} = 2 \) Conserv.

**Vertical Force Calculations:**

- Bridge Deck Width = 32.83 ft
- Water Depth = 4.00 ft
- Bridge Deck Length = 78.83 ft
- Water surface to bot. of girder = 7.24 ft
- Height of girder = 1.50 ft
- \( Av = 2588.36 \) sq ft
- Elevation to bot. of girder = 11.24 ft
- N = 12 girders
- Elevation to bot. of deck = 12.74 ft
- Storm Surge Depth + Wave Height = 13.00 ft
- Storm Surge Depth = 8.93 ft
- Wave Height = 4.07 ft
- (above storm surge elevation)

**Largest Unbroken Wave:** (0.455*storm surge depth)

- \( F_v = 43.07 \) kips 21.54 tons (Non Conservative Value)
- \( F_v = 86.14 \) kips 43.07 tons (Conservative Value)

**Vertical Resistance:** (from hand calculations)

- Self Weight = 279.83 kips
- Drift Bolt Withdrawal Capacity = 69.3 kips
- Total Vertical Resistance = 349.13 kips

**Comparison of Vertical Wave Force to Vertical Resistance:**

Bridge Condition:

<table>
<thead>
<tr>
<th>Fv</th>
<th>Rv</th>
</tr>
</thead>
<tbody>
<tr>
<td>43.07</td>
<td>&lt; 279.83</td>
</tr>
<tr>
<td>86.14</td>
<td>&lt; 279.83</td>
</tr>
</tbody>
</table>

**Horizontal Force Calculations:**

- Railing Height = 0.000 ft
- Deck Height = 0.500 ft
- Girder Height = 1.500 ft
- Bridge Total Height = 2.000 ft
- Bridge Length = 78.833 ft
- \( Ah = 157.67 \) sq ft
- N = 12 girders

**Largest Unbroken Wave:** (0.455*storm surge depth)
Wave Height = 4.07 ft  (above storm surge elevation)

● Resulting Horizontal Force:
  \[ F_h = 41.41 \text{ kips} \] 20.71 tons
  \[ F_h = 82.82 \text{ kips} \] 41.41 tons

● Horizontal Resistance: (from hand calculations)

  • Sources of Horizontal Resistance:
    - Self Weight = 259,330 lbs
    - Girder Seat Interface = Wood to Wood (wet)  \[ \mu = 0.2 \]
    - Frictional Resistance = 55,966 lbs
    - Drift Bolts Lateral Capacity = 14,405.4 lbs

  • Total Horizontal Resistance:
    \[ R_h = 70.37 \text{ kips} \]

● Comparison of Horizontal Wave Force to Horizontal Resistance:

<table>
<thead>
<tr>
<th></th>
<th>( F_h ) = 41.41 kips</th>
<th>(&lt;)</th>
<th>( R_h ) = 70.37 kips</th>
<th>( \text{Bridge Condition:} )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( F_h ) = 82.82 kips</td>
<td>( &gt;)</td>
<td>( R_h ) = 70.37 kips</td>
<td>( \text{Failure} )</td>
</tr>
</tbody>
</table>
Maipalaoa Stream Bridge: (Maili Channel)
Method For Estimating Wave Forces on Bridge Superstructures

Douglass (2006)

Vertical Force Equation:  
\[ F_v = C_v \cdot \gamma \cdot (\Delta \pi_a) \cdot A_b \]

Horizontal Force Equation:  
\[ F_h = \gamma \cdot (\Delta \pi_a) \cdot A_b \]

Constant Coefficients:
- \( C_v = 0.4 \)
- \( \gamma = 64 \) lb/cubic ft
- \( N = \) Various values (Nonconserv. and Conserv.)

Vertical Force Calculations:
- Bridge Deck Width = 64.33 ft
- Water Depth = 3.00 ft
- Bridge Deck Length = 100.67 ft
- Water surface to bot. of girder = 3.50 ft
- Height of girder = 3.00 ft
- \( \Delta \pi_a = 6476.22 \) sq ft
- Elevation to bot. of girder = 6.50 ft
- Storm Surge Depth + Wave Height = 12.00 ft (from NFIP Flood Hazard Assessment Tool)
- Storm Surge Depth = 8.25 ft
- \( N = 16 \) girders
- Wave Height = 3.75 ft (above storm surge elevation)
- Largest Unbroken Wave: \((0.455 \times \text{storm surge depth})\)
- Wave Height = 3.75 ft
- Resulting Vertical Force:
  - \( F_v = 1036.20 \) kips
  - \( F_v = 2072.39 \) kips
- Vertical Resistance: (from hand calculations)
  - Self Weight = 1406690 lbs
  - Total Vertical Resistance:
    - \( R_v = 1406.69 \) kips
- Comparison of Vertical Wave Force to Vertical Resistance:
  - \( F_v = 1036.20 \) kips < \( R_v = 1406.69 \) kips
  - Bridge Condition: O.K.
  - \( F_v = 2072.39 \) kips > \( R_v = 1406.69 \) kips
  - Bridge Condition: Failure

Horizontal Force Calculations:
- Railing Height = 2.000 ft
- Deck Height = 0.500 ft
- Girder Height = 3.000 ft
- Bridge Total Height = 5.500 ft
- Bridge Length = 100.667 ft
- \( \Delta \pi_a = 553.67 \) sq ft
- \( N = 16 \) girders
- Wave Height = 3.75 ft (above storm surge elevation)
• Resulting Horizontal Force:

\[ F_h = 682.12 \text{ kips} \quad 341.06 \text{ tons} \]

\[ F_h = 1364.23 \text{ kips} \quad 682.12 \text{ tons} \]

• Horizontal Resistance: (from hand calculations)

  - Sources of Horizontal Resistance:
    - Self Weight = 1406690 lbs
    - Girder Seat Interface = neoprene pad
      \[ \mu_s = 0.1 \]
    - Frictional Resistance = 140669 lbs
    - Beam Web Flexural Capacity = 92016 lbs

  - Total Horizontal Resistance:
    \[ R_h = 232.69 \text{ kips} \]

• Comparison of Horizontal Wave Force to Horizontal Resistance:

<table>
<thead>
<tr>
<th>Bridge Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Failure</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>( F_h )</th>
<th>( R_h )</th>
<th>Bridge Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>682.12 kips</td>
<td>&gt;</td>
<td>232.69 kips</td>
</tr>
<tr>
<td>1364.23 kips</td>
<td>&gt;</td>
<td>232.69 kips</td>
</tr>
</tbody>
</table>

Bridge Condition:

B-12
Moanalua Stream Bridge: (Spans 3, 4, 5 & 6) *Note: The calculations that follow are for one span only, therefore the bridge weight is divided by 8.

Method For Estimating Wave Forces on Bridge Superstructures

Douglass (2006)

Vertical Force Equation:

\[ F_v = c_{v-\text{wave}} \cdot F_{vq} \]

Horizontal Force Equation:

\[ F_h = \left[ 1 + G_{v} \cdot (W - 1) \right] + c_{h-\text{wave}} \cdot F_{hq} \]

\[ F_{vq} = \gamma \cdot (\Delta z_g) \cdot A_p \]

\[ F_{hq} = \gamma \cdot (\Delta z_g) \cdot A_p \]

Constant Coefficients:

- \( c_{v-\text{wave}} = 0.4 \)
- \( \gamma = 64 \text{ lb/cubic ft} \)
- \( c_{h-\text{wave}} = 1 \) Nonconserv.
- \( c_{h-\text{wave}} = 2 \) Conserv.
- \( c_{h-\text{wave}} = 1 \) Nonconserv.
- \( c_{h-\text{wave}} = 2 \) Conserv.

\[ \bullet \text{ Vertical Force Calculations:} \]

- Bridge Deck Width = 64.33 ft
- Water Depth = 2.50 ft
- Bridge Deck Length = 27.00 ft
- Water surface to bottom of girder = 4.00 ft
- Height of girder = 1.83 ft
- \( A_v = 1737.00 \text{ sq ft} \)
- Elevation to bottom of girder = 6.50 ft
- N = 9 girders
- Elevation to bottom of deck = 8.33 ft
- Storm Surge Depth = 6.87 ft
- Wave Height = 3.13 ft (above storm surge elevation)

- \( \bullet \text{ Largest Unbroken Wave: } (0.455 \text{storm surge depth}) \)

- \( \bullet \text{ Resulting Vertical Force:} \)

\[ F_v = 185.28 \text{ kips} \]

\[ F_v = 370.56 \text{ kips} \]

- (Non Conservative Value)

- (Conservative Value)

- \( \bullet \text{ Vertical Resistance: } (\text{from hand calculations}) \)

- Sources of Vertical Resistance:

  - Self Weight = 417270 lbs
  - Pier Cap Weight = 158400 lbs

- \( \bullet \text{ Total Vertical Resistance:} \)

\[ R_v = 575.67 \text{ kips} \]

- \( \bullet \text{ Comparison of Vertical Wave Force to Vertical Resistance:} \)

\[ F_v = 185.28 \text{ kips} < R_v = 575.67 \text{ kips} \]

\[ F_v = 370.56 \text{ kips} < R_v = 575.67 \text{ kips} \]

- Bridge Condition:

\[ \text{O.K.} \]

- \( \bullet \text{ Horizontal Force Calculations:} \)

- Railing Height = 3.729 ft
- Deck Height = 0.867 ft
- Girder Height = 1.833 ft
- Bridge Total Height = 6.229 ft
- Bridge Length = 27.000 ft
- \( A_h = 168.19 \text{ sq ft} \)
- N = 9 girders

- \( \bullet \text{ Largest Unbroken Wave: } (0.455 \text{storm surge depth}) \)
Wave Height = 3.13 ft (above storm surge elevation)

- **Resulting Horizontal Force:**
  
  \[
  F_h = 17.42 \text{ kips} \quad 8.71 \text{ tons} \\
  F_h = 34.85 \text{ kips} \quad 17.42 \text{ tons}
  \]

- **Horizontal Resistance:** *(from hand calculations)*
  
  - **Sources of Horizontal Resistance:**
    - Self Weight = 417,270 lbs
    - Girder Seat Interface = concrete to concrete
      - \( \mu_s = 0.8 \)
    - Frictional Resistance = 333,816 lbs
    - Dowel Shear Friction = 1,592,640 lbs
  
  - **Total Horizontal Resistance:**
    - \( R_h = 1,926.46 \text{ kips} \)

- **Comparison of Horizontal Wave Force to Horizontal Resistance:**
  
<table>
<thead>
<tr>
<th>( F_h )</th>
<th>( &lt; )</th>
<th>( R_h )</th>
</tr>
</thead>
<tbody>
<tr>
<td>17.42 \text{ kips}</td>
<td>( &lt; )</td>
<td>1,926.46 \text{ kips}</td>
</tr>
<tr>
<td>34.85 \text{ kips}</td>
<td>( &lt; )</td>
<td>1,926.46 \text{ kips}</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Bridge Condition:</th>
<th>O.K.</th>
</tr>
</thead>
</table>

*Note: Spans 2 & 7 are the weakest, as they are only tied at one pier, the other side of each span is on a concrete to concrete interface.*
Kalii Stream Bridge: (Typical Span)

Method For Estimating Wave Forces on Bridge Superstructures

*Note: The calculations that follow are for one span only, therefore the bridge weight is divided by 7.

Douglass (2006)

Vertical Force Equation:

\[ F_v = C_w \cdot \gamma \cdot (|A_h| + b) \cdot A_p \]

Horizontal Force Equation:

\[ F_h = [1 + C_r \cdot (W - 1)] \cdot \gamma \cdot (\Delta a) \cdot A_p \]

Constant Coefficients:

- \( C_r = 0.4 \)
- \( \gamma = 64 \text{ lbs/cubic ft} \)
- \( C_{w-\text{vol}} = 1 \) Nonconserv.
- \( C_{w-\text{vol}} = 2 \) Conserv.
- \( C_{s-\text{vol}} = 1 \) Nonconserv.
- \( C_{s-\text{vol}} = 2 \) Conserv.

Vertical Force Calculations:

- Bridge Deck Width = 88.33 ft
- Bridge Deck Length = 27.00 ft
- Water Depth = 2.50 ft
- Water surface to bot. of girder = 4.00 ft
- Height of girder = 1.83 ft
- Bridge Deck Length = 27.00 ft
- Water Depth = 2.50 ft
- Water surface to bot. of girder = 4.00 ft
- Height of girder = 1.83 ft
- Bridge Deck Length = 27.00 ft
- Water Depth = 2.50 ft
- Water surface to bot. of girder = 4.00 ft
- Height of girder = 1.83 ft

Av = 2385.00 sq ft

N = 13 girders

Storm Surge Depth + Wave Height = 10.00 ft

Largest Unbroken Wave: \((0.455 \times \text{storm surge depth})\)

Wave Height = 3.13 ft

Resulting Vertical Force:

Vertical Resistance:

- Self Weight = 565087 lbs = 565.087 kips

Comparison of Vertical Wave Force to Vertical Resistance:

<table>
<thead>
<tr>
<th>Fv (kips)</th>
<th>Rv (kips)</th>
<th>Bridge Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>254.40</td>
<td>565.087</td>
<td>O.K.</td>
</tr>
<tr>
<td>508.80</td>
<td>565.087</td>
<td>O.K.</td>
</tr>
</tbody>
</table>

Horizontal Force Calculations:

- Railing Height = 3.729 ft
- Deck Height = 0.667 ft
- Girder Height = 1.833 ft
- Bridge Total Height = 6.229 ft
- Bridge Length = 27.000 ft
- Water Depth = 2.50 ft
- Water surface to bot. of girder = 4.00 ft
- Elevation of centroid of Ah = 9.61 ft

Ah = 168.19 sq ft

N = 13 girders

Largest Unbroken Wave: \((0.455 \times \text{storm surge depth})\)

Wave Height = 3.13 ft

(above storm surge elevation)
• Resulting Horizontal Force:
  \[ F_h = 24.06 \text{ kips} \quad 12.03 \text{ tons} \]
  \[ F_h = 48.12 \text{ kips} \quad 24.06 \text{ tons} \]

• Horizontal Resistance: (from hand calculations)
  - Sources of Horizontal Resistance:
    - Self Weight = 565087 lbs
    - Girder Seat Interface = concrete to concrete
      \( \mu = 0.8 \)
    - Frictional Resistance = 452069.6 lbs
  - Total Horizontal Resistance:
    \[ R_h = 452.07 \text{ kips} \]

• Comparison of Horizontal Wave Force to Horizontal Resistance:

<table>
<thead>
<tr>
<th></th>
<th>( F_h )</th>
<th>( Rh )</th>
<th>Bridge Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>24.06 kips</td>
<td>452.07 kips</td>
<td>O.K.</td>
</tr>
<tr>
<td></td>
<td>48.12 kips</td>
<td>452.07 kips</td>
<td>O.K.</td>
</tr>
</tbody>
</table>
Nimitz Highway at Aloha Tower Slip Cover #2:
Method For Estimating Wave Forces on Bridge Superstructures
Douglass (2006)

Vertical Force Equation:  \[ F_v = C_v \cdot \gamma \cdot h \cdot A_v \]

Horizontal Force Equation:  \[ F_h = (1 + C_v) \cdot \gamma \cdot (A_h + A_g) \]

Constant Coefficients:
- \( C_v \) = 0.4
- \( \gamma \) = 64 lb/cubic ft
- \( A_v \) = 1 Nonconserv.
- \( A_g \) = 2 Conserv.
- \( A_h \) = 1 Nonconserv.
- \( A_g \) = 2 Conserv.

Vertical Force Calculations:
- Bridge Deck Width (1) = 67 ft
- Water Depth = 5.33 ft
- Bridge Deck Width (2) = 53.00 ft
- Water surface to bot. of girder = 4.50 ft
- Bridge Deck Length = 178.41 ft
- Height of girder = 2.50 ft
- \( A_v = 10704.60 \) sq ft
- Elevation to bot. of girder = 9.83 ft
- N = 11 girders
- Elevation to bot. of deck = 12.33 ft
- Storm Surge Depth = Wave Height = 8.00 ft (from NFIP Flood Hazard Assessment Tool)
- Storm Surge Depth = 5.50 ft
- Largest Unbroken Wave: \((0.455 \times \text{storm surge depth})\)
- Wave Height = 2.50 ft (above storm surge elevation)
- Resulting Vertical Force:
  \( F_v = 0.00 \) kips \( 0.00 \) tons (Non Conservative Value)
  \( F_v = 0.00 \) kips \( 0.00 \) tons (Conservative Value)
- Vertical Resistance: (from hand calculations)
  - Sources of Vertical Resistance:
    - Self Weight = 2069453 lbs 2069.45 kips
  - Total Vertical Resistance:
    \( R_v = 2069.45 \) kips
- Comparison of Vertical Wave Force to Vertical Resistance:
  \( F_v = 0.00 \) kips < \( R_v = 2069.45 \) kips Note: With 100 year flood data from the NFIP, the waves will miss the bottom elevation of the bridge

Horizontal Force Calculations:
- Railing Height = 1.667 ft
- Deck Height = 1.292 ft
- Girders Height = 2.500 ft
- Bridge Total Height = 5.458 ft
- Bridge Length = 178.410 ft
- \( A_h = 973.82 \) sq ft
- N = 11 girders
- Largest Unbroken Wave: \((0.455 \times \text{storm surge depth})\)
- Wave Height = 2.50 ft (above storm surge elevation)
- Resulting Horizontal Force:
  \( F_h = 0.00 \) kips \( 0.00 \) tons
  \( F_h = 0.00 \) kips \( 0.00 \) tons
- Horizontal Resistance: (from hand calculations)
- **Sources of Horizontal Resistance**:
  - **Self Weight**: 2069453 lbs
  - **Girder Seat Interface = Concrete to Concrete**: $\mu_s = 0.8$
  - **Frictional Resistance**: 1655562 lbs

- **Total Horizontal Resistance**: $R_h = 1655.56$ kips

- **Comparison of Horizontal Wave Force to Horizontal Resistance**:
  - $F_h = 0.00$ kips $< R_h = 1655.56$ kips
  - Note: With 100 year flood data from the NFIP, the waves will miss the bottom elevation of the bridge.

<table>
<thead>
<tr>
<th>$F_h$</th>
<th>$R_h$</th>
<th>Bridge Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00 kips</td>
<td>1655.56 kips</td>
<td>O.K.</td>
</tr>
</tbody>
</table>

- **Bridge Condition**: O.K.
Nimitz Highway at Aloha Tower Slip Cover #3:
Method For Estimating Wave Forces on Bridge Superstructures

Douglass (2006)

**Vertical Force Equation:**

\[ F_v = 0.4 \gamma \left( \rho_h - \rho_w \right) \frac{B}{L} \]

**Horizontal Force Equation:**

\[ F_h = \frac{1}{2} \rho_h \left( \Delta V \right) \frac{B}{L} \]

**Constant Coefficients:**

- \( \gamma = 64 \) lb/cubic ft
- \( \rho_h = 1 \) Nonconserv.
- \( \rho_w = 2 \) Conserv.
- \( \Delta V = 1 \) Nonconserv.
- \( \Delta h = 2 \) Conserv.

**Vertical Force Calculations:**

- Bridge Deck Width (1) = 29 ft
- Water Depth = 5.33 ft
- Bridge Deck Width (2) = 41.00 ft
- Water surface to bot. of girder = 4.50 ft
- Bridge Deck Width (3) = 32.50 ft
- Height of girder = 2.50 ft
- Bridge Deck Length (1) = 162.00 ft
- Bridge Deck Length (2) = 78.00 ft
- \( A_v = 836.50 \) sq ft
- Elevation to bot. of girder = 9.83 ft
- \( N = 13 \) girders
- Elevation to bot. of deck = 12.33 ft
- Storm Surge Depth + Wave Height = 8.00 ft (from NFIP Flood Hazard Assessment Tool)
- Storm Surge Depth = 5.50 ft

- Largest Unbroken Wave: \((0.455 \times \text{storm surge depth})\)
  - Wave Height = 2.50 ft (above storm surge elevation)

- Resulting Vertical Force:
  - \( F_v = 0.00 \) kips 0.00 tons (Non Conservative Value)
  - \( F_v = 0.00 \) kips 0.00 tons (Conservative Value)

- Vertical Resistance: (from hand calculations)
  - Sources of Vertical Resistance:
    - Self Weight = 2180023 lbs 2180.02 kips
  - Total Vertical Resistance:
    - \( R_v = 2180.02 \) kips

- Comparison of Vertical Wave Force to Vertical Resistance:
  - \( F_v = 0.00 \) kips < \( R_v = 2180.02 \) kips

- Bridge Condition: O.K.

**Horizontal Force Calculations:**

- Railing Height = 1.667 ft
- Deck Height = 1.292 ft
- Girder Height = 2.500 ft
- Bridge Total Height = 5.458 ft
- Bridge Length = 240.000 ft
- \( \Delta h = 5.485 \) ft
- Elevation to bot. of girder = 9.83 ft
- Elevation of centroid of \( Ah \) = 12.56 ft
- \( Ah = 1310.00 \) sq ft
- \( N = 13 \) girders

- Largest Unbroken Wave: \((0.455 \times \text{storm surge depth})\)
  - Wave Height = 2.50 ft (above storm surge elevation)

- Resulting Horizontal Force:
  - \( F_h = 0.00 \) kips 0.00 tons
  - \( F_h = 0.00 \) kips 0.00 tons
**Horizontal Resistance:** (from hand calculations)

- **Sources of Horizontal Resistance:**
  - Self Weight = 2180023 lbs
  - Girder Seat Interface = Concrete to Concrete $\mu_s = 0.8$
  - Frictional Resistance = 1744018 lbs

- **Total Horizontal Resistance:**
  $Rh = 1744.02$ kips

**Comparison of Horizontal Wave Force to Horizontal Resistance:**

<table>
<thead>
<tr>
<th>$F_h$ (kips)</th>
<th>$Rh$ (kips)</th>
<th>Bridge Condition</th>
<th>Note: With 100 year flood data from the NFIP, the waves will miss the bottom elevation of the bridge</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>1744.02</td>
<td>O.K.</td>
<td></td>
</tr>
</tbody>
</table>
Appendix C: McPherson Wave Estimation Method

Calculations
Kuliouou Bridge:
Method For Estimating Wave Forces on Bridge Superstructures

McPherson (2008)

Vertical Force Equation:

\[ \sum F_{\text{vertical}} = F_{\text{hydrostatic}} + F_{\text{bridge}} + F_{\text{fair\text{-}entrapment}} \]

Horizontal Force Equation:

\[ \sum F_{\text{horizontal}} = F_{\text{hydrostatic}} + F_{\text{bridge}} \]

Constant Coefficients:

\[ \gamma = 64 \text{ lb/cubic ft} \]

Vertical Force Calculations:

- Bridge Deck Width = 68.75 ft
- Bridge Deck Length = 48.40 ft
- Area (A) = 3327.21 sq ft
- Vol_bridge = 4825.18 cubic ft
- Vol_trapped air/2 = 3475.35 cubic ft
- Elevation to bot. of girder = 4.94 ft
- Storm Surge Depth + Wave Height = 8.00 ft (from NFIP Flood Hazard Assessment Tool)
- Storm Surge Depth = 5.50 ft
- Largest Unbroken Wave: \((0.455 \times \text{storm surge depth})\)
- Wave Height = 2.50 ft (above storm surge elevation)

Equation Values:

\[ \delta z = 0.61 \text{ ft} \] (Difference between top of deck and highest point on wave)

\[ F_w = 0 \text{ lbs} \]

\[ F_{\text{hydrostatic}} = 0 \text{ lbs} \]

\[ F_{\text{bridge}} = 308811.52 \text{ lbs} \]

\[ F_{\text{fair\text{-}entrapment}} = 222422.08 \text{ lbs} \]

Resulting Vertical Force:

\[ F_v = 531.23 \text{ kips} \]

(265.62 tons)

Vertical Resistance: (from hand calculations)

- Sources of Vertical Resistance:
  - Self Weight = 723.78 kips
  - Bearing Plates = 433.45 kips

- Total Vertical Resistance:
  \[ R_v = 1157.23 \text{ kips} \]

Comparison of Vertical Wave Force to Vertical Resistance:

\[ F_v = 531.23 \text{ kips} < R_v = 1157.23 \text{ kips} \]

Bridge Condition: O.K.

