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

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Abstract

This report 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 report 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 report 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 report 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

$$F_v = c_{v-va} * F_v^* \tag{2.3-1}$$

$$F_h = [1 + C_r * (N - 1)] * c_{h-va} * F_h^* \tag{2.3-2}$$

$$F_v^* = \gamma * (\Delta z_v) * A_v \tag{2.3-3}$$

$$F_h^* = \gamma * (\Delta z_h) * A_h \tag{2.3-4}$$

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 $c_{v\text{-}va}$ = 1 for non conservative, 2 for conservative design)

c_{h-va} = an empirical coefficient for the horizontal "varying" load (recommended value is $c_{h\text{-}va}$ = 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
γ	=	unit weight of salt water [64 lb/ft^3]
Δz_v	=	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	=	the area of the bridge contributing to vertical uplift, i.e. the projection of the bridge deck onto the horizontal plane [ft^2]
Δz_h	=	difference between the elevation of the crest of the maximum wave and the elevation of the centroid of A_h [ft]
A_h	=	the area of the projection of the bridge deck onto the vertical plane [ft^2]
η_{max}	=	$1.3H_s$ (where the maximum wave above the storm surge elevation can be no more than 0.8ds) [ft]
H_s	=	significant wave height [ft]
ds	=	still water level (including storm surge) [ft]

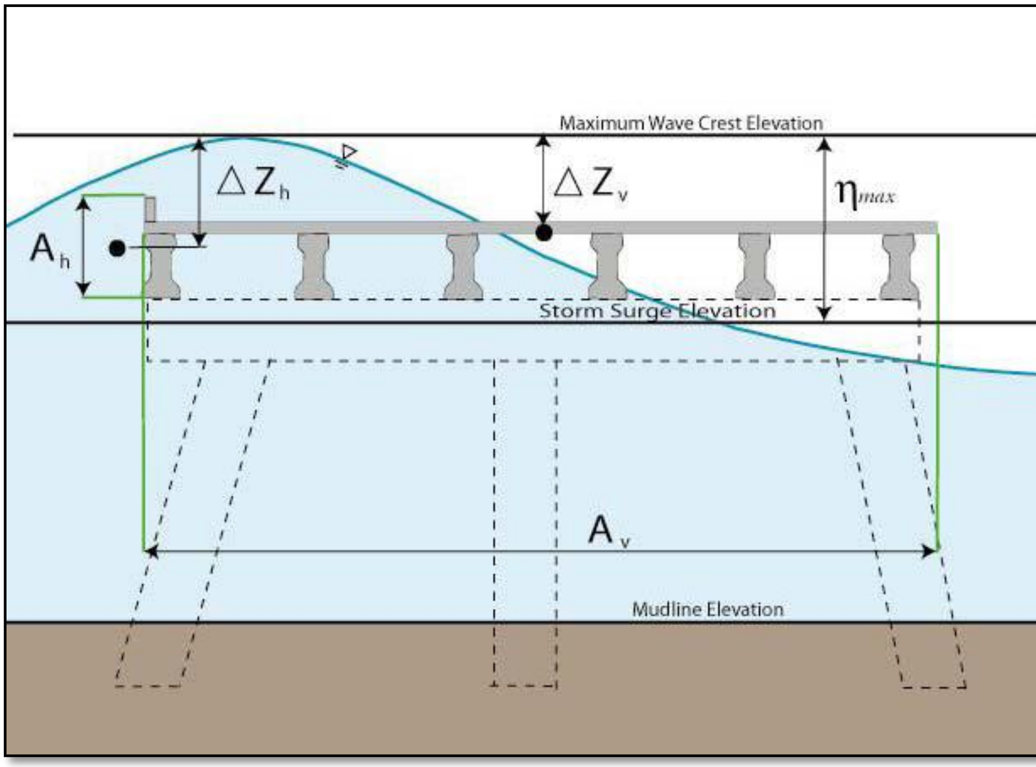


Figure 2-1: Douglass Wave Estimation Method Diagram [10]

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.