Horizontal Force Calculations:

- Railing Height = 2.708 ft
- Deck Height = 0.667 ft
- Girder Height = 3.000 ft
- Bridge Total Height = 6.375 ft

- Water Depth = 3.94 ft
- Water surface to bot. of girder = 1.00 ft
- Deck Height = 0.667 ft
- Water surface to bot. of girder = 1.00 ft
- Elevation to bot. of deck = 7.94 ft
- Storm Surge Depth = 5.50 ft
- Storm Surge Depth + Wave Height = 8.00 ft (from NFIP Flood Hazard Assessment Tool)
Bridge Length = 48.396 ft  
Elevation to bot. of deck = 7.94 ft

Ah = 308.52 sq ft  
N = 12 girders

● Largest Unbroken Wave: (0.455*storm surge depth)

Wave Height = 2.50 ft  (above storm surge elevation)

● Equation Values:

\[ \eta_{\text{max}} = 2.50 \text{ ft} \]  
\[ H_{\text{bridge}} = 6.38 \text{ ft} \]  
\[ h = 5.50 \text{ ft} \]  
\[ h_{\text{girder}} = 4.94 \text{ ft} \]  
\[ h_{\text{deck}} = 7.94 \text{ ft} \]

\[ F_{\text{hydrostatic\_front}} = 30210.615 \text{ lbs} \]  
\[ \eta_{\text{max}} < h_{\text{deck}} \]

\[ F_{\text{hydrostatic\_back}} = 482.686 \text{ lbs} \]  
\[ \text{SWL} > h_{\text{girder}} \]

● Resulting Vertical Force:

\[ F_h = 29.73 \text{ kips} \]  
14.86 tons

● Horizontal Resistance: (from hand calculations)

• Sources of Horizontal Resistance:
  
  - Self Weight = 723777 lbs
  - Girder Seat Interface = Neoprene Pads  
    \[ \mu_s = 0.1 \]
  - Frictional Resistance = 72377.7 lbs
  - Bearing Plates = 343312 lbs
  - Beam Web Capacity = 41010 lbs
  - Koko Head wing wall = 59400 lbs

• Total Horizontal Resistance:

\[ R_h = 518.10 \text{ kips} \]

● Comparison of Horizontal Wave Force to Horizontal Resistance:

\[ F_h = 29.73 \text{ kips} \]  
\[ < R_h = 518.10 \text{ kips} \]

Bridge Condition: O.K.
Kahaluu Bridge: Method For Estimating Wave Forces on Bridge Superstructures

McPherson (2008)

Vertical Force Equation:

\[
\begin{align*}
F_{v_{\text{Total}}} &= F_{v_{\text{Hydrostatic}}} + F_{v_{\text{Bridge}}} + F_{v_{\text{Fair}}}
F_{v_{\text{Hydrostatic}}} &= \gamma A \Delta z \\
F_{v_{\text{Bridge}}} &= \frac{1}{2} \rho v^2 (h - h_{\text{model}})
F_{v_{\text{Fair}}}} = \frac{1}{\pi} \tan^{-1} \left( \frac{h - h_{\text{model}}}{h} \right)
\end{align*}
\]

Horizontal Force Equation:

\[
\begin{align*}
F_{h_{\text{Total}}} &= F_{h_{\text{Hydrostatic}}} + F_{h_{\text{Bridge}}}
F_{h_{\text{Hydrostatic}}} &= \gamma \frac{1}{2} \rho v^2 A
F_{h_{\text{Bridge}}} &= \frac{1}{2} \rho v^2 (h - h_{\text{model}})A
\end{align*}
\]

Constant Coefficients:

\[\gamma = 64 \text{ lb/cubic ft}\]

**Vertical Force Calculations:**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge Deck Width</td>
<td>46.00 ft</td>
</tr>
<tr>
<td>Bridge Deck Length</td>
<td>318.00 ft</td>
</tr>
<tr>
<td>Area (A)</td>
<td>14626 sq ft</td>
</tr>
<tr>
<td>Vol_bridge</td>
<td>23960.2 cubic ft</td>
</tr>
<tr>
<td>Vol_trapped air/2</td>
<td>19161.1 cubic ft</td>
</tr>
<tr>
<td>n</td>
<td>8 girders</td>
</tr>
<tr>
<td>Elev. to bot. of girder</td>
<td>10.75 ft</td>
</tr>
<tr>
<td>Elev. to bot. of deck</td>
<td>15.25 ft</td>
</tr>
<tr>
<td>Storm Surge Depth + Wave Height</td>
<td>16.00 ft (from NFIP Flood Hazard Assessment Tool)</td>
</tr>
<tr>
<td>Wave Height</td>
<td>5.00 ft (above storm surge elevation)</td>
</tr>
</tbody>
</table>

**Equation Values:**

\[\delta z = 0.25 \text{ ft} \quad \text{(Difference between top of deck and highest point on wave)}\]

\[F_w = 117024 \text{ lbs} \quad F_{\text{Hydrostatic}} = 117024 \text{ lbs} \quad F_{\text{Bridge}} = 1533452.8 \text{ lbs} \quad F_{\text{Fair}} = 1226310.4 \text{ lbs}\]

**Resulting Vertical Force:**

\[F_v = 2876.79 \text{ kips} \quad 1438.39 \text{ tons}\]

**Vertical Resistance:** (from hand calculations)

**Sources of Vertical Resistance:**

- Self Weight = 3811550 lbs = 3811.55 kips

**Total Vertical Resistance:**

\[R_v = 3811.55 \text{ kips}\]

**Comparison of Vertical Wave Force to Vertical Resistance:**

\[F_v = 2876.79 \text{ kips} < R_v = 3811.55 \text{ kips}\]

**Bridge Condition:** O.K.

**Horizontal Force Calculations:**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Railing Height</td>
<td>2.125 ft</td>
</tr>
<tr>
<td>Deck Height</td>
<td>0.500 ft</td>
</tr>
<tr>
<td>Girder Height</td>
<td>4.500 ft</td>
</tr>
<tr>
<td>Bridge Total Height</td>
<td>7.125 ft</td>
</tr>
<tr>
<td>Bridge Length</td>
<td>318.000 ft</td>
</tr>
<tr>
<td>Water Depth</td>
<td>5.75 ft</td>
</tr>
<tr>
<td>Water surface to bot. of girder</td>
<td>5.00 ft</td>
</tr>
<tr>
<td>Elev. to bot. of girder</td>
<td>10.75 ft</td>
</tr>
<tr>
<td>Elev. to bot. of deck</td>
<td>15.25 ft</td>
</tr>
</tbody>
</table>
\[ Ah = 2265.75 \text{ sq ft} \]
\[ N = 8 \text{ girders} \]

- **Largest Unbroken Wave:** \((0.455 \times \text{storm surge depth})\)
  
  Wave Height = 5.00 ft \((\text{above storm surge elevation})\)

- **Equation Values:**
  
  \[
  \begin{align*}
  \eta_{\text{max}} &= 5.00 \text{ ft} \quad (\text{Wave height above storm surge elevation}) \\
  H_{\text{bridge}} &= 7.13 \text{ ft} \\
  h &= 11.00 \text{ ft} \quad (\text{SWL}) \\
  h_{\text{girder}} &= 10.75 \text{ ft} \\
  h_{\text{deck}} &= 15.25 \text{ ft}
  \end{align*}
  \]

  \[
  \begin{align*}
  F_{\text{hydrostatic\_front}} &= 380646 \text{ lbs} \\
  \eta_{\text{max}} &< h_{\text{deck}} \\
  F_{\text{hydrostatic\_back}} &= 618.636 \text{ lbs} \\
  \text{SWL} &> h_{\text{girder}}
  \end{align*}
  \]

- **Resulting Vertical Force:**
  
  \[
  F_h = 380.03 \text{ kips} \quad 190.01 \text{ tons}
  \]

- **Horizontal Resistance:** \((\text{from hand calculations})\)
  
  - Sources of Horizontal Resistance:
    
    \[
    \begin{align*}
    \text{Self Weight} &= 3811550 \text{ lbs} \\
    \mu_s &= 0.4 \\
    \text{Girder Seat Interface} &= \text{Steel to Concrete} \\
    \text{Frictional Resistance} &= 1524620 \text{ lbs} \\
    \text{Wing Walls} &= 299260 \text{ lbs}
    \end{align*}
    \]

  - Total Horizontal Resistance:
    
    \[
    Rh = 1823.88 \text{ kips}
    \]

- **Comparison of Horizontal Wave Force to Horizontal Resistance:**
  
  \[
  F_h = 380.03 \text{ kips} < Rh = 1823.88 \text{ kips} \quad \text{Bridge Condition: O.K.} \]
New South Punaluu Bridge: (span #2)

Method For Estimating Wave Forces on Bridge Superstructures

McPherson (2008)

Vertical Force Equation:

\[
F_{\text{Total}} = F_{\text{hydrostatic}} + F_{\text{bridge}} + F_{\text{entrapped}}
\]

\[
F_{\text{hydrostatic}} = \gamma A \Delta z
\]

\[
F_{\text{entrapped}} = \frac{1}{2} \rho A (H + h_{\text{model}}) d
\]

\[
F_{\text{bridge}} = \gamma V_{\text{bridge}}
\]

\[
F_{\text{entrapped}} = (n - 1) \gamma S V_{\text{air}}
\]

Horizontal Force Equation:

\[
F_{\text{Total}} = F_{\text{hydrostatic}, \text{wave}} + F_{\text{bridge}}
\]

\[
F_{\text{wave,hydrostatic}} = \delta z \rho A H
\]

\[
F_{\text{bridge}} = 0.5 \left( (H + h_{\text{girder}}) - (H + h_{\text{deck}}) \right)^2
\]

Constant Coefficients:

\[
\gamma = 64 \text{ lb/cubic ft}
\]

Vertical Force Calculations:

- Bridge Deck Width = 50.00 ft
- Bridge Deck Length = 66.00 ft
- Area (A) = 3300 sq ft
- Vol_bridge = 14462.83 cubic ft
- Vol_trapped air/2 = 4402.245 cubic ft
- n = 30 girders
- Elevation to bot. of girder = 4.92 ft
- Storm Surge Depth + Wave Height = 12.00 ft
- Wave Height = 3.75 ft
- Elevation to bot. of deck = 6.67 ft

- \( \delta z = 4.46 \) ft

- \( F_{\text{entrapped}} = 281743.68 \) lbs

- \( F_{\text{bridge}} = 938421.12 \) lbs

- \( F_{\text{hydrostatic}} = 470448 \) lbs

- \( F_{\text{wave}} = 470448 \) lbs

- \( F_{\text{Total}} = 1690.61 \) kips

- \( R_{\text{v}} = 1262.99 \) kips

- \( F_{\text{v}} > R_{\text{v}} \)

 Horizontal Force Calculations:

- Railing Height = 3.000 ft
- Deck Height = 0.875 ft
- Girder Height = 1.750 ft
- Bridge Total Height = 5.625 ft
- Bridge Length = 66.000 ft

- \( \text{Bridge Condition: Failure} \)

(from NFIP Flood Hazard Assessment Tool)
Ah = 371.25 sq ft
N = 30 girders

- **Largest Unbroken Wave:** (0.455*storm surge depth)
  
  Wave Height = 3.20 ft (above storm surge elevation)

- **Equation Values:**
  
  \[ \eta_{\text{max}} = 3.20 \text{ ft} \quad (\text{Wave height above storm surge elevation}) \]
  
  \[ H_{\text{bridge}} = 5.63 \text{ ft} \]
  
  \[ h = 8.25 \text{ ft} \]
  
  \[ h_{\text{girder}} = 4.92 \text{ ft} \]
  
  \[ h_{\text{deck}} = 6.67 \text{ ft} \]

  \[ F_{\text{hydrostatic\_front}} = 77522.0214 \text{ lbs} \quad \eta_{\text{max}} < h_{\text{deck}} \]
  
  \[ F_{\text{hydrostatic\_back}} = 23383.518 \text{ lbs} \quad \text{SWL} > h_{\text{girder}} \]

- **Resulting Vertical Force:**
  
  \[ F_h = 54.14 \text{ kips} \quad 27.07 \text{ tons} \]

- **Horizontal Resistance:** *(from hand calculations)*

  - Sources of Horizontal Resistance:
    
    |                      | Value   |
    |----------------------|---------|
    | Self Weight          | 907302 lbs |
    | Girder Seat Interface | Concrete to Concrete |
    | Frictional Resistance | 725841.6 lbs |
    | Bearing Pads         | 766770 lbs |

  - **Total Horizontal Resistance:**
    
    \[ R_h = 1492.61 \text{ kips} \]

- **Comparison of Horizontal Wave Force to Horizontal Resistance:**

  | \( F_h = 54.14 \text{ kips} \) | \( R_h = 1492.61 \text{ kips} \) | Bridge Condition: | O.K. |

C-6
**New Makaha No.3A:**
Method For Estimating Wave Forces on Bridge Superstructures

McPherson (2008)

**Vertical Force Equation:**

\[ F_{\text{vertical}} = \gamma A - F_w - F_{\text{bridge}} - F_{\text{fairing}} \]

\[ F_{\text{vertical}} = \gamma A - F_w - F_{\text{bridge}} - F_{\text{fairing}} \]

\[ F_{\text{bridge}} = \frac{8}{3} \pi D \rho g h^3 \]

\[ F_{\text{fairing}} = \pi \delta z^2 \frac{h}{2} \]

\[ F_w = g A \delta z \]

\[ F_{\text{hydrostatic}} = g A \delta z \]

\[ \delta z = h - h_{\text{max}} \]

\[ \gamma = 64 \text{ lb/cubic ft} \]

**Horizontal Force Equation:**

\[ F_{\text{horizontal}} = F_{\text{hydrostatic}} + F_{\text{bridge}} + F_{\text{fairing}} \]

\[ F_{\text{horizontal}} = F_{\text{hydrostatic}} + F_{\text{bridge}} + F_{\text{fairing}} \]

\[ F_{\text{bridge}} = \frac{8}{3} \pi D \rho g h^3 \]

\[ F_{\text{fairing}} = \pi \delta z^2 \frac{h}{2} \]

\[ F_w = g A \delta z \]

\[ F_{\text{hydrostatic}} = g A \delta z \]

\[ \delta z = h - h_{\text{max}} \]

\[ \gamma = 64 \text{ lb/cubic ft} \]

**Vertical Force Calculations:**

<table>
<thead>
<tr>
<th>Bridge Deck Width</th>
<th>46.83 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge Deck Length</td>
<td>70.00 ft</td>
</tr>
<tr>
<td>Area (A)</td>
<td>3278.33 sq ft</td>
</tr>
<tr>
<td>Vol_bridge</td>
<td>7745.12 cubic ft</td>
</tr>
<tr>
<td>Vol_trapped air/2</td>
<td>0 cubic ft</td>
</tr>
<tr>
<td>n</td>
<td>1 girders</td>
</tr>
<tr>
<td>Elevation to bot. of girder</td>
<td>9.59 ft</td>
</tr>
<tr>
<td>Deck Height</td>
<td>2.33 ft</td>
</tr>
<tr>
<td>Height of girder</td>
<td>2.33 ft</td>
</tr>
<tr>
<td>Water Depth</td>
<td>1.00 ft</td>
</tr>
<tr>
<td>Water surface to bot. of girder</td>
<td>9.59 ft</td>
</tr>
<tr>
<td>Girder Height</td>
<td>2.33 ft</td>
</tr>
<tr>
<td>Storm Surge Depth + Wave Height</td>
<td>13.00 ft</td>
</tr>
<tr>
<td>Storm Surge Depth</td>
<td>8.93 ft</td>
</tr>
</tbody>
</table>

\[ \delta z = -0.38 \text{ ft} \]

\[ F_w = 0 \text{ lbs} \]

\[ F_{\text{hydrostatic}} = 0 \text{ lbs} \]

\[ F_{\text{bridge}} = 495687.68 \text{ lbs} \]

\[ F_{\text{fairing}} = 0 \text{ lbs} \]

\[ F_{\text{vertical}} = 495.69 \text{ kips} = 247.84 \text{ tons} \]

**Horizontal Force Calculations:**

<table>
<thead>
<tr>
<th>Railing Height</th>
<th>1.167 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck Height</td>
<td>0.458 ft</td>
</tr>
<tr>
<td>Girder Height</td>
<td>2.333 ft</td>
</tr>
<tr>
<td>Bridge Total Height</td>
<td>3.958 ft</td>
</tr>
<tr>
<td>Bridge Condition</td>
<td>O.K.</td>
</tr>
<tr>
<td>Bridge Length</td>
<td>70.000 ft</td>
</tr>
</tbody>
</table>

\[ \delta z = h - h_{\text{max}} \]

\[ F_{\text{horizontal}} = 0 \text{ lbs} \]

\[ F_{\text{hydrostatic}} = 0 \text{ lbs} \]

\[ F_{\text{bridge}} = 1127130 \text{ lbs} = 1127.13 \text{ kips} \]

\[ F_{\text{fairing}} = 0 \text{ lbs} \]

\[ F_{\text{horizontal}} = 1127.13 \text{ kips} \]

\[ F_{\text{bridge}} = 1127.13 \text{ kips} \]

\[ F_{\text{fairing}} = 0 \text{ lbs} \]
\[ A_h = 277.08 \text{ sq ft} \]
\[ N = 1 \text{ girders} \]

- **Largest Unbroken Wave:** 
  \[ (0.455 \times \text{storm surge depth}) \]
  \[ \text{Wave Height} = 4.07 \text{ ft (above storm surge elevation)} \]

- **Equation Values:**
  \[ \eta_{\max} = 4.07 \text{ ft (Wave height above storm surge elevation)} \]
  \[ H_{\text{bridge}} = 3.96 \text{ ft} \]
  \[ h = 8.93 \text{ ft} \]
  \[ h_{\text{girder}} = 10.59 \text{ ft} \]
  \[ h_{\text{deck}} = 12.92 \text{ ft} \]

- **\( F_{\text{hydrostatic\_front}} = 21368.6667 \text{ lbs} \)**
  \[ \eta_{\max} < h_{\text{deck}} \]

- **\( F_{\text{hydrostatic\_back}} = 0.000 \text{ lbs} \)**
  \[ \text{SWL} < h_{\text{girder}} \]

- **Resulting Vertical Force:**
  \[ F_h = 21.37 \text{ kips 10.68 tons} \]

- **Horizontal Resistance:**
  
  - **Sources of Horizontal Resistance:**
    - Self Weight = 1161770 lbs
    - Girder Seat Interface = Concrete to Concrete, \( \mu_s = 0.8 \)
    - Frictional Resistance = 929416 lbs
    - Shear Friction = 8870400 lbs

  - **Total Horizontal Resistance:**
    \[ R_h = 9799.82 \text{ kips} \]

- **Comparison of Horizontal Wave Force to Horizontal Resistance:**
  \[ F_h = 21.37 \text{ kips} < R_h = 9799.82 \text{ kips} \]
  
  Bridge Condition: O.K.
Old Makaha No.3A:
Method For Estimating Wave Forces on Bridge Superstructures

McPherson (2008)

Vertical Force Equation:

\[ F_{total} = F_{hydrostatic} + F_{bridge} + F_{entrapment} \]

\[ F_{hydrostatic} = \gamma g A - F_w \]

\[ F_{bridge} = \frac{1}{2} \rho g A \left( h - h_{model} \right) A \]

\[ F_{entrapment} = (\kappa - 1) \gamma b y_{girder} d \]

Horizontal Force Equation:

\[ F_{total} = F_{hydrostatic} + F_{hydrostatic, h} \]

\[ F_{hydrostatic, h} = 0.5 \times \left( (h_{max} - h) + (h - h_{girder}) \right) b y_{girder} d - h_{girder} \]

\[ F_{hydrostatic, h} = 0 \]

\[ F_{bridge} = \frac{1}{2} \rho g A \left( h_{max} - h_{girder} \right) \]

Constant Coefficients:

\[ \gamma = 64 \text{ lb/cubic ft} \]

Vertical Force Calculations:

- Bridge Deck Width = 32.83 ft
- Bridge Deck Length = 78.83 ft
- Area (A) = 2588.36 sq ft
- Vol_bridge = 6933.77 cubic ft
- Vol_trapped air/2 = 1167.17 cubic ft
- n = 12 girders
- Storm Surge Depth + Wave Height = 13.00 ft
  (from NFIP Flood Hazard Assessment Tool)
- Storm Surge Depth = 8.93 ft

- Largest Unbroken Wave: \((0.455 \times \text{storm surge depth})\)
  Wave Height = 4.07 ft
  (above storm surge elevation)

- Equation Values:
  \( \delta z = -0.24 \text{ ft} \)
  (Difference between top of deck and highest point on wave)
- \( F_w = 0.00 \text{ lbs} \)
- \( F_{hydrostatic} = 0.00 \text{ lbs} \)
- \( F_{bridge} = 443761.28 \text{ lbs} \)
- \( F_{entrapment} = 74698.88 \text{ lbs} \)

- Resulting Vertical Force:
  \( F_v = 518.46 \text{ kips} \)
  259.23 tons

- Vertical Resistance: (from hand calculations)
  - Sources of Vertical Resistance:
    - Self Weight = 279.83 kips
      \( W_{self} = 279.83 \text{ kips} \)
    - Drift Bolt Withdrawal Capacity = 69.3 kips
      \( W_{drift} = 69.3 \text{ kips} \)
  - Total Vertical Resistance:
    \( R_v = 349.13 \text{ kips} \)

- Comparison of Vertical Wave Force to Vertical Resistance:
  \( F_v = 518.46 \text{ kips} \)
  \( R_v = 349.13 \text{ kips} \)

| Bridge Condition: | Failure |

Horizontal Force Calculations:

- Railing Height = 0.000 ft
- Deck Height = 0.500 ft
- Girder Height = 1.500 ft
- Bridge Total Height = 2.000 ft
- Elevation to bot. of girder = 11.24 ft
- Elevation to bot. of deck = 12.74 ft

- Wave Height = 4.07 ft
  (above storm surge elevation)

- Equation Values:
  \( \delta z = -0.24 \text{ ft} \)
  (Difference between top of deck and highest point on wave)
- \( F_w = 0.00 \text{ lbs} \)
- \( F_{hydrostatic} = 0.00 \text{ lbs} \)
- \( F_{bridge} = 443761.28 \text{ lbs} \)
- \( F_{entrapment} = 74698.88 \text{ lbs} \)

- Resulting Vertical Force:
  \( F_v = 518.46 \text{ kips} \)
  259.23 tons
Bridge Length = 78.833 ft  Elevation to bot. of deck = 12.74 ft

Ah = 157.67 sq ft  N = 12 girders

● Largest Unbroken Wave: (0.455*storm surge depth)

Wave Height = 4.07 ft  (above storm surge elevation)

● Equation Values:

\[ \eta_{\text{max}} = 4.07 \text{ ft} \quad \text{(Wave height above storm surge elevation)} \]
\[ H_{\text{bridge}} = 2.000 \text{ ft} \]
\[ h = 8.935 \text{ ft} \]
\[ h_{\text{girder}} = 11.240 \text{ ft} \]
\[ h_{\text{deck}} = 12.740 \text{ ft} \]

\[ F_{\text{hydrostatic front}} = 8879.78667 \text{ lbs} \quad \eta_{\text{max}} < h_{\text{deck}} \]
\[ F_{\text{hydrostatic back}} = 0.000 \text{ lbs} \quad \text{SWL} < h_{\text{girder}} \]

● Resulting Vertical Force:

\[ F_h = 8.88 \text{ kips} \quad 4.44 \text{ tons} \]

● Horizontal Resistance: (from hand calculations)

- Sources of Horizontal Resistance:
  - Self Weight = 279830 lbs
  - Girder Seat Interface = Wood to Wood (wet)\[ \mu = 0.2 \]
  - Frictional Resistance = 55966 lbs
  - Drift Bolts Lateral Capacity = 14405.4 lbs

- Total Horizontal Resistance:

\[ R_h = 70.37 \text{ kips} \]

● Comparison of Horizontal Wave Force to Horizontal Resistance:

\[ F_h = 8.88 \text{ kips} < R_h = 70.37 \text{ kips} \]

Bridge Condition: O.K.
Maipalaoa Bridge: (Maili Channel)

Method For Estimating Wave Forces on Bridge Superstructures

McPherson (2008)

Vertical Force Equation:

\[
\begin{align*}
\delta z &= (\kappa - 1)gSy_d d_d \\
F_w &= \rho g A \delta z \\
F_{\text{hydrostatic}} &= \rho g D \delta z \\
F_{\text{bridge}} &= \rho g A \delta z \\
F_{\text{fair entrapment}} &= \rho g A \delta z \\
F_{\text{total}} &= F_{\text{hydrostatic}} + F_{\text{bridge}} + F_{\text{fair entrapment}}
\end{align*}
\]

Horizontal Force Equation:

\[
\begin{align*}
F_{\text{total}} &= F_{\text{hydrostatic}} + F_{\text{bridge}} + F_{\text{fair entrapment}}
\end{align*}
\]

Constant Coefficients:

\[
\gamma = 64 \text{ lb/cubic ft}
\]

**Vertical Force Calculations:**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge Deck Width</td>
<td>64.33 ft</td>
</tr>
<tr>
<td>Bridge Deck Length</td>
<td>100.67 ft</td>
</tr>
<tr>
<td>Water Depth</td>
<td>3.00 ft</td>
</tr>
<tr>
<td>Water surface to bot. of girder</td>
<td>3.50 ft</td>
</tr>
<tr>
<td>Area (A)</td>
<td>6476.2222 sq ft</td>
</tr>
<tr>
<td>Height of girder</td>
<td>3.00 ft</td>
</tr>
<tr>
<td>Vol_bridge</td>
<td>7745.12 cubic ft</td>
</tr>
<tr>
<td>Railing Height</td>
<td>2.00 ft</td>
</tr>
<tr>
<td>Elevation to bot. of girder</td>
<td>6.50 ft</td>
</tr>
<tr>
<td>Elevation to bot. of deck</td>
<td>9.50 ft</td>
</tr>
<tr>
<td>Storm Surge Depth + Wave Height</td>
<td>12.00 ft</td>
</tr>
<tr>
<td>Storm Surge Depth</td>
<td>8.25 ft</td>
</tr>
</tbody>
</table>

**Largest Unbroken Wave:** (0.455*storm surge depth)

Wave Height = 3.75 ft (above storm surge elevation)

**Equation Values:**

\[
\delta z = 2.00 \text{ ft} \quad (\text{Difference between top of deck and highest point on wave})
\]

\[
\begin{align*}
F_w &= 414478.222 \text{ lbs} \\
F_{\text{hydrostatic}} &= 414478.222 \text{ lbs} \\
F_{\text{bridge}} &= 495687.68 \text{ lbs} \\
F_{\text{fair entrapment}} &= 399532.8 \text{ lbs}
\end{align*}
\]

**Resulting Vertical Force:**

\[
F_v = 1309.70 \text{ kips} \quad 654.85 \text{ tons}
\]

**Vertical Resistance:** (from hand calculations)

- **Sources of Vertical Resistance:**
  - Self Weight = 1406690 lbs = 1406.69 kips
  - Total Vertical Resistance:
    - \( R_v = 1406.69 \text{ kips} \)

**Comparison of Vertical Wave Force to Vertical Resistance:**

\[
F_v = 1309.70 \text{ kips} < R_v = 1406.69 \text{ kips}
\]

**Bridge Condition:** O.K.

**Horizontal Force Calculations:**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Railing Height</td>
<td>2.00 ft</td>
</tr>
<tr>
<td>Deck Height</td>
<td>0.50 ft</td>
</tr>
<tr>
<td>Water surface to bot. of girder</td>
<td>3.50 ft</td>
</tr>
<tr>
<td>Girder Height</td>
<td>3.00 ft</td>
</tr>
<tr>
<td>Bridge Total Height</td>
<td>5.500 ft</td>
</tr>
<tr>
<td>Bridge Length</td>
<td>100.667 ft</td>
</tr>
<tr>
<td>Elevation to bot. of girder</td>
<td>6.50 ft</td>
</tr>
<tr>
<td>Elevation to bot. of deck</td>
<td>9.50 ft</td>
</tr>
</tbody>
</table>
\[ \text{Ah} = 553.67 \text{ sq ft} \]
\[ \text{N} = 16 \text{ girders} \]

- **Largest Unbroken Wave:** \((0.455 \times \text{storm surge depth})\)

\[ \text{Wave Height} = 3.75 \text{ ft} \quad \text{(above storm surge elevation)} \]

- **Equation Values:**
  \[ \eta_{\text{max}} = 3.75 \text{ ft} \quad \text{(Wave height above storm surge elevation)} \]
  \[ H_{\text{bridge}} = 5.50 \text{ ft} \]
  \[ h = 8.25 \text{ ft} \]
  \[ h_{\text{girder}} = 6.50 \text{ ft} \]
  \[ h_{\text{deck}} = 9.50 \text{ ft} \]

\[ F_{\text{hydrostatic\_front}} = 97445.3333 \text{ lbs} \]
\[ \eta_{\text{max}} < h_{\text{deck}} \]

\[ F_{\text{hydrostatic\_back}} = 9836.296 \text{ lbs} \]
\[ \text{SWL} > h_{\text{girder}} \]

- **Resulting Vertical Force:**

\[ F_{h} = 87.61 \text{ kips} \quad 43.80 \text{ tons} \]

- **Horizontal Resistance:** \((\text{from hand calculations})\)
  - **Sources of Horizontal Resistance:**
    - Self Weight = 140669 lbs
    - Girder Seat Interface = neoprene pad
      \[ \mu_s = 0.1 \]
    - Frictional Resistance = 140669 lbs
    - Beam Web Flexural Capacity = 92016 lbs
  - **Total Horizontal Resistance:**
    \[ R_h = 232.69 \text{ kips} \]

- **Comparison of Horizontal Wave Force to Horizontal Resistance:**

\[ F_{h} = 87.61 \text{ kips} \quad < \quad R_h = 232.69 \text{ kips} \quad \text{Bridge Condition:} \quad \text{O.K.} \]

---

C-12
Moanalua Stream Bridge: (Spans 3, 4, 5 & 6)  
*Note: The calculations that follow are for one span only, therefore bridge volumes are divided by 8.

McPherson (2008)

**Vertical Force Equation:**

\[
\begin{align*}
F_{\text{vertical}} &= \gamma (V_{\text{bridge}} + \frac{V_{\text{trapped air}}}{2}) \\
F_{\text{hydrostatic}} &= \rho g \frac{V_{\text{bridge}}}{8} \\
F_{\text{installation}} &= \frac{V_{\text{bridge}}}{8} \\
\end{align*}
\]

**Horizontal Force Equation:**

\[
\begin{align*}
F_{\text{horizontal}} &= \frac{\rho g V_{\text{bridge}}}{8} \\
F_{\text{hydrostatic}} &= \frac{\rho g V_{\text{bridge}}}{8} \\
F_{\text{installation}} &= \frac{\rho g V_{\text{bridge}}}{8} \\
\end{align*}
\]

*Constant Coefficients:*

\[
\gamma = 64 \text{ lb/cubic ft} 
\]

**Vertical Force Calculations:**

- Bridge Deck Width = 64.33 ft
- Bridge Deck Length = 27.00 ft
- Area (A) = 1737 sq ft
- \(n = 9\) girders
- Vol_bridge = 2737.62 cubic ft
- Vol_trapped air/2 = 1390.11 cubic ft
- Decks Height = 0.667 ft
- Elevation to bot. of girder = 6.50 ft
- Elevation to bot. of deck = 8.33 ft
- Storm Surge Depth + Wave Height = 10.00 ft
- Storm Surge Depth = 6.87 ft
- Largest Unbroken Wave: \((0.455*\text{storm surge depth})\) Wave Height = 3.13 ft
- \(\delta z = 1.00\) ft
- \(F_w = 55584\) lbs
- \(F_{\text{hydrostatic}} = 55584\) lbs
- \(F_{\text{bridge}} = 175207.537\) lbs
- \(F_{\text{entrapment}} = 88967\) lbs
- \(F_v = 319.76\) kips
- \(R_v = 575.67\) kips
- Comparison of Vertical Wave Force to Vertical Resistance:
  \[F_v = 319.76\text{ kips} < R_v = 575.67\text{ kips}\]

**Horizontal Resistance: (from hand calculations)**

- Sources of Vertical Resistance:
  - Self Weight = \(417.22\) kips
  - Pier Cap Weight = \(158.4\) kips
- Total Vertical Resistance:
  \[R_v = 575.67\] kips

**Comparison of Vertical Wave Force to Vertical Resistance:**

- \(F_v = 319.76\) kips
- \(R_v = 575.67\) kips

**Bridge Condition:**

- O.K. C-13

---

C-13
Bridge Length = 27.000 ft  
Elevation to bot. of deck = 8.33 ft  
Ah = 168.19 sq ft  
N = 9 girders  

● Largest Unbroken Wave: (0.455*storm surge depth)  
Wave Height = 3.13 ft  
(above storm surge elevation)  

● Equation Values:  
\[ \eta_{\text{max}} = 3.13 \text{ ft} \]  
\[ H_{\text{bridge}} = 6.23 \text{ ft} \]  
\[ h = 6.87 \text{ ft} \]  
\[ h_{\text{girder}} = 6.50 \text{ ft} \]  
\[ h_{\text{deck}} = 8.33 \text{ ft} \]  

\[ F_{\text{hydrostatic\_front}} = 18837 \text{ lbs} \quad \eta_{\text{max}} < h_{\text{deck}} \]  
\[ F_{\text{hydrostatic\_back}} = 120.112 \text{ lbs} \quad \text{SWL} > h_{\text{girder}} \]  

● Resulting Vertical Force:  
\[ F_h = 18.72 \text{ kips} \quad 9.36 \text{ tons} \]  

● Horizontal Resistance: (from hand calculations)  

\bullet Sources of Horizontal Resistance:  
Self Weight = 417270 lbs  
Girder Seat Interface = concrete to concrete  
\[ \mu_s = 0.8 \]  
Frictional Resistance = 333816 lbs  
Dowel Shear Friction = 1592680 lbs  

\bullet Total Horizontal Resistance:  
\[ R_h = 1926.46 \text{ kips} \]  

● Comparison of Horizontal Wave Force to Horizontal Resistance:  
\[ F_h = 18.72 \text{ kips} \quad < \quad R_h = 1926.46 \text{ kips} \]  

<table>
<thead>
<tr>
<th>Bridge Condition:</th>
<th>O.K.</th>
</tr>
</thead>
</table>

*Note: Spans 2 & 7 are the weakest, as they are only tied at one pier, the other side of each span is on a concrete to concrete interface.
Kaliihi Stream Bridge: (Typical Span)

Method For Estimating Wave Forces on Bridge Superstructures

*Note: The calculations that follow are for one span only, therefore bridge volumes are divided by 7.

McPherson (2008)

Vertical Force Equation:

\[
\begin{align*}
\text{Vertical Force} & = \begin{cases} 
\text{Fv}_{\text{total}} = \text{Fhydrostatic} + \text{Fbridge} + \text{Fentrapment} \\
\text{Fhydrostatic} = \gamma A (z_0 - h) \\
\text{Fbridge} = \gamma V_{\text{bridge}} \\
\text{Fentrapment} = \gamma (n-1)S_y h_d g_d 
\end{cases}
\end{align*}
\]

Horizontal Force Equation:

\[
\begin{align*}
\text{Horizontal Force} & = \begin{cases} 
\text{Fh}_{\text{total}} = \text{Fhydrostatic} + \text{Fbridge} \\
\text{Fhydrostatic} = \gamma A (d - h) \\
\text{Fbridge} = \gamma V_{\text{bridge}} 
\end{cases}
\end{align*}
\]

Constant Coefficients:

\[
\begin{align*}
\gamma & = 64 \text{ lb/cubic ft} \\
\end{align*}
\]

Vertical Force Calculations:

- Bridge Deck Width = 88.33 ft
- Bridge Deck Length = 27.00 ft
- Area (A) = 2385 sq ft
- Vol_bridge = 1684.43 cubic ft
- Water Depth = 2.50 ft
- Height of girder = 1.83 ft
- Elevation to bot. of girder = 6.50 ft
- Elevation to bot. of deck = 8.33 ft
- Storm Surge Depth + Wave Height = 10.00 ft
- Storm Surge Depth = 6.87 ft
- Largest Unbroken Wave: \((0.455 \ast \text{storm surge depth})\)
- Wave Height = 3.13 ft
- Wave Height (above storm surge elevation)
- \(\delta z = 1.00 \text{ ft}\) (Difference between top of deck and highest point on wave)
- \(F_{\text{w}} = 76320 \text{ lbs}\)
- \(F_{\text{hydrostatic}} = 76320 \text{ lbs}\)
- \(F_{\text{bridge}} = 235802.236 \text{ lbs}\)
- \(F_{\text{entrapment}} = 62710.784 \text{ lbs}\)
- Resulting Vertical Force:
  \[F_v = 374.83 \text{ kips} = 187.42 \text{ tons}\]

Vertical Resistance: (from hand calculations)

- Sources of Vertical Resistance:
  - Self Weight = 565087.1 lbs = 565.09 kips
- Total Vertical Resistance
  \[R_v = 565.09 \text{ kips}\]

Comparison of Vertical Wave Force to Vertical Resistance:

- \(F_v = 374.83 \text{ kips} < R_v = 565.09 \text{ kips}\)
- Bridge Condition: O.K.

Horizontal Force Calculations:

- Railing Height = 3.729 ft
- Deck Height = 0.667 ft
- Girder Height = 1.833 ft
- Bridge Total Height = 6.229 ft
- Bridge Length = 27.000 ft
- Water Depth = 2.50 ft
- Water surface to bot. of girder = 4.00 ft
- Storm Surge Depth + Wave Height = 10.00 ft (from NFIP Flood Hazard Assessment Tool)
- Storm Surge Depth = 6.87 ft
- Wave Height = 3.13 ft (above storm surge elevation)
- \(\delta z = 1.00 \text{ ft}\) (Difference between top of deck and highest point on wave)
- \(F_{\text{w}} = 76320 \text{ lbs}\)
- \(F_{\text{hydrostatic}} = 76320 \text{ lbs}\)
- \(F_{\text{bridge}} = 235802.236 \text{ lbs}\)
- \(F_{\text{entrapment}} = 62710.784 \text{ lbs}\)
- Resulting Horizontal Force:
  \[F_h = 2517.80 \text{ kips} = 1258.90 \text{ tons}\]

Horizontal Resistance: (from hand calculations)

- Sources of Horizontal Resistance:
  - Self Weight = 565087.1 lbs = 565.09 kips
- Total Horizontal Resistance
  \[R_h = 565.09 \text{ kips}\]

Comparison of Horizontal Wave Force to Horizontal Resistance:

- \(F_h = 2517.80 \text{ kips} = R_h = 565.09 \text{ kips}\)
- Bridge Condition: O.K.
Ah = 168.19 sq ft
N = 13 girders

- Largest Unbroken Wave: (0.455*storm surge depth)
  Wave Height = 4.225 ft (above storm surge elevation)

- Equation Values:
  \( \eta_{\text{max}} = 4.23 \text{ ft} \) (Wave height above storm surge elevation)
  \( H_{\text{bridge}} = 6.23 \text{ ft} \)
  \( h = 6.87 \text{ ft} \)
  \( h_{\text{girder}} = 6.50 \text{ ft} \)
  \( h_{\text{deck}} = 8.33 \text{ ft} \)

\( F_{\text{hydrostatic front}} = 24745.6407 \text{ lbs} \)
\( \eta_{\text{max}} < h_{\text{deck}} \)
\( F_{\text{hydrostatic back}} = 120.112 \text{ lbs} \) SWL > \( h_{\text{girder}} \)

- Resulting Vertical Force:
  \( F_h = 24.63 \text{ kips} \) 12.31 tons

- Horizontal Resistance: (from hand calculations)
  - Sources of Horizontal Resistance:
    Self Weight = 565087.1 lbs
    Girder Seat Interface = concrete to concrete \( \mu_s = 0.8 \)
    Frictional Resistance = 452069.7 lbs
  
  - Total Horizontal Resistance:
    \( Rh = 452.07 \text{ kips} \)

- Comparison of Horizontal Wave Force to Horizontal Resistance:
  \( F_h = 24.63 \text{ kips} \) < \( Rh = 452.07 \text{ kips} \)

Bridge Condition: O.K.
Nimitz Highway Slip Cover #2:
Method For Estimating Wave Forces on Bridge Superstructures
McPherson (2008)

**Vertical Force Equation:**

\[
F_v = \frac{1}{2} \rho \mu (h + h_{model}) d \Delta z
\]

**Horizontal Force Equation:**

\[
F_h = \rho \mu (h + h_{model}) d
\]

**Constant Coefficients:**

\[
\gamma = 64 \text{ lb/cubic ft}
\]

**Vertical Force Calculations:**

<table>
<thead>
<tr>
<th>Bridge Deck Width (1)</th>
<th>67 ft</th>
<th>Water Depth = 5.33 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge Deck Width (2)</td>
<td>53.00 ft</td>
<td>Water surface to bot. of girder = 4.50 ft</td>
</tr>
<tr>
<td>Bridge Deck Length</td>
<td>178.41 ft</td>
<td></td>
</tr>
</tbody>
</table>

\[
\text{Area (A)} = 10704.6 \text{ sq ft}
\]

\[
\text{Vol}_{\text{bridge}} = 13797 \text{ cubic ft}
\]

\[
\text{Vol}_{\text{trapped air/2}} = 0 \text{ cubic ft}
\]

\[
n = 11 \text{ girders}
\]

\[
\text{Elevation to bot. of girder} = 9.83 \text{ ft}
\]

\[
\text{Elevation to bot. of deck} = 12.33 \text{ ft}
\]

\[
\text{Storm Surge Depth + Wave Height} = 8.00 \text{ ft (from NFIP Flood Hazard Assessment Tool)}
\]

\[
\text{Storm Surge Depth} = 5.50 \text{ ft}
\]

\[
\text{Largest Unbroken Wave: (0.455*storm surge depth)}
\]

\[
\text{Wave Height} = 2.50 \text{ ft (above storm surge elevation)}
\]

**Equation Values:**

\[
\delta z = -5.63 \text{ ft (Difference between top of deck and highest point on wave)}
\]

\[
F_w = 0 \text{ lbs}
\]

\[
F_{\text{hydrostatic}} = 0 \text{ lbs}
\]

\[
F_{\text{bridge}} = 0 \text{ lbs (bridge is not submerged, water level is too low)}
\]

\[
F_{\text{air_encroachment}} = 0 \text{ lbs}
\]

**Resulting Vertical Force:**

\[
F_v = 0.00 \text{ kips, 0.00 tons}
\]

**Vertical Resistance: (from hand calculations)**

- **Sources of Vertical Resistance:**
  - Self Weight = 2069453 lbs, 2069.453 kips

- **Total Vertical Resistance:**
  - \( R_v = 2069.453 \text{ kips} \)

**Comparison of Vertical Wave Force to Vertical Resistance:**

\[
F_v = 0.00 \text{ kips} < R_v = 2069.453 \text{ kips}
\]

**Bridge Condition:**

\[
- \text{O.K.}
\]

**Horizontal Force Calculations:**

<table>
<thead>
<tr>
<th>Railing Height = 1.667 ft</th>
<th>Water Depth = 5.33 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck Height = 1.292 ft</td>
<td>Water surface to bot. of girder = 4.50 ft</td>
</tr>
</tbody>
</table>
| Girder Height = 2.500 ft  |}

C-17
Bridge Total Height = 5.458 ft  Elevation to bot. of girder = 9.83 ft
Bridge Length = 178.410 ft  Elevation to bot. of deck = 12.33 ft