2.4 McPherson (2008): Hurricane Induced Wave and Surge Forces on Bridge Decks [11]

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

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

$$F_{Hydrostatic} = \gamma \delta_z A - F_w \tag{2.4-2}$$

$$F_{Bridge} = \gamma Vol_{Bridge} \tag{2.4-3}$$

$$F_{AirEntrapment} = (n-1)0.5\gamma \delta_G A_G \tag{2.4-4}$$

if $h \leq h\_model$,

$$F_w = \frac{1}{2}\gamma \delta A \tag{2.4-5}$$

and if $h > h\_model$,

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

The horizontal wave force is estimated by

$$F_{Total} = F_{Hydrostatic\_Front} - F_{Hydrostatic\_Back} \tag{2.4-7}$$

if $\eta_{max} < h_{deck}$,

$$F_{Hydrostatic\_Front} = 0.5 * (\eta_{max} + h - h_{girders}) H_{bridge} L_{bridge} \gamma \tag{2.4-8}$$

and if $\eta_{max} > h_{deck}$,

$$F_{Hydrostatic\_Front} = 0.5 * [(\eta_{max} + h - h_{girders}) + (\eta_{max} - h_{deck})]H_{bridge}L_{bridge}\gamma \quad (2.4\text{-}9)$$

if $SWL < h_{girders}$,

$$F_{Hydrostatic\_back} = 0 \quad (2.4\text{-}10)$$

and if $SWL > h_{girders}$,

$$F_{Hydrostatic\_back} = 0.5(h - h_{girder})^2 L_{bridge}\gamma \quad (2.4\text{-}11)$$

In the above equations

γ	=	unit weight of salt water [64 lb/ft^3]
δ_z	=	distance from the top of the bridge deck to the top of the wave [ft]
δ_G	=	height of the bridge girders [ft]
δ	=	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_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]
η_{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_{bridge}	=	height of bridge impacted by lateral wave forces [ft]
L_{bridge}	=	length of bridge impacted by lateral wave forces [ft]
SWL	=	still water level including storm surge [ft]

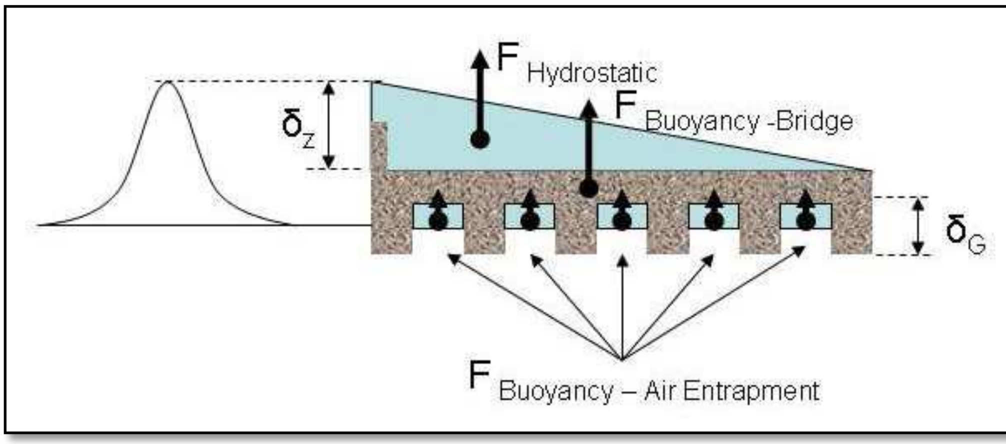


Figure 2-2: McPherson Vertical Wave Estimation Method Diagram [11]