Ah = 973.82 sq ft  N = 11 girders

● Largest Unbroken Wave: \((0.455 \times \text{storm surge depth})\)

Wave Height = 2.50 ft  \((\text{above storm surge elevation})\)

● Equation Values:

\[ \eta_{\text{max}} = 2.50 \text{ ft} \]  
\[ H_{\text{bridge}} = 5.46 \text{ ft} \]  
\[ h = 8.00 \text{ ft} \]  
\[ h_{\text{girder}} = 9.83 \text{ ft} \]  
\[ h_{\text{deck}} = 12.33 \text{ ft} \]

\[ F_{\text{hydrostatic\_front}} = 0 \text{ lbs} \]  
\[ \eta_{\text{max}} < h_{\text{deck}} \]  \((\text{bridge is not submerged})\)

\[ F_{\text{hydrostatic\_back}} = 0.000 \text{ lbs} \]  
\[ \text{SWL} < h_{\text{girder}} \]

● Resulting Vertical Force:

\[ F_h = 0.00 \text{ kips} \]  
\[ 0.00 \text{ tons} \]

● Horizontal Resistance: \((\text{from hand calculations})\)

- **Sources of Horizontal Resistance:**
  - Self Weight = 2069453 lbs
  - Girder Seat Interface = Concrete to Concrete  
    \[ \mu \] = 0.8
  - Frictional Resistance = 1655562 lbs

- **Total Horizontal Resistance:**

\[ R_h = 1655.56 \text{ kips} \]

● Comparison of Horizontal Wave Force to Horizontal Resistance:

<table>
<thead>
<tr>
<th>(F_h)</th>
<th>(R_h)</th>
<th>Bridge Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>1655.56</td>
<td>O.K.</td>
</tr>
</tbody>
</table>

C-18
Nimitz Highway Slip Cover #3:
Method For Estimating Wave Forces on Bridge Superstructures

McPherson (2008)

Vertical Force Equation:

\[ F_{\text{vertical}} = \gamma V_{\text{bridge}} - F_{\text{bridge}} - F_{\text{fair\textunderscore}\text{entrapment}} \]

\[ F_{\text{vertical\textunderscore}\text{stability}} = \gamma \left( V_{\text{bridge}} - V_{\text{trapped \ air}} / 2 \right) \]

Horizontal Force Equation:

\[ F_{\text{horizontal}} = F_{\text{hydrostatic\textunderscore}\text{wave}} + F_{\text{hydrostatic\textunderscore}\text{bridge}} + F_{\text{fair\textunderscore}\text{entrapment}} \]

\[ F_{\text{wave\textunderscore}\text{breaking}} = \frac{1}{2} \rho c^2 \cdot A \cdot (h - h_{\text{model}}) \]

\[ F_{\text{bridge\textunderscore}\text{breaking}} = \frac{1}{2} \rho c^2 \cdot A \cdot (h - h_{\text{model}}) \]

\[ F_{\text{fair\textunderscore}\text{entrapment}} = (k - 1) \rho c^2 A \]

Constant Coefficients:

\[ \gamma = 64 \text{ lb/cubic ft} \]

Vertical Force Calculations:

<table>
<thead>
<tr>
<th>Bridge Deck Width (1)</th>
<th>29 ft</th>
<th>Water Depth = 5.33 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge Deck Width (2)</td>
<td>41.00 ft</td>
<td>Water surface to bot. of girder = 4.50 ft</td>
</tr>
<tr>
<td>Bridge Deck Width (3)</td>
<td>32.50 ft</td>
<td></td>
</tr>
<tr>
<td>Bridge Deck Length (1)</td>
<td>162.00 ft</td>
<td>Height of girder = 2.50 ft</td>
</tr>
<tr>
<td>Bridge Deck Length (2)</td>
<td>78.00 ft</td>
<td>Deck Height = 1.292 ft</td>
</tr>
<tr>
<td>( Av = 8536.5 \text{ sq ft} )</td>
<td>Railing Height = 1.667 ft</td>
<td></td>
</tr>
<tr>
<td>( V_{\text{bridge}} = 14499 \text{ cubic ft} )</td>
<td>Elevation to bot. of girder = 9.83 ft</td>
<td></td>
</tr>
<tr>
<td>( V_{\text{trapped \ air}} / 2 = 0 \text{ cubic ft} )</td>
<td>Elevation to bot. of deck = 12.33 ft</td>
<td></td>
</tr>
<tr>
<td>( n = 13 \text{ girders} )</td>
<td>Storm Surge Depth + Wave Height = 8.00 ft</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Storm Surge Depth = 5.50 ft</td>
<td></td>
</tr>
</tbody>
</table>

\[ \delta z = -5.63 \text{ ft} \] (Difference between top of deck and highest point on wave)

\[ F_w = 0 \text{ lbs} \]

\[ F_{\text{hydrostatic}} = 0 \text{ lbs} \]

\[ F_{\text{bridge}} = 0 \text{ lbs} \] (bridge is not submerged, water level is too low)

\[ F_{\text{fair\textunderscore}\text{entrapment}} = 0 \text{ lbs} \]

Resulting Vertical Force:

\[ F_v = 0.00 \text{ kips} \]

Vertical Resistance: (from hand calculations)

<table>
<thead>
<tr>
<th>Sources of Vertical Resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Self Weight = 2069453 lbs</td>
</tr>
<tr>
<td>Total Vertical Resistance</td>
</tr>
<tr>
<td>( R_v = 2069.453 \text{ kips} )</td>
</tr>
</tbody>
</table>

Comparison of Vertical Wave Force to Vertical Resistance:

\[ F_v = 0.00 \text{ kips} < R_v = 2069.453 \text{ kips} \]

Horizontal Force Calculations:

<table>
<thead>
<tr>
<th>Railing Height = 1.667 ft</th>
<th>Water Depth = 5.33 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck Height = 1.292 ft</td>
<td>Water surface to bot. of girder = 4.50 ft</td>
</tr>
<tr>
<td>Girder Height = 2.500 ft</td>
<td></td>
</tr>
</tbody>
</table>

Bridge Total Height = 5.458 ft | Elevation to bot. of girder = 9.83 ft
Bridge Length = 240.00 ft  
Elevation to bot. of deck = 12.33 ft

Ah = 1310.00 sq ft  
N = 13 girders

● Largest Unbroken Wave: (0.78*storm surge depth)

Wave Height = 6.39 ft  
(above storm surge elevation)

● Equation Values:

\[
\eta_{\text{max}} = 6.39 \text{ ft} \quad (\text{Wave height above storm surge elevation}) \\
H_{\text{bridge}} = 5.46 \text{ ft} \\
h = 8.00 \text{ ft} \\
h_{\text{girder}} = 9.83 \text{ ft} \\
h_{\text{deck}} = 12.33 \text{ ft}
\]

\[F_{\text{hydrostatic\_front}} = 0 \text{ lbs} \quad \eta_{\text{max}} < h_{\text{deck}} \quad \text{(bridge is not submerged)}\] 

\[F_{\text{hydrostatic\_back}} = 0.000 \text{ lbs} \quad \text{SWL} < h_{\text{girder}}\]

● Resulting Vertical Force:

\[F_h = 0.00 \text{ kips} \quad 0.00 \text{ tons}\]

● Horizontal Resistance: (from hand calculations)

- **Sources of Horizontal Resistance:**
  - Self Weight = 2180023 lbs
  - Girder Seat Interface = Concrete to Concrete  
    \[\mu_s = 0.8\]  
  - Frictional Resistance = 1744018 lbs

- **Total Horizontal Resistance:**
  \[R_h = 1744.02 \text{ kips}\]

● Comparison of Horizontal Wave Force to Horizontal Resistance:

\[F_h = 0.00 \text{ kips} \quad < \quad R_h = 1744.02 \text{ kips}\]

<table>
<thead>
<tr>
<th>Bridge Condition:</th>
<th>O.K.</th>
</tr>
</thead>
</table>

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Appendix D: AASHTO Guide Specifications
Calculations
### Average Water Depth Over Fetch Length:

<table>
<thead>
<tr>
<th>Location</th>
<th>Average Water Depth (ft)</th>
<th>Total Average Water Depth (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Kulōnou Bridge:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>105</td>
<td>43</td>
<td>27</td>
</tr>
<tr>
<td><strong>Kahalu Stream Bridge:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>32</td>
<td>30</td>
<td>14</td>
</tr>
<tr>
<td><strong>New South Punalu’u Bridge:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>238</td>
<td>135</td>
<td>53</td>
</tr>
<tr>
<td><strong>Moanalua Bridge:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>8</td>
<td>7</td>
</tr>
<tr>
<td><strong>Kalāhe Bridge:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>72</td>
<td>53</td>
<td>35</td>
</tr>
<tr>
<td><strong>New Makaha Stream #3A Bridge:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>72</td>
<td>53</td>
<td>35</td>
</tr>
<tr>
<td><strong>Old Makaha Stream #3A Bridge:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>72</td>
<td>53</td>
<td>35</td>
</tr>
<tr>
<td><strong>Maipalāoa Bridge:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>40</td>
<td>35</td>
<td>29</td>
</tr>
<tr>
<td><strong>Moanalua Bridge:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.50</td>
<td>5.50</td>
<td>5.50</td>
</tr>
<tr>
<td><strong>Nimitz Hwy. Slip Cover #2:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.50</td>
<td>5.50</td>
<td>5.50</td>
</tr>
<tr>
<td><strong>Nimitz Hwy. Slip Cover #3:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.50</td>
<td>5.50</td>
<td>5.50</td>
</tr>
</tbody>
</table>
Kuliouou Bridge:
Method For Estimating Wave Forces on Bridge Superstructures

**AASHTO (2008)**

- **50 year wind speed = 134.00 ft/sec**
- **100 year wind speed = 164.78 ft/sec**
- **Wind Period (t) = 3 sec**
- **W = 68.75 ft**
- **Constant Coefficients:**
  - **50 year wind speed = 105 mph**
  - **W* = 66.25 ft**
  - **Specific weight water = 0.064 kip/cubic ft**
  - **r = 2.71 ft**
  - **g = 32.2 ft/sec^2**
  - **η**

### Bridge Properties:
- **Hmax = 3.58 ft**
- **W = 68.75 ft**
- **D = 3.94 ft**
- **Girder to Girder Width = 66.25 ft**
- **Deck Thickness = 0.67 ft**
- **Height of railing = 2.71 ft**
- **Zc = -0.56 ft**
- **Storm Water Level = 4.94 ft**
- **Av = 3327.21 sq ft**
- **N = 12 girders**
- **ds = 5.50 ft**

### Design Wave Parameters:
- **(AASHTO Sec 6.2.2.4)**

<table>
<thead>
<tr>
<th>Determination of wave period:</th>
<th>50 year wind speed = 134.00 ft/sec</th>
<th>Gust Period (t) = 3 sec</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>(g<em>d)/(Ut</em>)^2</strong></td>
<td>0.010</td>
<td></td>
</tr>
<tr>
<td><strong>Tp</strong></td>
<td>9.86 sec</td>
<td></td>
</tr>
</tbody>
</table>

- **Determination of time duration to develop fetch limited waves:**
  - **t = 6044.74 sec > 3600 sec**

- **U_1hr = 170.55 ft/sec**
  - To compute U_1hr use eq 6.2.2.4-5

#### Iteration Process:
- **First Iteration:**
  - **Ut = 169.45 ft/sec**
  - **Tp = 9.95 sec**

#### Fifth Iteration:
  - **Ut* = 297.47 ft/sec**
  - **Tp = 9.95 sec**
  - **t = 5895.41 sec**

### Wave height and wave length:
- **Hs = 12.37 ft**
- **λ = 132.28 ft**

### Maximum Wave Height:
- **Hmax = 22.26 ft**

### Resulting Storm Wave Properties:
- **Tp = 9.95 sec**
Maximum Quasi-Static Vertical Force and Associated Forces and Moments: (AASHTO Sec 6.1.2.2)

Maximum Quasi-Static Vertical Force: (AASHTO Sec 6.1.2.2.1)

Determination of Forces and moments: (eq 6.1.2.2.1-1)

- $W_{hat} = 86.86$ ft
- $W_{hat} / W = 1.26 > 0.15$ Therefore $W_{hat} = W_{hat}$
- $\eta_{max} - Zc = 3.96 < \eta_{max} = 3.67$
- $W_{hat} = 86.86$ ft

For Girders Spaces: (eq 6.1.2.2.1-1)

- $b0 = -0.888$
- $b1 = 56.16$
- $b2 = 0.0538$
- $b3 = -192.89$

Tapped Air Factor:

- $A_{air} = 0.005755$
- $B_{air} = 0.394561$

$\eta_{max} - Zc$ / $d_{g} = 1.020833 > 1$

Assume 50% air pocket:

- $\%A_{air} = 50$
- $TAF = 0.662525 < 1$ (O.K.)
- $TAF = 0.662525$

Quasi-Static Vertical Force: (eq 6.1.2.2.1-1)

- $Fv_{max} = 2.9180604$ kip/ft
- $Length of Bridge = 48.40$ ft
- $Fv_{max}$ Total = 141.22 kips, 70.61 tons

Associated Vertical Slamming Force: (AASHTO Sec 6.1.2.2.2)

- $B = -1.280133$
- $Zc / \eta_{max} = -0.223776 < 0$
- $A = 0.0282657$

Vertical Slamming Force: (eq 6.1.2.2.2-1)

- $Fs = 2.3523513$ kip/ft
- $Length of Bridge = 48.40$ ft
- $Fs$ Total = 113.84 kips, 56.92 tons

Associated Horizontal Quasi Static Wave Force: (AASHTO Sec 6.1.2.2.3)

- Note: Girders used on the Kuliouou Bridge are similar to the AASHTO Type III)

From Table 6.1.2.2.3-1: (for AASHTO Type III Girders)

- $a0 = -1.0867$
- $a1 = -1.4792$
- $a2 = 0.5367$
- $a3 = -0.0877$
- $x = 0.480922$
- $y = 0.023026$

Horizontal Quasi-Static Wave Force: (eq 6.1.2.2.3)
Fl-av = 0.1231737 kip/ft
Length of Bridge = 48.40 ft
Fl-av Total = 5.96 kips 2.98 tons

**Associated Moment about the Trailing Edge Due to the Quasi-static and Slamming Forces (AASHTO Sec 6.1.2.2.4)**

- For Girder Spans
  - a_m = 0.8463125 ft
  - b_m = -0.0503888 ft
  - c_m = -0.0045918 ft
  - W' = 2.50 ft
  - W* = 66.25 ft

  Associated Moment about Trailing Edge: (eq 6.1.2.4-4)
  - M_t-av = 108.58 (kip/ft)-ft

  Length of Bridge = 48.40 ft
  - M_t-av Total = 5254.72 kip-ft 2627.36 tons-ft

**Resulting Quasi-Static Vertical Force and Associated Forces and Moments (AASHTO Sec 6.1.2.2)**

- Fv-max Total = 141.22 kips (Quasi-Static Vertical Force)
- Fs Total = 113.84 kips (Vertical Slamming Force)
- Fh-av Total = 5.96 kips (Quasi-Static Horizontal Force)
- Mt-av = 5254.72 kip-ft (Associated Moment about Trailing Edge)

**Maximum Horizontal Wave Force and Associated Forces and Moments (AASHTO Sec 6.1.2.3)**

- Maximum Horizontal Wave Force:
  - ω check: (eq 6.1.2.3.1-3 or eq 6.1.2.3.1-4)
    - check = 109.57 > W = 68.75 ft Use eq 6.1.2.3.1-4 for omega
  - ω = 68.75 ft
  - Reference Horizontal Force: (eq 6.1.2.3.1-2)
    - F*h-max = 2.44 kip/ft
  - Horizontal Wave Force: (eq 6.1.2.3.1-1)
    - Fh-max = 0.5705167 kip/ft

  Length of Bridge = 48.40 ft
  - Fh-max Total = 27.61 kip 13.81 tons

**Associated Quasi-Static Vertical Force (AASHTO Sec 6.1.2.3.2)**

- α check: (eq 6.1.2.3.2-3 or eq 6.1.2.3.2-4)
  - check = 33.41 < W = 68.75 ft Use eq 6.1.2.3.2-3 for alpha
  - α = 33.41 ft
  - Reference Vertical Force: (eq 6.1.2.3.2-2)
    - F*v-ah = 6.55 kip/ft
  - Quasi-Static Vertical Wave Force: (eq 6.1.2.3.2-1)
    - Fv-ah = 3.6122748 kip/ft

  Length of Bridge = 48.40 ft
  - Fv-ah Total = 174.82 kip 87.41 tons

**Associated Vertical Slamming Forces (AASHTO Sec 6.1.2.3.3)**

- Note: Slamming forces calculated using the same method as AASHTO sec 6.1.2.2.2

  Vertical Slamming Force: (eq 6.1.2.2.2-1)
  - Fs = 2.3525137 kip/ft

  Length of Bridge = 48.40 ft
**Associated Moment About Trailing Edge:** (AASHTO Sec 6.1.2.3.4)

- Reference Moment:
  \[ M\text{-ah} = 277.01574 \text{ (kip/ft)} \cdot \text{ft} \]

- Associated Moment about Trailing Edge:
  \[ M\text{-ah} = 316.06886 \text{ (kip/ft)} \cdot \text{ft} \]

Length of Bridge: 48.40 ft

- Total Associated Moment:
  \[ M\text{-ah Total} = 15296.27 \text{ kip-ft} \]

**Resulting Maximum Horizontal Wave Force and Associated Forces and Moments:** (AASHTO Sec 6.1.2.3)

- Maximum Horizontal Wave Force:
  \[ F\text{h-max Total} = 27.61 \text{ kips} \]

- Quasi-Static Vertical Force:
  \[ F\text{v-ah Total} = 174.82 \text{ kips} \]

- Vertical Slamming Force:
  \[ F\text{s Total} = 113.84 \text{ kips} \]

- Associated Moment about Trailing Edge:
  \[ M\text{-ah Total} = 15296.27 \text{ kip-ft} \]

**Current Loads on Superstructure:** (AASHTO Sec 6.1.2.4)

*Note: Current loads are not considered in this study.

- \[ F\text{hc} = 0 \text{ kips} \]
Kahaluu Bridge: Method for Estimating Wave Forces on Bridge Superstructures

AASHTO (2008)

**Determination Coefficients:**
- 50 year wind speed = 105.00 mph
- specific weight water = 62.2 lbs/cubic ft
- g = 32.2 ft/sec^2

**Wave Calculations:**
- **Bridge Properties:**
  - Bridge Deck Width = 46.00 ft
  - Bridge Deck Length = 318.00 ft
  - Girder to Girder Width = 41.92 ft
  - Height of girder = 4.50 ft
  - Deck Thickness = 0.50 ft
  - Height of railing = 2.13 ft
  - Zc = -0.25 ft
  - Storm Water Level = 10.75 ft
  - Storm Surge + Local Wind Setup = 11.00 ft
- **Water surface to bot. of girder = 5.00 ft**
- **Water surface to bot. of deck = 15.25 ft**

**Design Wave Parameters:**
- 50 year wind speed = 154.00 ft/sec
- Gust Period (t) = 3 sec
- 100 year wind speed = 164.78 ft/sec
- Ut* = 287.33 ft/sec
- Formula from NFIP Flood Hazard Assessment Tool
- d = 16.38 ft (average water depth over fetch length)
- Fetch Length = 4767610.00 ft
- [(g*d)/(Ut*)^2] = 0.006
- Tp = 8.37 sec
- t = 4127.98 sec > 3600 sec
- U_1hr = 166.27 ft/sec
- To compute U_1hr use eq 6.2.2.4-5

**Iteration Process:** (until ‘t’ converges)

- **Second Iteration:**
  - Ut* = 289.75 ft/sec
  - [(g*d)/(Ut*)^2] = 0.006
  - Tp = 8.39 sec
  - t = 4102.79 sec

- **Third Iteration:**
  - Ut* = 289.78 ft/sec
  - [(g*d)/(Ut*)^2] = 0.006
  - Tp = 8.39 sec
  - t = 4102.50 sec

- **Fourth Iteration:**
  - Ut* = 289.78 ft/sec
  - [(g*d)/(Ut*)^2] = 0.006
  - Tp = 8.39 sec
  - t = 4102.50 sec

- **Fifth Iteration:**
  - Ut* = 289.78 ft/sec
  - [(g*d)/(Ut*)^2] = 0.006
  - Tp = 8.39 sec
  - t = 4102.50 sec

- **Sixth Iteration:**
  - Ut* = 289.78 ft/sec
  - [(g*d)/(Ut*)^2] = 0.006
  - Tp = 8.39 sec
  - t = 4102.50 sec

- **(the value of ‘t’ has converged)**

**Wave height and wave length:**
- Hs = 8.72 ft
- Wave Length (λ) = 156.93 ft
- Hrms = 1.570 ft

**Maximum Wave Height:**
- Hmax = 15.70 ft
- Hmax < 0.65*ds = 7.15 ft
- Hmax < λ/7.0 = 22.43 ft
- Therefore Hmax = 7.15 ft

**Resulting Storm Wave Properties:**
- Tp = 8.39 sec
Maximum Quasi-Static Vertical Force and Associated Forces and Moments (AASHTO Sec 6.1.2.2)

- **Maximum Quasi-Static Vertical Force (AASHTO Sec 6.1.2.2.1)**
  - **Determination of Pinch-up parameters** (eq 6.1.2.2.1-1)
    - \( W_{\hat{h}} = 83.95 \) ft
    - \( W_{\hat{h}} / W = 1.83 > 0.15 \) Therefore \( W_{\hat{h}} / W = W_{\hat{h}} \)
    - \( \eta_{\text{max}} - Z_c = 5.26 > 0.0 \)
    - \( b = 1.00 \)
    - \( \beta = 1.00 \)
    - \( x = 0.0455613 \)
    - \( y = 0.534895 \)

- **For Girders Spans** (eq 6.1.2.2.1 a)
  - \( b_0 = -1.038 \) \( b_4 = -0.00057 \)
  - \( b_1 = 55.89 \) \( b_5 = 0.22 \)
  - \( b_2 = 0.058 \) \( b_6 = 11.01 \)
  - \( b_3 = -192.5416 \)

- **Tapped Air Factor**
  - \( A_{\text{air}} = 0.005953 \)
  - \( B_{\text{air}} = 0.4099234 \)

- **\( (\eta_{\text{max}} - Z_c) / d g = 1.1677778 > 1 \)**
  - \%Air = 100.00
  - Assume 50% air pocket:
    - \%Air = 50
    - TAF = 0.6994975 < 1 (O.K.)

- **Quasi-Static Vertical Force** (eq 6.1.2.2.1-1)
  - \( F_{v_{\text{max}}} = 7.4058389 \) kip/ft
  - Length of Bridge = 318.00 ft
  - \( F_{v_{\text{max}}} \text{ Total} = 2355.06 \) kips 1177.53 tons

- **Associated Vertical Slamming Force** (AASHTO Sec 6.1.2.2.2)
  - \( B = -1.218169 \)
  - \( Z_c / \eta_{\text{max}} = -0.04995 < 0 \)
  - \( A = 0.0308557 \)

- **Vertical Slamming Force** (eq 6.1.2.2.2-1)
  - \( F_s = 4.3469490 \) kip/ft
  - Length of Bridge = 318.00 ft
  - \( F_s \text{ Total} = 1382.33 \) kips 691.17 tons

- **Associated Horizontal Quasi-Static Wave Force** (AASHTO Sec 6.1.2.2.3)
  - *Note: Girders used on the Kahaluu Bridge are AASHTO Type IV)
  - From Table 6.1.2.2.3-1:
    - \( a_0 = -0.0911 \) \( a_5 = 0.0048 \)
    - \( a_1 = 1.5445 \) \( a_6 = 0.0113 \)
    - \( a_2 = -1.4684 \) \( a_7 = 0.6785 \)
    - \( a_3 = 0.54 \) \( a_8 = -0.2661 \)
    - \( a_4 = -0.001 \)
  - \( x = 0.3757429 \)
  - \( y = 0.0485613 \)

- **Horizontal Quasi-Static Wave Force** (eq 6.1.2.2.3)
\[
\text{\(F_{h-av} = 0.7988378 \text{ kip/ft}\)}
\]
Length of Bridge = 318.00 ft
\[
\text{\(F_{h-av \ Total} = 254.03 \text{ kips} \quad 127.02 \text{ tons}\)}
\]

**Associated Moment about the Trailing Edge Due to the Quasi-static and Slamming Forces:** (AASHTO Sec 6.1.2.2.4)

- **For Girder Spans**
  - \(a_m = 0.8369375 \text{ ft}\)
  - \(b_m = -0.045446 \text{ ft}\)
  - \(c_m = -0.004767 \text{ ft}\)
  - \(W' = 4.08 \text{ ft}\)
  - \(W* = 41.92 \text{ ft}\)

\[
\text{\(M_{t-av} = 142.22 \text{ (kip/ft)-ft}\)}
\]
Length of Bridge = 318.00 ft
\[
\text{\(M_{t-av \ Total} = 45224.59 \text{ kip-ft} \quad 22612.29 \text{ tons-ft}\)}
\]

**Resulting Quasi-Static Vertical Force and Associated Forces and Moments:** (AASHTO sec 6.1.2.2)
- \(F_{v-max \ Total} = 2355.06 \text{ kips} \quad \text{(Quasi-Static Vertical Force)}\)
- \(F_{s \ Total} = 1382.33 \text{ kips} \quad \text{(Vertical Slamming Force)}\)
- \(F_{h-av \ Total} = 254.03 \text{ kips} \quad \text{(Quasi-Static Horizontal Force)}\)
- \(M_{t-av} = 45224.59 \text{ kip-ft} \quad \text{(Associated Moment about Trailing Edge)}\)

**Maximum Horizontal Wave Force and Associated Forces and Moments:** (AASHTO Sec 6.1.2.3)

**Maximum Horizontal Wave Force:** (AASHTO Sec 6.1.2.3.1)
- \(\omega \text{ check: eq 6.1.2.3.1-3 or eq 6.1.2.3.1-4}\)
- \(\text{check} = 120.44 > W = 46.00 \text{ ft}\)
- \(\omega = 46.00 \text{ ft}\)

\[
\text{Reference Horizontal Force: eq 6.1.2.3.1-2)}
\]
\[
\text{\(F_{h-max} = 3.24 \text{ kip/ft}\)}
\]

\[
\text{Horizontal Wave Force: eq 6.1.2.3.1-1)}
\]
\[
\text{\(F_{h-max} = 2.0358549 \text{ kip/ft}\)}
\]
Length of Bridge = 318.00 ft
\[
\text{\(F_{h-max \ Total} = 647.40 \text{ kip} \quad 323.70 \text{ tons}\)}
\]

**Associated Quasi-Static Vertical Force:** (AASHTO Sec 6.1.2.3.2)
- \(\alpha \text{ check: eq 6.1.2.3.2-3 or eq 6.1.2.3.2-4)}
- \(\text{check} = 32.29 < W = 46.00 \text{ ft}\)
- \(\alpha = 32.29 \text{ ft}\)

\[
\text{Reference Vertical Force: eq 6.1.2.3.2-2)}
\]
\[
\text{\(F_{v-ah} = 10.86 \text{ kip/ft}\)}
\]
\[
\text{Quasi-Static Vertical Wave Force: eq 6.1.2.3.2-1)}
\]
\[
\text{\(F_{v-ah} = 6.5944605 \text{ kip/ft}\)}
\]
Length of Bridge = 318.00 ft
\[
\text{\(F_{v-ah \ Total} = 2073.19 \text{ kip} \quad 1036.60 \text{ tons}\)}
\]

**Associated Vertical Slamming Forces:** (AASHTO Sec 6.1.2.3.3)
- \(\text{Note: Slamming forces calculated using the same method as AASHTO sec 6.1.2.2.2)}

\[
\text{Vertical Slamming Force: eq 6.1.2.2.2-1)}
\]
\[
\text{\(F_s = 4.3499001 \text{ kip/ft}\)}
\]
Length of Bridge = 318.00 ft
Fs Total = 1382.33 kips  691.17 tons

- Associated Moment About Trailing Edge: (AASHTO Sec 6.1.2.3.4)
  Reference Moment: (eq 6.1.2.3.4-2)
  \[ M_{ah} = 347.54253 \text{ (kip/ft)} \text{-ft} \]
  Associated Moment about Trailing Edge: (eq 6.1.2.3.4-1)
  \[ M_{t-ah} = 352.75575 \text{ (kip/ft)} \text{-ft} \]
  Length of Bridge = 318.00 ft
  \[ M_{ah} = 112276.33 \text{ kip-ft} \]

- Resulting Maximum Horizontal Wave Force and Associated Forces and Moments: (AASHTO sec 6.1.2.3)
  \[ F_{h-max} = 647.40 \text{ kips} \text{ (Maximum Horizontal Wave Force)} \]
  \[ F_{v-ah} = 2073.19 \text{ kips} \text{ (Quasi-Static Vertical Force)} \]
  \[ F_s = 1382.33 \text{ kips} \text{ (Vertical Slamming Force)} \]
  \[ M_{ah} = 112276.33 \text{ kip-ft} \text{ (Associated Moment about Trailing Edge)} \]

- Current Loads on Superstructure: (AASHTO Sec 6.1.2.4)
  *Note: Current loads are not considered in this study.
  \[ F_{hc} = 0 \text{ kips} \]
**New South Punaluu Bridge:**

**Method For Estimating Wave Forces on Bridge Superstructures**

**AASHTO (2008)**

### Assumed Conditions:
- 50 year wind speed = 105 mph
- Specific weight water = 62.4 kip/cubic ft
- Water Depth = 4.92 ft
- Water surface to bottom of deck = 4.92 ft
- 50 year wind speed = 105 mph
- Bridge Deck Width = 50.00 ft
- Bridge Deck Length = 66.00 ft
- Girder to Girder Width = 48.84 ft
- Height of girder = 1.75 ft
- Height of railing = 3.00 ft
- Height of bottom of girder = 0.00 ft
- Girder to Girder Width = 48.84 ft
- Water depth = 4.92 ft
- Height of girder = 1.75 ft
- Height of railing = 3.00 ft
- Height of bottom of girder = 0.00 ft
- Wave Calculations:

#### Bridge Properties:

- **Bridge Properties:**
  - Bridge Deck Width = 50.00 ft
  - Bridge Deck Length = 66.00 ft
  - Girder to Girder Width = 48.84 ft
  - Height of girder = 1.75 ft
  - Height of railing = 3.00 ft
  - Height of bottom of girder = 0.00 ft

#### Design Wave Parameters:

- **Design Wave Parameters:**
  - Hmax = 5.36 ft
  - λ = 222.87 ft
  - Water Depth = 4.92 ft
  - Water surface to bottom of deck = 4.92 ft
  - Height of girder = 1.75 ft
  - Height of railing = 3.00 ft
  - Height of bottom of girder = 0.00 ft

- **Determination of wave period:**
  - 50 year wind speed = 154.00 ft/sec
  - Gust Period (t) = 3 sec
  - 100 year wind speed = 164.78 ft/sec
  - Ut* = 287.33 ft/sec
  - d = 61.22 ft
  - Tp = (g*d)/(Ut*)^2
  - Tp = 13.35 sec

- **Determination of time duration to develop fetch limited waves:**
  - t = 12270.88 sec

#### Iteration Process (until T converges):

- First Iteration:
  - Ut = 177.00 ft/sec
- Second Iteration:
  - Ut* = 313.76 ft/sec
  - Tp = 13.35 sec
- Third Iteration:
  - Ut = 177.00 ft/sec
  - Ut* = 313.76 ft/sec
- Fourth Iteration:
  - Ut = 177.00 ft/sec
  - Ut* = 313.76 ft/sec
  - Tp = 13.35 sec
- Fifth Iteration:
  - Ut = 177.00 ft/sec
  - Ut* = 313.76 ft/sec
  - Tp = 13.35 sec

- **Wave Height and wave length:**
  - Hs = 24.41 ft
  - λ = 222.87 ft

- **Maximum Wave Height:**
  - Hmax = 43.93 ft
  - Hmax < 0.65*ds = 5.36 ft
  - Hmax < λ/7.0 = 31.84 ft

- Therefore Hmax = 5.36 ft

---

**Design for Storm Wave Properties:**

- **Design wave forces:**
  - Tp = 13.36 sec
Hmax = 5.36 ft
Wave Length (λ) = 222.87 ft
η max = 3.75 ft

**Maximum Quasi-Static Vertical Force and Associated Forces and Moments: (AASHTO Sec 6.1.2.2)**

**Maximum Quasi-Static Vertical Force: (AASHTO Sec 6.1.2.2.1)**

- **Determination of Forces parameters:**
  - W_hat = 249.84 ft
  - W_hat / W = 5.00 > 0.15 Therefore W_hat = W_hat
  - η max - Zc = 7.08 > db = 2.63 ft
  - x = 0.0240609
  - y = 1.120979

**For Girder Spans:**

- b0 = -0.763
- b1 = 56.385
- b2 = 0.053
- b3 = -193.1884

**Tapped Air Factor:**

- A_air = -0.0007189
- B_air = 1.8008872

- (η max - Zc) / dg = 4.0478571 > 1
- %Air = 100.00

- Assume 50% air pocket:
  - %Air = 50
  - TAF = 1.7649423 > 1 (N.G.) set TAF = 1
  - TAF = 1

**Quasi-Static Vertical Force:**

- Fv-max = 13.50676 kip/ft
- Length of Bridge = 66.00 ft
- Fv-max Total = 891.45 kips 445.72 tons

**Associated Vertical Slamming Force:**

- B = -1.1507468
- Zc / η max = -0.8871129 < 0
- A = 0.056382

**Vertical Slamming Force:**

- Fs = 2.4600879 kip/ft
- Length of Bridge = 66.00 ft
- Fs Total = 162.76 kips 81.38 tons

**Associated Horizontal Quasi-Static Wave Force:**

*Note: Girders used on the New South Punaluu Bridge are similar to the AASHTO Type III*

**From Table 6.1.2.3.1:**

- a1 = 1.2593333
- a2 = 0.0240609

**Horizontal Quasi-Static Wave Force:**

- a1 = 1.2593333
- a2 = 0.0240609
\[ F_{h-av} = 0.3958665 \text{ kip/ft} \]

Length of Bridge = 66.00 ft
Fh-av Total = 26.13 kips 13.06 tons

**Associated Moment about the Trailing Edge Due to the Quasi-static and Slamming Forces (AASHTO Sec 6.1.2.2.4)**

For Girder Spans:
- \( a_m = 0.8556875 \) ft
- \( b_m = -0.0553313 \) ft
- \( c_m = -0.0044163 \) ft
- \( W' = 1.16 \) ft
- \( W = 48.84 \) ft

Associated Moment about Trailing Edge: (eq 6.1.2.2.4-1)
- \( M_{t-av} = 87.89 \) (kip/ft)-ft

Length of Bridge = 66.00 ft
M_t-av Total = 5801.05 kip-ft 2900.53 tons-ft

**Resultant Quasi-Static Vertical Force and Associated Forces and Moments (AASHTO sec 6.1.2.2)**

\[ F_{v-max} \text{ Total} = 891.45 \text{ kips} \] (Quasi-Static Vertical Force)
\[ F_{s} \text{ Total} = 162.76 \text{ kips} \] (Vertical Slamming Force)
\[ F_{h-av} \text{ Total} = 26.13 \text{ kips} \] (Quasi-Static Horizontal Force)
\[ M_{t-av} = 5801.05 \text{ kip-ft} \] (Associated Moment about Trailing Edge)

**Maximum Horizontal Wave Force and Associated Forces and Moments (AASHTO Sec 6.1.2.3)**

\[ \omega \text{ check: (eq 6.1.2.3.1-3 or eq 6.1.2.3.1-4)} \]
- \( \omega \text{ check} = 236.35 > W = 50.00 \) ft Use eq 6.1.2.3.1-4 for omega
- \( \omega = 50.00 \) ft

Reference Horizontal Force: (eq 6.1.2.3.1-2)
- \( F_{h-max} = 1.43 \text{ kip/ft} \)

Horizontal Wave Force: (eq 6.1.2.3.1-1)
- \( F_{h-max} = 1.1434822 \text{ kip/ft} \)

Length of Bridge = 66.00 ft
Fh-max Total = 75.47 kip 37.73 tons

**Associated Quasi-Static Vertical Force (AASHTO Sec 6.1.2.3.2)**

\[ \alpha \text{ check: (eq 6.1.2.3.2-3 or eq 6.1.2.3.2-4)} \]
- \( \alpha \text{ check} = 96.09 > W = 50.00 \) ft Use eq 6.1.2.3.2-4 for alpha
- \( \alpha = 50.00 \) ft

Reference Vertical Force: (eq 6.1.2.3.2-2)
- \( F_{v-ah} = 22.67 \text{ kip/ft} \)

Quasi-Static Vertical Wave Force: (eq 6.1.2.3.2-1)
- \( F_{v-ah} = 10.825717 \text{ kip/ft} \)

Length of Bridge = 66.00 ft
Fv-ah Total = 714.50 kip 357.25 tons

**Associated Vertical Slamming Forces (AASHTO Sec 6.1.2.3.3)**

*Note: Slamming forces are calculated using the same method as AASHTO sec 6.1.2.2.2)*

\[ F_{s} \text{ Total} = 2.4660879 \text{ kip/ft} \]

Length of Bridge = 66.00 ft
\[ Fs_{\text{Total}} = 162.76 \text{ kips} \quad 81.38 \text{ tons} \]

**Associated Moment About Trailing Edge:** (AASHTO Sec 6.1.2.3.4)

**Reference Moment:** (eq 6.1.2.3.4-2)
\[ M_{\text{t-ah}} = 449.49207 \text{ (kip/ft)-ft} \]

**Associated Moment about Trailing Edge:** (eq 6.1.2.3.4-1)
\[ M_{\text{t-ah}} = 218.2946 \text{ (kip/ft)-ft} \]

- Length of Bridge = 66.00 ft
- Mo-tah Total = 14407.44 kip-ft 7203.72 ton-ft

\[ \mathbf{\text{Resulting Maximum Horizontal Wave Force and Associated Forces and Moments: (AASHTO sec 6.1.2.3)}} \]

- \( F_{h_{\text{max}} \text{ Total}} = 75.47 \text{ kips} \) (Maximum Horizontal Wave Force)
- \( F_{v_{\text{ah}} \text{ Total}} = 714.50 \text{ kips} \) (Quasi-Static Vertical Force)
- \( F_{s_{\text{Total}} = 162.76 \text{ kips} \) (Vertical Slamming Force)
- \( M_{\text{t-ah Total}} = 14407.44 \text{ kip-ft} \) (Associated Moment about Trailing Edge)

**Current Loads on Superstructure:** (AASHTO Sec 6.1.2.4)

- *Note: Current loads are not considered in this study.*
- \( F_{hc} = 0 \text{ kips} \)
New Makaha #3A Bridge
Method for Estimating Wave Forces on Bridge Superstructures

**AASHTO (2008)**

- **Wave Calculations:**
  - **Bridge Properties:**
    - Water Depth = 1.00 ft
    - Bridge Deck Length = 70.00 ft
    - Bridge Deck Width = 46.83 ft
    - Girder to Girder Width = 2.33 ft
    - Height of girder = 1.17 ft
    - Height of railing = 1.17 ft
    - Deck Thickness = 0.46 ft
    - Water surface to bottom of girder = 9.59 ft
    - Storm Water Level = 10.59 ft
    - Elevations to bottom of girder = 10.94 ft
    - Elevations to bottom of deck = 12.92 ft
  - **Design Wave Parameters:** (AASHTO Sec. 6.2.2.4)
    - Determination of wave period:
      - 50 year wind speed = 154.00 ft/sec
      - Gust Period (t) = 3 sec
      - 100 year wind speed = 164.78 ft/sec
      - $U_t^* = 287.33$ ft/sec
      - $d_s = 8.93$ ft
    - Fetch Length = $4767610.00$ ft
      \[ \left( \frac{g \cdot d}{U_t^*} \right)^2 = 0.011 \]
      - $T_p = 10.06$ sec
    - Determination of time duration to develop fetch limited waves:
      \[ t = 6336.05 \text{ sec} > 3600 \text{ sec} \]
      - $U_{1hr} = 171.09$ ft/sec
      - To compute $U_{1hr}$ use eq 6.2.2.4-5
    - Iteration Process (final $T_p$ converges)
      - First Iteration:
        - $U_t = 169.91$ ft/sec
      - Second Iteration:
        - $U_t^* = 298.38$ ft/sec
        - $T_p = 10.16$ sec
      - Third Iteration:
        - $U_t = 169.96$ sec
        - $T_p = 10.16$ sec
      - Fourth Iteration:
        - $U_t = 169.96$ sec
        - $T_p = 10.16$ sec
      - Fifth Iteration:
        - $U_t = 169.96$ sec
        - $T_p = 10.16$ sec
      - Sixth Iteration:
        - $U_t = 169.96$ sec
        - $T_p = 10.16$ sec

- **Resulting Storm Wave Properties:** (see Fig)
  - $T_p = 10.16$ sec

**D-14**

- **Storm Water Level**
- **Storm Surge + Local Wind Setup**
- **Wind Setup**
Hmax = 5.80 ft
Wave Length (\(\lambda\)) = 171.95 ft
\(\eta_{\text{max}} = 4.06\ ft\)

**Maximum Quasi-Static Vertical Force and Associated Forces and Moments (AASHTO Sec 6.1.2.2)**

- **Maximum Quasi-Static Vertical Force (AASHTO Sec 6.1.2.2.1)**
  - Determination of \(Fv_{\text{max}}\) parameters (eq 6.1.2.2.1-1)
    - \(W_{\text{hat}} = W\)
    - \(W_{\text{hat}} / W = 0.79 > 0.15\) Therefore \(W_{\text{hat}} = W\)
    - \(W_{\text{hat}} = 36.80\ ft\)
    - \(\eta_{\text{max}} - Zc = 2.40 < 2.79\ ft\)
    - \(\beta = 0.86\)
    - \(x = 0.037599\)
    - \(y = 2.14015\)

  - For Semi-Span (eq 6.1.2.2.1 b)
    - \(b_0 = -0.196975\)
    - \(b_1 = 35.067917\)
    - \(b_2 = 0.0471417\)
    - \(b_3 = -62.98042\)

  - **Tapped Air Factor**
    - \(A_{\text{air}} = 0.0074484\)
    - \(B_{\text{air}} = 0.2845313\)
    - \((\eta_{\text{max}} - Zc) / dg = 1.0299214 > 1\)

  - **Quasi-Static Vertical Force** (eq 6.1.2.2.1-1)
    - \(Fv_{\text{max}} = 4.2371596\ kip/ft\)
    - Length of Bridge = 70.00 ft
    - \(Fv_{\text{max}}\) Total = 296.60 kips 148.30 tons

- **Associated Vertical Slamming Force** (AASHTO Sec 6.1.2.2.2)
  - \(B = -0.863728\)
  - \(Zc / \eta_{\text{max}} = 0.40855 > 0\)
  - \(A = 0.0376874\)

  - **Vertical Slamming Force** (eq 6.1.2.2.2-1)
    - \(Fs = 1.5170513\ kip/ft\)
    - Length of Bridge = 70.00 ft
    - \(Fs\) Total = 106.19 kips 53.10 tons

**Associated Horizontal Quasi-Static Wave Force** (AASHTO Sec 6.1.2.2.3)
*Note: Girders used on the New Makaha #3A Bridge are similar to 36 in Adjacent Box Girders*

From Table 6.1.2.2.3-1 (for Box Girders):

\[
a_0 = 0.40281 \\
a_1 = -1.116 \\
a_2 = -11.116 \\
a_3 = -1.116 \\
a_4 = -1.116 \\
b_0 = 0.037599 \\
\]

\[
x = 0.037599 \\
y = 0.037599 \\
\]
### Horizontal Quasi-Static Wave Force: (eq 6.1.2.2.3)
- \( F_{h-av} = 0.3482357 \text{ kip/ft} \)
- Length of Bridge = 70.00 ft
- \( F_{h-av \text{ Total}} = 24.38 \text{ kips} \)
- \( 12.19 \text{ tons} \)

#### Associated Moment about the Trailing Edge due to the Quasi-Static and Slamming Forces: (AASHTO Sec 6.1.2.2.4)
- For Slab Spans:
  - \( a_m = 0.8243958 \text{ ft} \)
  - \( b_m = -0.040658 \text{ ft} \)
  - \( c_m = -0.0049 \text{ ft} \)
  - \( W' = 0.00 \text{ ft} \)
  - \( W* = 46.83 \text{ ft} \)

#### Associated Moment about Trailing Edge: (eq 6.1.2.2.4-1)
- \( M_{t-av} = 48.1 \text{ (kip/ft)-ft} \)
- Length of Bridge = 70.00 ft
- \( M_{t-av \text{ Total}} = 3366.74 \text{ kip-ft} \)
- \( 1683.37 \text{ tons-ft} \)

### Resulting Quasi-Static Vertical Force and Associated Forces and Moments: (AASHTO Sec 6.1.2.2)
- \( F_{v-max \text{ Total}} = 296.60 \text{ kips} \) (Quasi-Static Vertical Force)
- \( F_{s \text{ Total}} = 106.19 \text{ kips} \) (Vertical Slamming Force)
- \( F_{h-av \text{ Total}} = 24.38 \text{ kips} \) (Quasi-Static Horizontal Force)
- \( M_{t-av} = 3366.74 \text{ kip-ft} \) (Associated Moment about Trailing Edge)