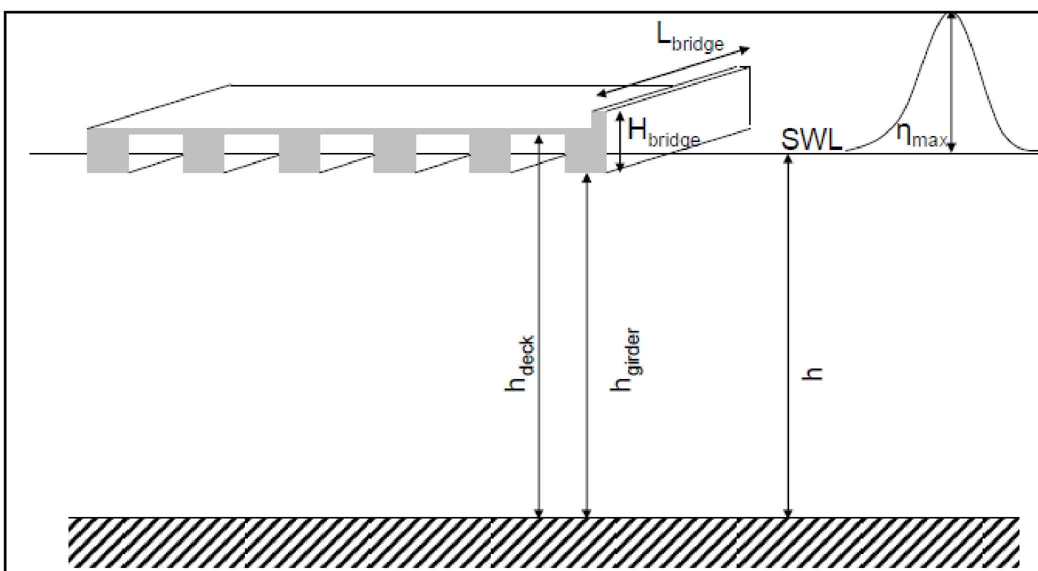


Figure 2-3: McPherson Horizontal Wave Estimation Method Diagram [11]

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.

2.5 Boon-intra (2010): Development of a Guideline for Estimating Tsunami Forces on Bridge Superstructures [13]

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

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

in which

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

The vertical tsunami force is estimated by

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

in which

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

$u_{x,max}^*$ = 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

$$F_{tw} = F_b + F_s + F_d + F_l + F_i \qquad (2.6\text{-}1)$$

in which

F_{tw}	=	total force		
F_b	=	vertical buoyancy force		
F_s	=	horizontal slamming force	$= 0.5 * \rho * C_s * A * u^2$	(2.6-2)
F_d	=	horizontal drag force	$= 0.5 * \rho * C_d * A * u^2$	(2.6-3)
F_l	=	vertical lift force	$= 0.5 * \rho * C_l * A * u^2$	(2.6-4)
F_i	=	acceleration-dependent inertia force	$= \rho * C_m * V * a$	(2.6-5)
C_s	=	slamming coefficient (range: π - 2π)		
ρ	=	mass density of sea water [slugs/ft^3]		

A = vertical deck area subjected to wave crest [ft^2]

u = horizontal fluid velocity of the wave crest [ft/sec]

C_m = inertia coefficient

V = volume of the deck inundated [ft^3]

a = water acceleration [ft/sec^2]

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

$$F_s' = F_e * F_s \qquad (2.6\text{-}6)$$

in which

F_e = dynamic loading factor = $2 * \pi * \alpha * \left(\frac{t_d}{T_n}\right)$

α = 0.5 (triangular loading) or $\frac{2}{\pi}$ (half sine loading)

t_d = impact duration [sec]

T_n = natural period of deck [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 report. 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.

2.7 Robertson, et al. (2007): Lessons from Hurricane Katrina Storm Surge on Bridges and Buildings [15]

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 complete 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 prestresssed 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 sea water (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.

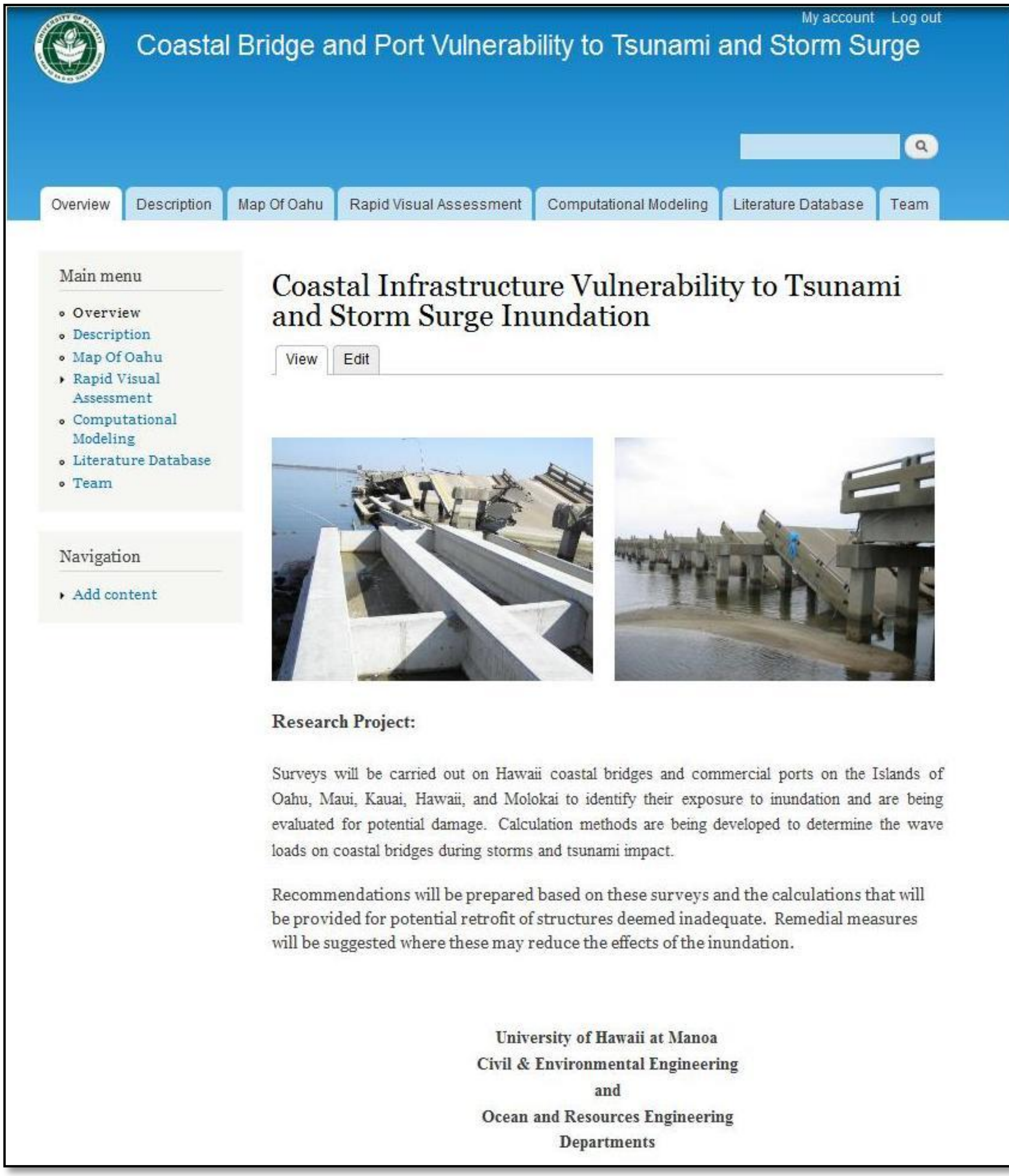


Figure 3-1: Website Homepage

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.