### Maximum Horizontal Wave Force and Associated Forces and Moments: (AASHTO Sec 6.1.2.3)

#### Maximum Horizontal Wave Force: (AASHTO Sec 6.1.2.3.1)
- **ω check:** (eq 6.1.2.3.1-3 or eq 6.1.2.3.1-4)
  - \( \text{check} = 104.37 > W = 46.83 \text{ ft} \)
  - Use eq 6.1.2.3.1-4 for ω
- \( \omega = 46.83 \text{ ft} \)
- **Reference Horizontal Force:** (eq 6.1.2.3.1-2)
  - \( F_{h-max} = 1.34 \text{ kip/ft} \)
- **Horizontal Wave Force:** (eq 6.1.2.3.1-1)
  - \( F_{h-max} = 0.76856 \text{ kip/ft} \)
- Length of Bridge = 70.00 ft
- \( F_{h-max \text{ Total}} = 53.80 \text{ kip} \) (26.90 tons)

#### Associated Quasi-Static Vertical Force: (AASHTO Sec 6.1.2.3.2)
- **α check:** (eq 6.1.2.3.2-3 or eq 6.1.2.3.2-4)
  - \( \text{check} = 141.0 < W = 46.83 \text{ ft} \)
  - Use eq 6.1.2.3.2-3 for α
- \( \alpha = 141.0 \text{ ft} \)
- **Reference Vertical Force:** (eq 6.1.2.3.2-2)
  - \( F_{v-ah} = 2.18 \text{ kip/ft} \)
- **Quasi-Static Vertical Wave Force:** (eq 6.1.2.3.2-1)
  - \( F_{v-ah} = 3.7502229 \text{ kip/ft} \)
- Length of Bridge = 70.00 ft
- \( F_{v-ah \text{ Total}} = 262.52 \text{ kip} \) (131.26 tons)

#### Associated Vertical Slamming Forces: (AASHTO Sec 6.1.2.3.3)
- *Note: Slamming forces are calculated using the same method as AASHTO sec 6.1.2.2.2)
- **Vertical Slamming Force:** (eq 6.1.2.2.2-1)
  - \( F_{s} = 1.5130513 \text{ kip/ft} \)
Length of Bridge = 70.00 ft
Fs Total = 106.19 kips 53.10 tons

**Associated Moment About Trailing Edge** (AASHTO Sec 6.1.2.3.4)

Reference Moment: (eq 6.1.2.3.4-2)
\[ M_{t-ah} = 167.49822 \text{ (kip/ft)-ft} \]

Associated Moment about Trailing Edge: (eq 6.1.2.3.4-1)
\[ M_{t-ah} = 188.53083 \text{ (kip/ft)-ft} \]

Length of Bridge = 70.00 ft
Mt-ah Total = 13197.16 kip-ft 6598.58 ton-ft

**Resulting Maximum Horizontal Wave Force and Associated Forces and Moments** (AASHTO secs 6.1.2.2.3)

<table>
<thead>
<tr>
<th>Force</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fh-max Total</td>
<td>53.80 kips</td>
<td>(Maximum Horizontal Wave Force)</td>
</tr>
<tr>
<td>Fv-ah Total</td>
<td>262.52 kips</td>
<td>(Quasi-Static Vertical Force)</td>
</tr>
<tr>
<td>Fs Total</td>
<td>106.19 kips</td>
<td>(Vertical Slamming Force)</td>
</tr>
<tr>
<td>Mt-ah Total</td>
<td>13197.16 kip-ft</td>
<td>(Associated Moment about Trailing Edge)</td>
</tr>
</tbody>
</table>

**Current Loads on Superstructure** (AASHTO Sec 6.1.2.2.4)

*Note: Current load are not considered in this study.

<table>
<thead>
<tr>
<th>Rhc</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0 kips</td>
</tr>
</tbody>
</table>
Old Makaha #3A Bridge:

Method for Estimating Wave Forces on Bridge Superstructures

AASHTO (2008)

Constants Coefficients:
- 50 year wind speed = 105 mph
- Water depth, to center of girder = 4.00 ft
- Water Depth = 7.24 ft
- Height of girder = 1.50 ft
- Water surface to bottom of girder = 7.24 ft
- Bridge Deck Length = 78.83 ft
- Bridge Deck Width = 32.83 ft
- Girder to Girder Width = 32.83 ft
- girder = 1.50 ft
- Water surface to bottom of girder = 7.24 ft
- Girder to Girder Width = 32.83 ft
- Bridge Deck Width = 32.83 ft
- Water Depth = 4.00 ft
- Elevation to bottom of girder = 11.24 ft
- Height of girder = 1.50 ft
- Height of railing = 0.00 ft
- Storm Water Level = 2.31 ft
- Storm Surge + Local Wind Setup = 8.93 ft
- Design Wave Parameters: (AASHTO Sec 6.2.2.4)

- Wave Calculations:

  - Bridge Properties:
    - Bridge Deck Length = 78.83 ft
    - Bridge Deck Width = 32.83 ft
    - Girder to Girder Width = 32.83 ft
    - Deck Thickness = 0.50 ft
    - Height of railing = 0.00 ft

  - Constant Coefficients:
    - 50 year wind speed = 105 mph
    - Specific weight water = 64 kip/ft³
    - g = 32.2 ft/sec²
    - 60 year wind speed = 154.00 ft/sec
    - Gust Period (t) = 3 sec
    - 100 year wind speed = 164.78 ft/sec
    - Ut* = 287.33 ft/sec
    - (from NFIP Flood Hazard Assessment Tool)
    - (average water depth over fetch length)
    - Fetch Length = 1114 ft
    - & = 27.10 ft
    - d = 0.05 ft
    - (g*d)/(Ut*)² = 0.011
    - Tp = 10.06 sec
    - t = 6336.05 sec > 3000 sec

- Design Wave Parameters: (AASHTO Sec 6.2.2.4)

  - Determination of wave period:
    - 50 year wind speed = 154.00 ft/sec
    - Gust Period (t) = 3 sec
    - 100 year wind speed = 164.78 ft/sec
    - Ut* = 287.33 ft/sec
    - (from NFIP Flood Hazard Assessment Tool)
    - (average water depth over fetch length)
    - Fetch Length = 1114 ft
    - & = 27.10 ft
    - d = 0.05 ft
    - (g*d)/(Ut*)² = 0.011
    - Tp = 10.06 sec
    - t = 6336.05 sec > 3000 sec

  - Determination of time duration to develop fetch limited waves:
    - t = 6336.05 sec > 3000 sec
    - U_1hr = 171.96 ft/sec
    - To compute U_1 hr use eq 6.2.2.4-5

- Wave Height and Wave Slope:

  - Wave Height:
    - Hs = 12.92 ft
    - Wave Length (λ) = 171.96 ft
    - Hs = 23.25 ft
    - Maximum Wave Height:
      - Hs = 23.25 ft

  - Wave Slope:
    - Hs = 0.05 Hs = 3.80 ft
    - Hs = 23.25 ft
    - Therefore: Hs = 5.80 ft

- Resulting Storm Wave Property: (see Fig)

  - Tp = 10.16 sec
Maximum Quasi-Static Vertical Force and Associated Forces and Moments (AASHTO Sec 6.1.2.2)

- Maximum Quasi-Static Vertical Force (AASHTO Sec 6.1.2.2.1)
  - Determination of Parameters (eq 6.1.2.2.1-1)
    - \( W_{\text{hat}} = 17.54 \, \text{ft} \)
    - \( W_{\text{hat}} / W = 0.53 > 0.15 \) Therefore \( W_{\text{hat}} = W_{\text{hat}} \)
    - \( \eta_{\text{max}} - Z_c = 1.75 < dB = 2.00 \, \text{ft} \)
    - \( x = 0.0337569 \)
    - \( y = 0.1020329 \)

- For Girder Spans (eq 6.1.2.2.1-1)
  - \( b_0 = -0.738 \)
  - \( b_4 = -0.00039 \)
  - \( b_1 = 56.43 \)
  - \( b_5 = 41.332 \)
  - \( b_2 = 0.0496 \)
  - \( b_6 = 4.71 \)
  - \( b_3 = -193.2472 \)

- Tapped Air Factor
  - \( A_{\text{air}} = 0.0074333 \)
  - \( B_{\text{air}} = 0.2975314 \)

- Quasi-Static Vertical Force (eq 6.1.2.2.1-1)
  - \( F_{v_{\text{max}}} = 1.3744929 \, \text{kip/ft} \)

- Length of Bridge = 78.83 ft
  - \( F_{v_{\text{max}}} \, \text{Total} = 108.36 \, \text{kips} \)
  - 54.18 tons

- Associated Vertical Slamming Force (AASHTO Sec 6.1.2.2.2)
  - \( B = -0.74079 \)
  - \( Z_c / \eta_{\text{max}} = 0.5685244 > 0 \)
  - \( A = 0.040071 \)

- Vertical Slamming Force (eq 6.1.2.2.2-1)
  - \( F_s = 0.8505833 \, \text{kip/ft} \)

- Length of Bridge = 78.83 ft
  - \( F_s \, \text{Total} = 67.05 \, \text{kips} \)
  - 33.53 tons

- Associated Horizontal Quasi-Static Wave Force (AASHTO Sec 6.1.2.2.3)
  - *Note: Girders used on the Old Makaha S1A Bridge are similar to 21 in Voided Slabs

- From Table 6.1.2.2.3-1 (for 2" Voided Slab)
  - \( a_0 = 0.088 \)
  - \( a_5 = 0.02009 \)
  - \( a_1 = 1.3927 \)
  - \( a_6 = -0.0626 \)
  - \( a_2 = -1.40718 \)
  - \( a_7 = -0.5684 \)
  - \( a_3 = 0.2056 \)
  - \( a_8 = -0.4112 \)
  - \( a_4 = -0.005 \)

- \( x = 0.876575 \)
- \( y = 0.0337569 \)

- Horizontal Quasi-Static Wave Force (eq 6.1.2.2.3)
\[
F_{h-av} = 0.4398445 \text{ kip/ft}
\]

Length of Bridge = 78.83 ft

\[F_{h-av} \text{ Total} = 34.67 \text{ kips} \quad 17.34 \text{ tons}\]

- **Associated Moment about the Trailing Edge Due to the Quasi-static and Slamming Forces** (AASHTO Sec 6.1.2.2.4)
  - For Girder Spans:
    \[
    \begin{align*}
    a_m &= 0.901 \text{ ft} \\
    b_m &= -0.07922 \text{ ft} \\
    c_m &= -0.003568 \text{ ft} \\
    W' &= 0.00 \text{ ft} \\
    W^* &= 32.83 \text{ ft}
    \end{align*}
    \]
  - Associated Moment about Trailing Edge (eq 6.1.2.2.4-1)
    \[
    M_{t-av} = 1494.13 \text{ kip-ft} \quad 747.06 \text{ tons-ft}
    \]

- **Resulting Quasi-static Vertical Force and Associated Forces and Moments** (AASHTO sec 6.1.2.2)
  - **Maximum Horizontal Wave Force and Associated Forces and Moments** (AASHTO Sec 6.1.2.3)
  - **Maximum Horizontal Wave Force** (AASHTO Sec 6.1.2.3.1)
    - \(\omega\) check:
      \[
      \omega = \frac{W}{a_m} = \frac{32.83}{0.901} = 36.44
      \]
    - \(\alpha\) check:
      \[
      \alpha = \frac{W}{b_m} = \frac{32.83}{-0.07922} = 416.83
      \]
    - Reference Horizontal Force:
      \[
      F_{h-max} = 0.49 \text{ kip/ft}
      \]
    - Horizontal Wave Force:
      \[
      F_{h-max} = 0.4433121 \text{ kip/ft}
      \]
  - **Associated Quasi-static Vertical Force** (AASHTO Sec 6.1.2.3.2)
    - \(\alpha\) check:
      \[
      \alpha = \frac{W}{c_m} = \frac{32.83}{-0.003568} = 9176.74
      \]
      Use eq 6.1.2.3.2-3 for \(\alpha\)
    - Reference Vertical Force:
      \[
      F_{v-ah} = 0.76 \text{ kip/ft}
      \]
  - **Quasi-static Vertical Wave Force**:
    \[
    F_{v-ah} = 1.3787925 \text{ kip/ft}
    \]
  - **Associated Vertical Slamming Forces** (AASHTO Sec 6.1.2.3.3)
    - Note: Slamming forces calculated using the same method as AASHTO sec 6.1.2.2.2)
Fs Total = 67.05 kips 33.53 tons

- **Associated Moment About Trailing Edge:** (AASHTO Sec 6.1.2.3.4)

  Reference Moment: (eq 6.1.2.3.4-2)
  \[ M_{ah} = 49.685183 \ (\text{kip}/\text{ft}) - \text{ft} \]

  Associated Moment about Trailing Edge: (eq 6.1.2.3.4-1)
  \[ M_{t-ah} = 55.454519 \ (\text{kip}/\text{ft}) - \text{ft} \]

  Length of Bridge = 78.93 ft
  \[ M_{ah \ Total} = 4371.66 \ (\text{kip-ft}) 218.583 \ (\text{ton-ft}) \]

- **Resulting Maximum Horizontal Wave Force and Associated Forces and Moments:** (AASHTO sec 6.1.2.3)

  \[ F_{h-max \ Total} = 34.95 \ (\text{Maximum Horizontal Wave Force}) \]
  \[ F_{v-ah \ Total} = 108.69 \ (\text{Quasi-Static Vertical Force}) \]
  \[ F_{s \ Total} = 67.05 \ (\text{Vertical Slamming Force}) \]
  \[ M_{ah \ Total} = 4371.66 \ (\text{kip-ft}) 218.583 \ (\text{ton-ft}) \]

- **Current Loads on Superstructure:** (AASHTO Sec 6.1.2.4)

  *Note: Current loads are not considered in this study.*

  \[ F_{hc} = 0 \ (\text{kips}) \]
**Maripalaoa Bridge:**
Method For Estimating Wave Forces on Bridge Superstructures

**AASHTO (2008)**

**Dwelling Coefficients:**
- 50 year wind speed = 154 mph
- 100 year wind speed = 164.78 ft/sec
- Specific weight water = 0.064 kip/cubic ft
- G = 32.2 ft/sec^2

**Wave Calculations:**

- Bridge Properties:
  - Bridge Deck Width: 64.3 ft
  - Bridge Deck Length: 100.67 ft
  - Girder to Girder Width: 59.00 ft
  - Height of girder: 3.00 ft
  - Deck Thickness: 0.50 ft
  - Height of railing: 2.00 ft

- Water surface to bot. of girder = 3.50 ft
- Girder to girder width = 59.00 ft
- Height of railing = 2.00 ft
- Water depth = 3.00 ft
- Z_c = -1.75 ft
- Storm water level = 6.50 Ft
- Storm Surge + Local Wind Setup: 8.25 ft

- 50 year wind speed = 105 mph
  (from AASHTO Fig 6.2.2-1 b)

- Constant Coefficients:
  - 50 year wind speed = 105 mph
  (from AASHTO Fig 6.2.2-1 b)
  - W* = 59.00 ft
  - Specific weight water = 0.064 kip/cubic ft
  - R = 2.00 ft
  - G = 32.2 ft/sec^2
  - * = 3.75 ft

**Wave Calculations:**

- *Bridge Properties:*
  - *Hmax = 5.36 ft*
  - db = 3.50 ft
  - Bridge Deck Width = 64.33 ft
  - Water Depth = 3.00 ft
  - dg (ft) = 3.00
  - Bridge Deck Length = 100.67 ft
  - Height of girder = 3.00 ft
  - Girder to Girder Width = 59.00 ft
  - Height of railing = 2.00 ft
  - Deck Thickness = 0.50 ft
  - Height of railing = 2.00 ft
  - Water surface to bot. of girder = 3.50 ft

**Design Wave Parameters:**

- *Wave Calculations:
  - Determination of wave period:
    - 50 year wind speed = 154.00 ft/sec
    - Gust Period (t) = 3 sec
    - U*[t] = 287.33 ft/sec
    - *d = 20.59 ft
    - (average water depth over fetch length)
    - Fetch Length = 5476.22 ft
    - [(g*d)/(Ut*)^2] = 0.008
    - Tp = 9.10 sec
  - Determination of time duration to develop fetch limited waves:
    - t = 5022.33 sec
  - U_1hr = 168.44 ft/sec
    To compute U_1hr use eq 6.2.2.4-5

**Iteration Process (until 't' converges):**

- First Iteration:
  - Ut = 167.68 ft/sec
  - Ut*[t] = 292.56 ft/sec
  - [(g*d)/(Ut*)^2] = 0.008
  - Tp = 9.10 sec
  - t = 4944.65 sec

- Second Iteration:
  - Ut* = 292.56 ft/sec
  - [(g*d)/(Ut*)^2] = 0.008
  - Tp = 9.16 sec
  - t = 4943.88 sec

- Third Iteration:
  - Ut = 167.71 sec
  - Ut*[t] = 292.63 ft/sec
  - [(g*d)/(Ut*)^2] = 0.008
  - Tp = 9.16 sec
  - t = 4943.87 sec

- Fourth Iteration:
  - Ut = 167.71 sec
  - Ut*[t] = 292.63 ft/sec
  - [(g*d)/(Ut*)^2] = 0.008
  - Tp = 9.16 sec
  - t = 4943.87 sec

- Fifth Iteration:
  - Ut = 167.71 sec
  - Ut*[t] = 292.63 ft/sec
  - [(g*d)/(Ut*)^2] = 0.008
  - Tp = 9.16 sec
  - t = 4943.87 sec

**Wave Height and Wave Depth:**

- *Hs = 10.43 ft
- Wave Length (λ) = 148.88 ft
- Hmax = 18.77 ft

**Maximum Wave Height** (least of the following)

- Hmax = 18.77 ft
- Hmax ≤ 0.65*ds = 5.36 ft
- Hmax ≤ λ/7.0 = 21.27 ft

Therefore Hmax = 5.36 ft

**Resulting Storm Wave Properties:**

- *Tp = 9.16 sec*
Maximum Quasi-Static Vertical Force and Associated Forces and Moments (AASHTO Sec 6.1.2.2)

Maximum Quasi-Static Vertical Force: (AASHTO Sec 6.1.2.2.1)

Determination of Parameters (eq 6.1.2.2.1-1)

\[ \frac{W_{\text{hat}}}{W} = 1.91 > 0.15 \text{ Therefore } W_{\text{hat}} = W \]

\[ W_{\text{hat}} = 123.02 \text{ ft} \]

\[ \frac{\eta_{\text{max}} - Z_c}{\beta} = 1.8345833 > 1 \]

\[ \%\text{Air} = -83.46 \]

Assume 50% air pocket:

\[ \%\text{Air} = 50 \]

\[ \text{TAF} = 0.797079 < 1 \text{ (O.K.)} \]

Quasi-Static Vertical Force: (eq 6.1.2.2.1-1)

\[ F_{v\text{-max}} = 9.6207819 \text{ kip/ft} \]

\[ L_{\text{bridge}} = 100.67 \text{ ft} \]

\[ F_{v\text{-max Total}} = 968.49 \text{ kips 484.25 tons} \]

Associated Vertical Slamming Force: (AASHTO Sec 6.1.2.2.2)

\[ B = -1.3000709 \]

\[ \frac{Z_c}{\eta_{\text{max}}} = -0.4662005 < 0 \]

\[ A = 0.0246536 \]

Vertical Slamming Force: (eq 6.1.2.2.2-1)

\[ F_s = 3.415099 \text{ kip/ft} \]

\[ L_{\text{bridge}} = 100.67 \text{ ft} \]

\[ F_s \text{ Total} = 343.78 \text{ kips 171.89 tons} \]

Associated Horizontal Quasi-Static Wave Force: (AASHTO Sec 6.1.2.2.3)

*Note: Girders used on the Maipalaoa Bridge are similar to AASHTO Type III girders

\[ \text{From Table 6.1.2.2.3-1 (for AASHTO Type III)} \]

\[ a_1 = 1.0197 \]

\[ a_6 = 0.0116 \]

\[ a_2 = -1.40562 \]

\[ a_7 = -0.9944 \]

\[ a_3 = 0.8107 \]

\[ a_8 = -0.243 \]

\[ a_4 = -0.0877 \]

\[ x = 1.0000818 \]

\[ y = 0.03002 \]

Horizontal Quasi-Static Wave Force: (eq 6.1.2.2.3)
\( F_{h-av} = 0.3885508 \text{ kip/ft} \)

Length of Bridge = 100.67 ft

\( F_{h-av \text{ Total}} = 39.11 \text{ kips} 19.56 \text{ tons} \)

- **Associated Moment about the Trailing Edge Due to the Quasi-static and Slamming Forces** (AASHTO Sec 6.1.2.2.4)
  - For Girder Spans
    - \( a_m = 0.85725 \text{ ft} \)
    - \( b_m = -0.056155 \text{ ft} \)
    - \( c_m = -0.004387 \text{ ft} \)
    - \( W' = 5.33 \text{ ft} \)
    - \( W* = 59.00 \text{ ft} \)
  - **Associated Moment about Trailing Edge** (eq 6.1.2.2.4-3)
    - \( M_{t-av} = 166.69 \text{ (kip/ft)-ft} \)
  - Length of Bridge = 100.67 ft
  - \( M_{t-av \text{ Total}} = 16779.78 \text{ kip-ft} 8389.89 \text{ tons-ft} \)

- **Resulting Quasi-Static Vertical Force and Associated Forces and Moments** (AASHTO sec 6.1.2.2)
  - \( F_{v-\text{max Total}} = 968.49 \text{ kips} \) (Quasi-Static Vertical Force)
  - \( F_s \text{ Total} = 343.78 \text{ kips} \) (Vertical Slamming Force)
  - \( F_{h-av \text{ Total}} = 39.11 \text{ kips} \) (Quasi-Static Horizontal Force)
  - \( M_{t-av} = 16779.78 \text{ kip-ft} \) (Associated Moment about Trailing Edge)

- **Maximum Horizontal Wave Force and Associated Forces and Moments** (AASHTO Sec 6.1.2.3)
  - **Maximum Horizontal Wave Force** (AASHTO Sec 6.1.2.3.1)
    - \( \omega \text{ check} \) (eq 6.1.2.3.1-3 or eq 6.1.2.3.1-4)
      - \( \text{check} = 135.95 \text{ ft} \) \( W = 64.33 \text{ ft} \)
        - Use eq 6.1.2.3.1-4 for \( \omega \)
      - \( \alpha = 64.33 \text{ ft} \)
    - **Reference Horizontal Force** (eq 6.1.2.3.1-2)
      - \( F_{h-\text{max}} = 2.67 \text{ kip/ft} \)
    - **Horizontal Wave Force** (eq 6.1.2.3.1-1)
      - \( F_{h-\text{max}} = 1.2744943 \text{ kip/ft} \)
  - Length of Bridge = 100.67 ft
  - \( F_{h-\text{max Total}} = 128.30 \text{ kip} 64.15 \text{ tons} \)

- **Associated Quasi-Static Vertical Force** (AASHTO Sec 6.1.2.3.2)
  - **\( \alpha \text{ check} \)** (eq 6.1.2.3.2-3 or eq 6.1.2.3.2-4)
    - \( \text{check} = 47.32 \text{ ft} \) \( W = 64.33 \text{ ft} \)
      - Use eq 6.1.2.3.2-3 for \( \alpha \)
    - \( \alpha = 47.32 \text{ ft} \)
  - **Reference Vertical Force** (eq 6.1.2.3.2-2)
    - \( P_{v-\text{ah}} = 16.67 \text{ kip/ft} \)
  - **Quasi-Static Vertical Wave Force** (eq 6.1.2.3.2-1)
    - \( P_{v-\text{ah}} = 8.0648175 \text{ kip/ft} \)
  - Length of Bridge = 100.67 ft
  - \( P_{v-\text{ah \text{ Total}}} = 811.86 \text{ kip} 405.93 \text{ tons} \)

- **Associated Vertical Slamming Force** (AASHTO Sec 6.1.2.3.3)
  - **Note:** Slamming force is calculated using the same method as AASHTO sec 6.1.2.2.2)
  - **Vertical Slamming Force** (eq 6.1.2.2.2-1)
  - \( F_s = 3.45 \text{ kip} \)
  - Length of Bridge = 100.67 ft
\[ F_{\text{Total}} = 343.78 \text{ kips} \quad 171.89 \text{ tons} \]

- **Associated Moment About Trailing Edge:** (AASHTO Sec 6.1.2.3.4)
  - Reference Moment: (eq 6.1.2.3.4-2)
    \[ M_{\text{t-ah}} = 489.35969 \text{ (kip-ft)} \]
  - Associated Moment about Trailing Edge: (eq 6.1.2.3.4-1)
    \[ M_{\text{t-ah}} = 386.36777 \text{ (kip-ft)} \]
  - Length of Bridge = 100.67 ft
    \[ M_{\text{Total-ah}} = 38712.95 \text{ kip-ft} \quad 19356.48 \text{ ton-ft} \]

- **Resulting Maximum Horizontal Wave Force and Associated Forces and Moments:** (AASHTO sec 6.1.2.3)
  - \[ F_{\text{h-max}} = 128.30 \text{ kips} \quad \text{(Maximum Horizontal Wave Force)} \]
  - \[ F_{\text{v-ah}} = 811.86 \text{ kips} \quad \text{(Quasi-Static Vertical Force)} \]
  - \[ F_{\text{s}} = 343.78 \text{ kips} \quad \text{(Vertical Slamming Force)} \]
  - \[ M_{\text{ah Total}} = 38712.95 \text{ kip-ft} \quad \text{(Associated Moment about Trailing Edge)} \]

- **Current Loads on Superstructure:** (AASHTO Sec 6.1.2.4)
  - \[ F_{\text{hc}} = 0 \text{ kips} \]
  - *Note: Current loads are not considered in this study.*
Moanalua Bridge (single span)  

Method For Estimating Wave Forces on Bridge Superstructures

AASHTO (2008)  

Assumptions:  
- 50 year wind speed = 115 mph  
- 100 year wind speed = 164.78 mph  
- Specific weight water = 62.43# cu/ft  
- g = 32.2 ft/sec²

Wave Calculations:

- Bridge Properties:
  - Bridge Deck Width = 64.33 ft
  - Bridge Deck Length = 27.00 ft
  - Girder to Girder Width = 27.00 ft
  - Girder to Girder Height of girder = 1.83 ft
  - Height of railing = 3.73 ft
  - Height of masonry = 3.73 ft

- Water Properties:
  - Water Depth = 2.50 ft
  - Water surface to bot. of girder = 4.00 ft
  - Height of masonry = 3.73 ft

- Design Wave Parameters (AASHTO Sec 6.2.2.4)

  - Determination of wave period:  
    - 50 year wind speed = 154.00 ft/sec
    - Gust Period (t) = 3 sec
    - 100 year wind speed = 164.78 ft/sec
    - Uₜ* = 287.33 ft/sec

  - Fetch Length = 4767610.00 ft

  - [(g*d)/(Ut*)²] = 0.003
  - Tₚ = 6.24 sec

  - Determination of time duration to develop fetch limited waves:
    - t = 2081.03 sec < 3600 sec

  - Iteration Process (until Tₚ converges):
    - First Iteration:
      - Ut = 110.17 ft/sec
    - Second Iteration:
      - Ut = 110.17 ft/sec
    - Third Iteration:
      - Ut = 110.17 ft/sec
    - Fourth Iteration:
      - Ut = 110.17 ft/sec
    - Fifth Iteration:
      - Ut = 110.17 ft/sec

  - Wave height and wavelength:
    - Hₛ = 3.13 ft
    - Wave Length (λ) = 80.54 ft

  - Maximum Wave Height (least of the following):
    - Hₘₐₓ = 6.71 ft

  - Breaking Section Wave Properties (see Fig):
    - Tₚ = 5.49 sec
Maximum Quasi-Static Vertical Force and Associated Forces and Moments:

(AASHTO Sec 6.1.2.2)

- Maximum Quasi-Static Vertical Force:
  - Determination of \( F_{v-max} \) parameters:
    - \( W_{hat} = 46.95 \) ft
    - \( W_{hat} / W = 0.73 > 0.15 \) Therefore \( W_{hat} = W_{hat} \)
    - \( \eta_{max} - Z_{c} = 3.50 > dB = 2.50 \) ft
    - \( \beta = 1.00 \)
    - \( x = 0.055442 \)
    - \( y = 0.5828575 \)
    - For Girder Spans:
      - \( b_{0} = -0.7713333 \)
      - \( b_{1} = 56.37 \)
      - \( b_{2} = 0.0505333 \)
      - \( b_{3} = -193.1688 \)
      - \( b_{4} = -0.00041 \)
      - \( b_{5} = -0.27067 \)
      - \( b_{6} = 5.41 \)
      - \( b_{7} = 0.00000 \)
    - Tapped Air Factor:
      - \( A_{air} = 0.0069199 \)
      - \( B_{air} = 0.2927231 \)
    - \( (\eta_{max} - Z_{c}) / dg = 1.9068273 > 1 \)
    - \%Air = 100.00
    - Assume 50% air pocket:
      - \%Air = 50
      - \%Air = 0.6387196 < 1 (O.K.)
    - \( F_{v-max} = 3.3863839 \) kip/ft
    - Length of Bridge = 27.00 ft
    - \( F_{v-max \ Total} = 91.43 \) kips 45.72 tons

- Associated Vertical Slamming Force:
  - \( B = -1.2473094 \)
  - \( Z_{c} / \eta_{max} = -0.1183678 < 0 \)
  - \( A = 0.0298363 \)
  - Vertical Slamming Force:
    - \( F_s = 1.4043874 \) kip/ft
    - Length of Bridge = 27.00 ft
    - \( F_s \ Total = 37.92 \) kips 18.96 tons

- Associated Horizontal Quasi-Static Wave Force:
  - (AASHTO Sec 6.1.2.2.3)
  - *Note: Girders used on the Maipalaoa Bridge are similar to Florida Bulb - T 72s)*
  - From Table 6.1.2.2.3-1:
    - \( a_{0} = -0.2076 \)
    - \( a_{1} = 1.5772 \)
    - \( a_{2} = -1.048 \)
    - \( a_{3} = 0.0551 \)
    - \( a_{4} = 0.093 \)
    - \( a_{5} = -0.0167 \)
    - \( a_{6} = -0.0346 \)
    - \( a_{7} = 0.5282 \)
    - \( a_{8} = -0.139 \)
    - \( b_{0} = -0.5612067 \)
    - \( b_{1} = 0.055442 \)
    - Horizontal Quasi-Static Wave Force:
      - \( D-27 \)
\[ F_{h-av} = 0.2478735 \text{ kip/ft} \]
\[ \text{Length of Bridge} = 27.00 \text{ ft} \]
\[ F_{h-av \text{ Total}} = 6.69 \text{ kips} \quad 3.35 \text{ tons} \]

**Associated Moment about the Trailing Edge Due to the Quasi-static and Slamming Forces** (AASHTO Sec 6.1.2.2.4)

For Girder Spans:

\[ a_m = 0.8481354 \text{ ft} \]
\[ b_m = -0.0513498 \text{ ft} \]
\[ c_m = -0.0045576 \text{ ft} \]

\[ W' = 3.50 \text{ ft} \]
\[ W* = 60.83 \text{ ft} \]

Associated Moment about Trailing Edge: (eq 6.1.2.2.4-1)

\[ F_{m-t} = 66.36 \text{ (kip/ft)-ft} \]

\[ \text{Length of Bridge} = 27.00 \text{ ft} \]
\[ M_{t-av \text{ Total}} = 1791.59 \text{ kip-ft} \quad 895.79 \text{ tons-ft} \]

**Resulting Quasi-Static Vertical Force and Associated Forces and Moments** (AASHTO sec 6.1.2.2)

\[ F_{v-max \text{ Total}} = 91.43 \text{ kips} \]

(Quasi-Static Vertical Force)

\[ F_{s \text{ Total}} = 37.92 \text{ kips} \]

(Vertical Slamming Force)

\[ F_{h-av \text{ Total}} = 6.69 \text{ kips} \]

(Quasi-Static Horizontal Force)

\[ M_{t-av} = 1791.59 \text{ kip-ft} \]

(Associated Moment about Trailing Edge)

**Maximum Horizontal Wave Force and Associated Forces and Moments** (AASHTO Sec 6.1.2.3)

\[ F_{h-max \text{ Total}} = 20.59 \text{ kip} \quad 10.30 \text{ tons} \]

**Associated Quasi-Static Vertical Force** (AASHTO Sec 6.1.2.3.2)

\[ \alpha \text{ check: } (eq 6.1.2.3.2-3 \text{ or eq 6.1.2.3.2-4}) \]

\[ \check{\alpha} = 18.06 < W = 64.33 \text{ ft} \quad \text{Use eq 6.1.2.3.2-3 for } \alpha \]

Referene Vertical Force: (eq 6.1.2.3.2-2)

\[ F_{v-ah} = 4.98 \text{ kip/ft} \]

\[ \text{Length of Bridge} = 27.00 \text{ ft} \]
\[ F_{v-ah \text{ Total}} = 51.93 \text{ kip} \quad 25.96 \text{ tons} \]

**Associated Vertical Slamming Forces** (AASHTO Sec 6.1.2.3.3)

*Note: Slamming forces calculated using the same method as AASHTO sec 6.1.2.2.2)

\[ F_{s} = 1.403 \text{ kip/ft} \]

\[ \text{Length of Bridge} = 27.00 \text{ ft} \]
Fs Total = 37.92 kips  18.96 tons

**Associated Moment About Trailing Edge:** (AASHTO Sec 6.1.2.3.4)

Reference Moment: (eq 6.1.2.3.4-2)

\[ M_{ah} = 147.46652 \text{ (kip-ft)} \]

**Associated Moment about Trailing Edge:** (eq 6.1.2.3.4-1)

\[ M_{ah} = 124.02284 \text{ (kip-ft)} \]

Length of Bridge = 27.00 ft

\[ M_{ah} \text{ Total} = 3348.62 \text{ kip-ft} 1674.31 \text{ ton-ft} \]

**Resulting Maximum Horizontal Wave Force and Associated Forces and Moments:** (AASHTO sec 6.1.2.3)

\[ F_{h-max} \text{ Total} = 20.59 \text{ kips} \text{ (Maximum Horizontal Wave Force)} \]

\[ F_{v-ah} \text{ Total} = 51.93 \text{ kips} \text{ (Quasi-Static Vertical Force)} \]

\[ F_s \text{ Total} = 37.92 \text{ kips} \text{ (Vertical Slamming Force)} \]

\[ M_{ah} \text{ Total} = 3348.62 \text{ kip-ft} \text{ (Associated Moment about Trailing Edge)} \]

**Current Loads on Superstructure:** (AASHTO Sec 6.1.2.4)

*Note: Current loads are not considered in this study.*

\[ F_{hc} = 0 \text{ kips} \]
Kalihi Bridge: (single span)

Method for Estimating Wave Forces on Bridge Superstructures

\[ \lambda = 80.54 \text{ ft} \]  
\[ W = 88.33 \text{ ft} \]

Constant Coefficients:

- 50 year wind speed = 105 mph (from AASHTO Fig 6.2.2.5-1 b)
- \( \mu = 32.2 \) \( \text{bsec}^{-2} \)

\[ \eta_{\text{max}} = 3.13 \text{ ft} \]

Wave Calculations:

### Bridge Properties:
- Bridge Deck Width = 86.33 ft
- Bridge Deck Length = 27.00 ft
- Girder to Girder Width = 88.33 ft
- Height of girder = 1.83 ft
- Height of railing = 3.73 ft
- Deck Thickness = 0.67 ft
- Water surface to bot. of girder = 4.00 ft
- Girder to Girder Width = 86.33 ft
- Height of girder = 1.83 ft
- Height of railing = 3.73 ft
- Water surface to bot. of girder = 4.00 ft

### Design Wave Parameters: (AASHTO Sec 6.2.4.2)

#### Determination of wave period:
- 50 year wind speed = 154.00 ft/sec
- Gust Period (\( t \)) = 3 sec

#### Determination of time duration to develop fetch limited waves:
- \( t = 2081.03 \text{ sec} \) < 3600 sec

#### Iteration Process: (until \( t \) converges)

**First Iteration:**
- \( U_1 = 110.17 \) \( \text{ft/sec} \)

**Second Iteration:**
- \( U_2 = 175.12 \) \( \text{ft/sec} \)
- \( \eta_{\text{max}} = 0.006 \)
- \( T_p = 5.80 \) \( \text{sec} \)
- \( t = 2994.55 \) \( \text{sec} \)

**Third Iteration:**
- \( U_3 = 109.45 \) \( \text{ft/sec} \)
- \( \eta_{\text{max}} = 0.006 \)
- \( T_p = 5.49 \) \( \text{sec} \)
- \( t = 3012.77 \) \( \text{sec} \)

**Fourth Iteration:**
- \( U_4 = 109.43 \) \( \text{ft/sec} \)
- \( \eta_{\text{max}} = 0.006 \)
- \( T_p = 5.49 \) \( \text{sec} \)
- \( t = 3012.77 \) \( \text{sec} \)

**Fifth Iteration:**
- \( U_5 = 109.43 \) \( \text{ft/sec} \)
- \( \eta_{\text{max}} = 0.006 \)
- \( T_p = 5.49 \) \( \text{sec} \)
- \( t = 3012.77 \) \( \text{sec} \) (the value of \( t \) has converged)

#### Wave Height and wave length:

- \( H_s = 3.73 \) \( \text{ft} \)
- \( \lambda = 80.54 \) \( \text{ft} \)
- \( H_w = 6.71 \) \( \text{ft} \)

Maximum Wave Height: (least of the following)

- \( H_{\text{max}} = 6.71 \) \( \text{ft} \)
- \( H_{\text{max}} \leq 0.65 \text{ds} = 4.47 \) \( \text{ft} \)
- \( H_{\text{max}} \leq \lambda / 7.0 = 11.51 \) \( \text{ft} \)

Therefore \( H_{\text{max}} = 4.47 \) \( \text{ft} \)

### Breaking Storm Wave Properties:

- \( T_p = 5.49 \) \( \text{sec} \)
Maximum Quasi-Static Vertical Force and Associated Forces and Moments: (AASHTO Sec 6.1.2.2)

Maximum Quasi-Static Vertical Force: (AASHTO Sec 6.1.2.2.1)

- Determination of Formulas and Parameters: (eq 6.1.2.2.1-1)
  - \( W_{\text{hat}} = \frac{46.95}{46.95} \Rightarrow W_{\text{hat}} = W_{\text{hat}} \)
  - \( \eta_{\text{max}} - Z_{\text{c}} = 3.50 > 2.50 \Rightarrow \beta = 1.00 \)
  - \( x = 0.05442 \)
  - \( y = 0.582857 \)
  - For Girder Spans: (eq 6.1.2.2.1 a)
    - \( b_0 = -0.771333 \), \( b_4 = -0.00041 \)
    - \( b_1 = 56.37 \), \( b_5 = 0.27607 \)
    - \( b_2 = 0.050533 \), \( b_6 = 5.41 \)
  - Tapped Air Factor:
    - \( A_{\text{air}} = 0.0073638 \)
    - \( B_{\text{air}} = 0.2576135 \)
  - \( \frac{\eta_{\text{max}} - Z_{\text{c}}}{d_{\text{g}}} = 1.9068273 > 1 \)
  - \( %\text{Air} = 100.00 \)
  - Assume 50% air pocket:
    - \( %\text{Air} = 50 \)
  - TAF = 0.6259827 < 1 (O.K.)
  - TAF = 0.6259827

Quasi-Static Vertical Force: (eq 6.1.2.2.1-1)
- \( F_{v_{-\text{max}}} = 3.3179007 \) kip/ft
- Length of Bridge = 27.00 ft
- \( F_{v_{-\text{max}}} \) Total = 89.58 kips 44.79 tons

Associated Vertical Slamming Force: (AASHTO Sec 6.1.2.2.2)
- \( B = -1.2473094 \)
- \( Z_c / \eta_{\text{max}} = -0.1183678 < 0 \)
- \( A = 0.0298363 \)

Vertical Slamming Force: (eq 6.1.2.2.2-1)
- \( F_s = 1.4043874 \) kip/ft
- Length of Bridge = 27.00 ft
- \( F_s \) Total = 37.92 kips 18.96 tons

- Associated Horizontal Quasi-Static Wave Force: (AASHTO Sec 6.1.2.2.3)
- (Note: Girders used on the Maipalaoa Bridge are similar to Florida Bulb - T72)

From Table 6.1.2.2.3-1: (for AASHTO Florida Bulb - T72)
- \( a_0 = -0.2076 \), \( a_5 = -0.0167 \)
- \( a_1 = 1.5772 \), \( a_6 = -0.0346 \)
- \( a_2 = -1.048 \), \( a_7 = 0.5282 \)
- \( a_3 = 0.0551 \), \( a_8 = -0.139 \)
- \( a_4 = 0.093 \)
- \( x = 0.5612067 \)
- \( y = 0.05442 \)

Horizontal Quasi-Static Wave Force: (eq 6.1.2.2.3)
\[ F_{h-av} = 0.2232638 \text{ kip/ft} \]

**Length of Bridge = 27.00 ft**

**Fh-av Total = 6.03 kips 3.01 tons**

**Associated Moment about the Trailing Edge Due to the Quasi-static and Slamming Forces** (AASHTO Sec 6.1.2.2.4)

For Girder Spans:

\[ a_m = 0.8481354 \text{ ft} \]
\[ b_m = -0.0513498 \text{ ft} \]
\[ c_m = -0.0045576 \text{ ft} \]
\[ W' = 2.00 \text{ ft} \]
\[ W^* = 86.33 \text{ ft} \]

**Associated Moment about Trailing Edge** (eq 6.1.2.2.4-6)

\[ M_{t-av} = 86.60 \text{ (kip-ft)} \]

**Length of Bridge = 27.00 ft**

**Mt-av Total = 2338.26 kip-ft 1169.13 tons-ft**

**Resulting Quasi-Static Vertical Force and Associated Forces and Moments** (AASHTO sec 6.1.2.2)

**Fv-max Total = 89.58 kips** (Quasi-Static Vertical Force)

**Fs Total = 37.92 kips** (Vertical Slamming Force)

**Fh-av Total = 6.03 kips** (Quasi-Static Horizontal Force)

**Mt-av = 2338.26 kip-ft** (Associated Moment about Trailing Edge)

**Maximum Horizontal Wave Force and Associated Forces and Moments** (AASHTO Sec 6.1.2.3)

**Maximum Horizontal Wave Force** (AASHTO Sec 6.1.2.3.1)

\[ \omega \text{ check: } \sqrt{6.1.2.3.1-3 \text{ or } eq 6.1.2.3.1-4} \]

\[ \check{\omega} = 63.74 \text{ ft} \]

**Reference Horizontal Force** (eq 6.1.2.3.1-2)

\[ F_{h-max} = 4.58 \text{ kip/ft} \]

**Horizontal Wave Force** (eq 6.1.2.3.1-1)

\[ F_{h-max} = 0.7626165 \text{ kip/ft} \]

**Length of Bridge = 27.00 ft**

**Fh-max Total = 20.59 kip 10.30 tons**

**Associated Quasi-Static Vertical Force** (AASHTO Sec 6.1.2.3.2)

\[ \alpha \text{ check: } \sqrt{6.1.2.3.2-3 \text{ or } eq 6.1.2.3.2-4} \]

\[ \check{\alpha} = 18.06 \text{ ft} \]

**Reference Vertical Force** (eq 6.1.2.3.2-2)

\[ F_{v-ah} = 4.84 \text{ kip/ft} \]

**Quasi-Static Vertical Wave Force** (eq 6.1.2.3.2-1)

\[ F_{v-ah} = 1.8842961 \text{ kip/ft} \]

**Length of Bridge = 27.00 ft**

**Fv-ah Total = 50.88 kip 25.44 tons**

**Associated Vertical Slamming Forces** (AASHTO Sec 6.1.2.3.3)

*Note: Slamming force is calculated using the same method as AASHTO sec 6.1.2.2.2)*

**Vertical Slamming Force** (eq 6.1.2.2.2-1)

\[ F_s = 1.4043875 \text{ kip/ft} \]

**Length of Bridge = 27.00 ft**
• Associated Moment About Trailing Edge: (AASHTO Sec 6.1.2.3.4)

  Reference Moment (eq 6.1.2.3.4-2)
  \[ M_{t-ah} = 198.41738 \text{ (kip/ft)-ft} \]

  Associated Moment about Trailing Edge (eq 6.1.2.3.4-1)
  \[ M_{t-ah} = 166.87372 \text{ (kip/ft)-ft} \]

  Length of Bridge = 27.00 ft
  \[ M_{t-ah} \text{ Total} = 4505.59 \text{ kip-ft} \]

• Resulting Maximum Horizontal Wave Force and Associated Forces and Moments: (AASHTO sec 6.1.2.3)

  \[ F_h \text{ max Total} = 20.59 \text{ kips} \]
  \[ F_v \text{ Total} = 50.88 \text{ kips} \]
  \[ F_s \text{ Total} = 37.92 \text{ kips} \]
  \[ M_{t-ah} \text{ Total} = 4505.59 \text{ kip-ft} \]

• Current Loads on Superstructure: (AASHTO Sec 6.1.2.4)

  *Note: Current loads are not considered in this study.
  \[ F_{hc} = 0 \text{ kips} \]
**Nimitz Highway at Aloha Tower: Slip Cover #2**

**Method for Estimating Wave Forces on Bridge Superstructures**

\[ \lambda = 70.34 \text{ ft} \]

AASHTO (2008)

\[ W = 67.00 \text{ ft} \]

**Constant Coefficients:**

- 50 year wind speed = 105 mph
  - (from AASHTO Fig 6.2.2.2-1 b)
  - \( W^* = 67.00 \text{ ft} \)

- Specific weight water = 0.064 kip/cubic ft
  - \( g = 32.2 \text{ ft/sec}^2 \)

- \( \eta_{\text{max}} = 2.50 \text{ ft} \)

**Wave Calculations:**

**Bridge Properties:**

- Bridge Deck Width = 67.00 ft
- Bridge Deck Length = 178.41 ft
- Girder to Girder Width = 67.00 ft
- Girder to Girder Width = 2.50 ft
- Height of railing = 1.67 ft
- Height of girder = 2.50 ft
- Deck Thickness = 1.29 ft
- Zc = 4.33 ft

- Storm Water Level = 9.83 ft
- Elevation to bot. of girder = 3.58 ft
- Elevation to bot. of deck = 12.33 ft
- Elev. to bot. of girder = 9.83 ft
- Elev. to bot. of deck = 12.33 ft

- Storm Surge + Local Wind Setup = 5.50 ft

**Design Wave Parameters:**

- (AASHTO Sec 6.2.2.4)

**Determination of wave period:**

- 50 year wind speed = 154.00 ft/sec
- Gust Period (t) = 3 sec
- 100 year wind speed = 164.78 ft/sec
- \( Ut^* = 287.33 \text{ ft/sec} \)

- \( d = 6.88 \text{ ft} \)
  - (average water depth over fetch length)
- Fetch Length = 4767610.00 ft
- \( \left( \frac{g \times d}{(Ut^*)^2} \right) = 0.003 \)
- \( T_p = 6.07 \text{ sec} \)

**Determination of time duration to develop fetch limited waves:**

- \( t = 1947.76 \text{ sec} < 3600 \text{ sec} \)

- To compute \( U_{1hr} \) use eq 6.2.2.4-4

**Iteration Process:** (until 't' converges)

- First Iteration:
  - \( U_t = 110.32 \text{ ft/sec} \)
- Second Iteration:
  - \( U_t^* = 179.42 \text{ ft/sec} \)
- Third Iteration:
  - \( U_t = 109.56 \text{ sec} \)
  - \( T_p = 5.35 \text{ sec} \)
  - \( t = 2800.42 \text{ sec} \)
- Fourth Iteration:
  - \( U_t = 109.55 \text{ sec} \)
  - \( T_p = 5.33 \text{ sec} \)
  - \( t = 2818.03 \text{ sec} \)
- Fifth Iteration:
  - \( U_t = 109.55 \text{ sec} \)
  - \( T_p = 5.33 \text{ sec} \)
  - \( t = 2818.30 \text{ sec} \)

- \( T_p = 5.33 \text{ sec} \)

- \( t = 2818.30 \text{ sec} \) (the value of 't' has converged)

**Wave height and wave length:**

- \( H_s = 3.53 \text{ ft} \)
- \( \lambda = 70.34 \text{ ft} \)

- Maximum Wave Height:
  - \( H_{\text{max}} = 6.35 \text{ ft} \)
  - \( H_{\text{max}} < 0.65d = 3.58 \text{ ft} \)
  - \( H_{\text{max}} < \frac{\lambda}{7.0} = 9.50 \text{ ft} \)

- Therefore \( H_{\text{max}} = 3.58 \text{ ft} \)

**Resulting Storm Wave Properties:** (see Fig)

- \( T_p = 5.33 \text{ sec} \)
Maximum Quasi-Static Vertical Force and Associated Forces and Moments: (AASHTO Sec 6.1.2.2)

Determination of Design Parameters (eq 6.1.2.2.1-1)

- Maximum Quasi-Static Vertical Force: (AASHTO Sec 6.1.2.2.1)
  - Determination of \( Fv_{max} \) parameters:
    - \( W_{hat} = -50.09 \) ft
    - \( \frac{W_{hat}}{W} = -0.75 < 0.15 \) Therefore \( W_{hat} = 0.15 W \)
    - \( \eta_{max} - Zc = -1.83 < 0 \) dB = 3.79 ft
    - \( \beta = 1.00 \)
    - \( x = 0.0998238 \)
    - \( y = -0.75212 \)
  
  For Slab Spans: (eq 6.1.2.2.1 b)
  - \( b_0 = -0.147175 \)
  - \( b_1 = 40.357917 \)
  - \( b_2 = 0.0397417 \)
  - \( b_3 = -93.38042 \)
  
  Tapped Air Factor:
  - \( A_{air} = 0.0114354 \)
  - \( B_{air} = 0.1053956 \)
  
  \( \frac{(\eta_{max} - Zc)}{dg} = -0.73233 < 1 \)
  
  %Air = variable
  
  However, the bridge is not a girder type bridge therefore:
  
  %Air = 0
  
  TAF = 1 > 1 (OK.)
  
  Quasi-Static Vertical Force: (eq 6.1.2.2.1-1)
  - \( Fv_{max} = -4.844062 \) kip/ft
  - Length of Bridge = 178.41 ft
  - \( Fv_{max} \) Total = -864.23 kips -432.11 tons

Associated Vertical Slamming Force: (AASHTO Sec 6.1.2.2.2)

- \( B = 1.716191 \)
- \( Zc / \eta_{max} = 1.7316017 > 0 \)
- \( A = 0.0575009 \)

Vertical Slamming Force: (eq 6.1.2.2.2-1)

- \( Fs = 0.0002861 \) kip/ft
- Length of Bridge = 178.41 ft
- \( Fs \) Total = 0.05 kips 0.03 tons

Associated Horizontal Quasi-Static Wave Force: (AASHTO Sec 6.1.2.2.3)

*Note: Girders used on the New Makaha #3A Bridge are similar to 36 in Adjacent Box Girders

D-35
Horizontal Quasi-Static Wave Force: \( \text{(eq 6.1.2.2.3)} \)
\[
F_{h-av} = -0.129509 \text{ kip/ft} < 0 \quad \text{therefore set equal to zero}
\]

Length of Bridge = 178.41 ft
\( F_{h-av} \) Total = 0.00 kips 0.00 tons

- Associated Moment about the Trailing Edge Due to the Quasi-static and Slamming Forces: \( \text{(AASHTO Sec 6.1.2.2.4)} \)

For Slab Span:
\[
\begin{align*}
a_m &= 0.0317498 \text{ ft} \\
b_m &= -0.042158 \text{ ft} \\
c_m &= -0.0049 \text{ ft} \\
w' &= 0 \text{ ft} \\
w^* &= 67.00 \text{ ft}
\end{align*}
\]

- Associated Moment about Trailing Edge: \( \text{(eq 6.1.2.2.4-1)} \)
\[
M_{t-av} = -0.48 \text{ (kip/ft)-ft}
\]

Length of Bridge = 178.41 ft
\( M_{t-av} \) Total = -84.88 kip-ft -42.44 tons-ft

- Resulting Quasi-Static Vertical Force and Associated Forces and Moments: \( \text{(AASHTO Sec 6.1.2.2)} \)
\[
F_{v-max} \text{ Total} = -864.23 \text{ kips (Quasi-Static Vertical Force)}
\]
\[
F_{s} \text{ Total} = 0.05 \text{ kips (Vertical Slamming Force)}
\]
\[
F_{h-av} \text{ Total} = 0.00 \text{ kips (Quasi-Static Horizontal Force)}
\]

- Maximum Horizontal Wave Force and Associated Forces and Moments: \( \text{(AASHTO Sec 6.1.2.3)} \)

Maximum Horizontal Wave Force: \( \text{(AASHTO Sec 6.1.2.3.1)} \)
\[
\omega \text{ check:} \quad \text{eq 6.1.2.3.1-3 or eq 6.1.2.3.1-4}
\]
\[
\omega = 10.12 \text{ ft} \quad \text{Use eq 6.1.2.3.1-3 for } \omega
\]

Reference Horizontal Force: \( \text{(eq 6.1.2.3.1-2)} \)
\[
F_{h-max} = 0.66 \text{ kip/ft}
\]

Horizontal Wave Force: \( \text{(eq 6.1.2.3.1-1)} \)
\[
F_{h-max} = \#NUM! \text{ kip/ft}
\]

Length of Bridge = 178.41 ft
\( F_{h-max} \) Total = \#NUM! kip \#NUM! tons

- Associated Quasi-Static Vertical Force: \( \text{(AASHTO Sec 6.1.2.3.2)} \)

\( \beta \text{ check:} \quad \text{eq 6.1.2.3.2-3 or eq 6.1.2.3.2-4}
\]
\[
\beta = -19.27 \text{ ft} \quad \text{Use eq 6.1.2.3.2-3 for } \beta
\]

Reference Vertical Force: \( \text{(eq 6.1.2.3.2-2)} \)
\[
F_{v-ah} = 2.26 \text{ kip/ft}
\]

Quasi-Static Vertical Wave Force: \( \text{(eq 6.1.2.3.2-1)} \)
\[
F_{v-ah} = \#NUM! \text{ kip/ft}
\]

Length of Bridge = 178.41 ft
\( F_{v-ah} \) Total = \#NUM! kip \#NUM! tons

- Associated Vertical Slamming Forces: \( \text{(AASHTO Sec 6.1.2.3.3)} \)

\( *\text{Note: Slamming force is calculated using the same method as AASHTO sec 6.1.2.2.2)} \)

Vertical Slamming Force: \( \text{(eq 6.1.2.2.2-1)} \)
\[
F_{s} = 0.000286 \text{ kip/ft}
\]
Length of Bridge = 178.41 ft
Fx Total = 0.05 kips 0.03 tons

**Associated Moment About Trailing Edge:** (AASHTO Sec 6.1.2.3.4)

- Reference Moment: (eq 6.1.2.3.4-2)
  \[ M_{c\text{-ah}} = \text{NUM!} \text{ kip-ft} \]

- Associated Moment about Trailing Edge: (eq 6.1.2.3.4-1)
  \[ M_{t\text{-ah}} = \text{NUM!} \text{ kip-ft} \]

Length of Bridge = 178.41 ft
Mt-ah Total = #NUM! kip-ft #NUM! ton-ft

**Resulting Maximum Horizontal Wave Force and Associated Forces and Moments:** (AASHTO sec 6.1.2.3)

- Fh-max Total = #NUM! kips (Maximum Horizontal Wave Force)
- Fv-ah Total = #NUM! kips (Quasi-Static Vertical Force)
- Fs Total = 0.05 kips (Vertical Slamming Force)
- Mt-ah Total = #NUM! kip-ft (Associated Moment about Trailing Edge)

**Current Loads on Superstructure:** (AASHTO Sec 6.1.2.4)

*Note: Current loads are not considered in this study.*

Fhc = 0 kips
Nimitz Highway at Aloha Tower: Slip Cover #3
Method for Estimating Wave Forces on Bridge Superstructures

AASHTO (2008)

**Wave Calculations:**

- **Bridge Properties:**
  - Bridge Deck Width = 41.00 ft
  - Bridge Deck Length = 240.00 ft
  - Girder to Girder Width = 41.00 ft
  - Height of girder = 2.50 ft
  - Height of railing = 1.67 ft
  - Deck Thickness = 1.29 ft
  - Height of girder = 2.50 ft
  - Storm Water Level = 4.33 ft

- **Design Wave Parameters:**
  - (AASHTO Sec 6.2.2.4)

  ○ **Determination of wave period:**
    - 50 year wind speed = 154.00 ft/sec
    - Gust Period (t) = 3 sec
    - 100 year wind speed = 164.78 ft/sec
    - Ut* = 287.33 ft/sec
    - ds = 5.50 ft
    - (from NFIP Flood Hazard Assessment Tool)
  
  ○ **Determination of time duration to develop fetch limited waves:**
    - t = 1947.76 sec < 3600 sec
    - U_1hr = 109.17 ft/sec

  ○ **Resulting Storm Wave Properties:**
    - (see Fig)
    - Tp = 5.33 sec

- **Constant Coefficients:**
  - 50 year wind speed = 105 mph
  - Specific weight water = 0.064 kip/cubic ft
  - g = 32.2 ft/sec^2
  - η_max = 2.50 ft

- **Waves Height and wave length:**
  - Hs = 3.53 ft
  - Wave Length (λ) = 70.34 ft
  - Hmax = 6.35 ft

- **Maximum Wave Height:**
  - Hmax > 0.40Hs = 3.58 ft

- **Sketch:**
  - Storm Wave Level
  - Storm Surge + Local Wind Setup
  - Bed

---

**Iteration Process:** (until 't' converges)

**First Iteration:**
- Ut = 110.32 ft/sec
- Ut* = 173.92 ft/sec
- [(g*d)/(Ut*)^2] = 0.007
- Tp = 5.33 sec
- t = 2800.42 sec

**Second Iteration:**
- Ut = 109.55 sec
- Ut* = 173.90 ft/sec
- [(g*d)/(Ut*)^2] = 0.007
- Tp = 5.33 sec
- t = 2818.29 sec

**Third Iteration:**
- Ut = 109.55 sec
- Ut* = 173.90 ft/sec
- [(g*d)/(Ut*)^2] = 0.007
- Tp = 5.33 sec
- t = 2818.30 sec

**Fourth Iteration:**
- Ut = 109.55 sec
- Ut* = 173.90 ft/sec
- [(g*d)/(Ut*)^2] = 0.007
- Tp = 5.33 sec
- t = 2818.30 sec

**Fifth Iteration:**
- Ut = 109.55 sec
- Ut* = 173.90 ft/sec
- [(g*d)/(Ut*)^2] = 0.007
- Tp = 5.33 sec
- t = 2818.30 sec

**Sixth Iteration:**
- Ut = 109.55 sec
- Ut* = 173.90 ft/sec
- [(g*d)/(Ut*)^2] = 0.007
- Tp = 5.33 sec
- t = 2818.30 sec

(the value of 't' has converged)


- **Maximum Quasi-Static Vertical Force and Associated Forces and Moments** (AASHTO Sec 6.1.2.2)

  - **Maximum Quasi-Static Vertical Force** (AASHTO Sec 6.1.2.2.1)
    - Determination of Force parameters (eq 6.1.2.2.1-1)
      
      | Parameter | Value |
      |-----------|-------|
      | $W_{\text{hat}}$ | -56.09 ft |
      | $W_{\text{hat}} / W$ | -1.22 |
      | $\eta_{\text{max}} - Z_c$ | -1.83 |
      | $\beta$ | 1.00 |
      | $\eta_{\text{max}} - Z_c = \eta_{\text{max}} - Z_c$ | -1.83 |
      | $\beta = 1.00$ |
      | $x = 0.0998288$ |
      | $y = -0.752123$ |

    - For Slab Spans (eq 6.1.2.2.1 b)
      
      | Parameter | Value |
      |-----------|-------|
      | $b_0$ | -0.147175 |
      | $b_1$ | 40.357917 |
      | $b_2$ | 0.0397417 |
      | $b_3$ | -93.38042 |
      | $b_4$ | 0.00066 |
      | $b_5$ | 0.613995 |
      | $b_6$ | 31.43333 |
      | $b_7$ | -0.147175 |
    - Tapped Air Factor
      
      | Parameter | Value |
      |-----------|-------|
      | $A_{\text{air}}$ | 0.0107478 |
      | $B_{\text{air}}$ | 0.1284632 |
    - Quasi-Static Vertical Force (eq 6.1.2.2.1)
      
      | Parameter | Value |
      |-----------|-------|
      | $F_{v_{\text{max}}}$ | -4.844062 kip/ft |
      | $\eta_{\text{max}} - Z_c = \eta_{\text{max}} - Z_c$ | 9.11253 |
      | $\beta = 1.00$ |
      | $x = 0.0998288$ |
      | $y = -0.752123$ |
      | $W_{\text{max}} = 670.00 kips$ |
      | $W_{\text{max}} = 303.29 tons$ |

  - **Associated Vertical Slamming Force** (AASHTO Sec 6.1.2.2.2)
    - Vertical Slamming Force (eq 6.1.2.2.2-1)
      
      | Parameter | Value |
      |-----------|-------|
      | $A_{\text{air}}$ | 0.0574009 |
      | $B_{\text{air}}$ | 0.0508238 |
    - Quasi-Static Vertical Force (eq 6.1.2.2.1)
      
      | Parameter | Value |
      |-----------|-------|
      | $F_{v_{\text{max}}}$ | -4.844062 kip/ft |
      | $\eta_{\text{max}} - Z_c = \eta_{\text{max}} - Z_c$ | 9.11253 |
      | $\beta = 1.00$ |
      | $x = 0.0998288$ |
      | $y = -0.752123$ |
      | $W_{\text{max}} = 670.00 kips$ |
      | $W_{\text{max}} = 303.29 tons$ |

- **Associated Horizontal Quasi-Static Wave Force** (AASHTO Sec 6.1.2.2.3)
  - Note: Girders used on the New Makaha #3A Bridge are similar to 36 in Adjacent Box Girders

  - From Table 6.1.2.2.3-1 (for Box Girders)
    
    | Parameter | Value |
    |-----------|-------|
    | $a_0$ | -0.03086 |
    | $a_1$ | -1.4168 |
    | $a_2$ | -1.1168 |
    | $a_3$ | 0.3455 |
    | $a_4$ | -0.0438 |
    | $x = -0.33542$ |
    | $y = 0.0908238$ |
**Horizontal Quasi-Static Wave Force:** (eq 6.1.2.2.3)

\[ F_{h-av} = -0.21334 \text{ kip/ft} \]

Length of Bridge = 240.00 ft

\[ F_{h-av \text{ Total}} = -51.20 \text{ kips} - 25.60 \text{ tons} \]

**Associated Moment about the Trailing Edge Due to the Quasi-static and Slamming Forces:** (AASHTO Sec 6.1.2.2.4)

For Slab Spans:

\[ a_{m} = 0.8317458 \text{ ft} \]
\[ b_{m} = -0.042158 \text{ ft} \]
\[ c_{m} = -0.0049 \text{ ft} \]
\[ W' = 0.00 \text{ ft} \]
\[ W* = 41.00 \text{ ft} \]

\[ M_{t-av} = -0.76 \text{ (kip/ft)-ft} \]

Length of Bridge = 240.00 ft

\[ M_{t-av \text{ Total}} = -183.42 \text{ kip-ft} - 91.71 \text{ tons-ft} \]

**Resulting Quasi-Static Vertical Force and Associated Forces and Moments:** (AASHTO Sec 6.1.2.2)

\[ F_{v-max \text{ Total}} = -1162.57 \text{ kips} \]

\[ F_{s \text{ Total}} = 0.07 \text{ kips} \]

\[ F_{h-av \text{ Total}} = -51.20 \text{ kips} \]

\[ M_{t-av \text{ Total}} = -183.42 \text{ kip-ft} - 91.71 \text{ tons-ft} \]

**Maximum Horizontal Wave Force and Associated Forces and Moments:** (AASHTO Sec 6.1.2.3)

**Maximum Horizontal Wave Force:** (AASHTO Sec 6.1.2.3.1)

\[ \omega \text{ check: (eq 6.1.2.3.1-3 or eq 6.1.2.3.1-4)} \]
\[ \text{check} = 28.74 < W = 41.00 \text{ ft} \]

\[ \omega = 28.74 \text{ ft} \]

Reference Horizontal Force: (eq 6.1.2.3.1-2)

\[ F_{h-max} = 3.16 \text{ kip/ft} \]

Horizontal Wave Force: (eq 6.1.2.3.1-1)

\[ F_{h-max} \text{ Total} = \#NUM! \text{ kip} \]

Length of Bridge = 240.00 ft

\[ F_{h-max \text{ Total}} = \#NUM! \text{ kips} \]

**Associated Quasi-Static Vertical Force:** (AASHTO Sec 6.1.2.3.2)

\[ \alpha \text{ check: (eq 6.1.2.3.2.3 or eq 6.1.2.3.2.4)} \]
\[ \text{check} = -4.95 < W = 41.00 \text{ ft} \]

\[ \alpha = -4.95 \text{ ft} \]

Reference Vertical Force: (eq 6.1.2.3.2-2)

\[ F_{v-ah} = 0.58 \text{ kip/ft} \]

Quasi-Static Vertical Wave Force: (eq 6.1.2.3.2-1)

\[ F_{v-ah} \text{ Total} = \#NUM! \text{ kip} \]

Length of Bridge = 240.00 ft

\[ F_{v-ah \text{ Total}} = \#NUM! \text{ kips} \]

**Associated Vertical Slamming Forces:** (AASHTO Sec 6.1.2.3.3)

*Note: Slamming force is calculated using the same method as AASHTO section 6.1.2.2.2)

\[ F_{s} = 0.006286 \text{ kip/ft} \]
Length of Bridge = 240.00 ft
Fx Total = 0.07 kips 0.03 tons

• Associated Moment About Trailing Edge: (AASHTO Sec 6.1.2.3.4)
  Reference Moment: (eq 6.1.2.3.4-2)
  M\(\text{c-ah}\) = #NUM! kip-ft
  Associated Moment about Trailing Edge: (eq 6.1.2.3.4-1)
  M\(\text{t-ah}\) = #NUM! kip-ft

Length of Bridge = 240.00 ft
M\(\text{t-ah}\) Total = #NUM! kip-ft

<table>
<thead>
<tr>
<th>Resulting Maximum Horizontal Wave Force and Associated Forces and Moments</th>
<th>(AASHTO sec 6.1.2.3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F(\text{h-ah}) Total</td>
<td>#NUM! kips (Maximum Horizontal Wave Force)</td>
</tr>
<tr>
<td>F(\text{v-ah}) Total</td>
<td>#NUM! kips (Quasi-Static Vertical Force)</td>
</tr>
<tr>
<td>Fx Total</td>
<td>0.07 kips (Vertical Slamming Force)</td>
</tr>
<tr>
<td>M(\text{t-ah}) Total</td>
<td>#NUM! kip-ft (Associated Moment about Trailing Edge)</td>
</tr>
</tbody>
</table>

• Current Loads on Superstructure: (AASHTO Sec 6.1.2.4)
  *Note: Current loads are not considered in this study.
  F\(\text{hc}\) = 0 kips