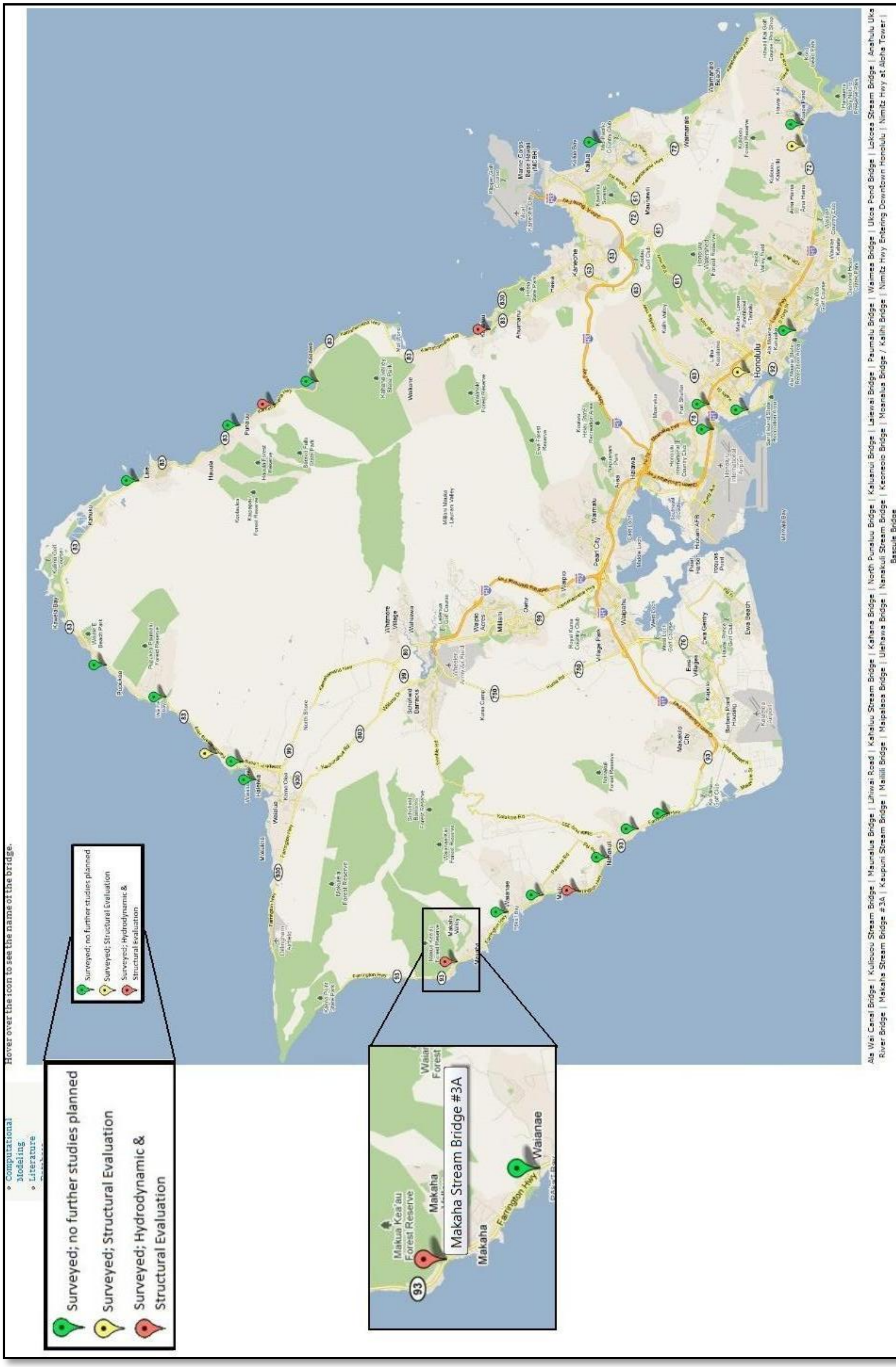


Figure 3-2: Map of Oahu

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).

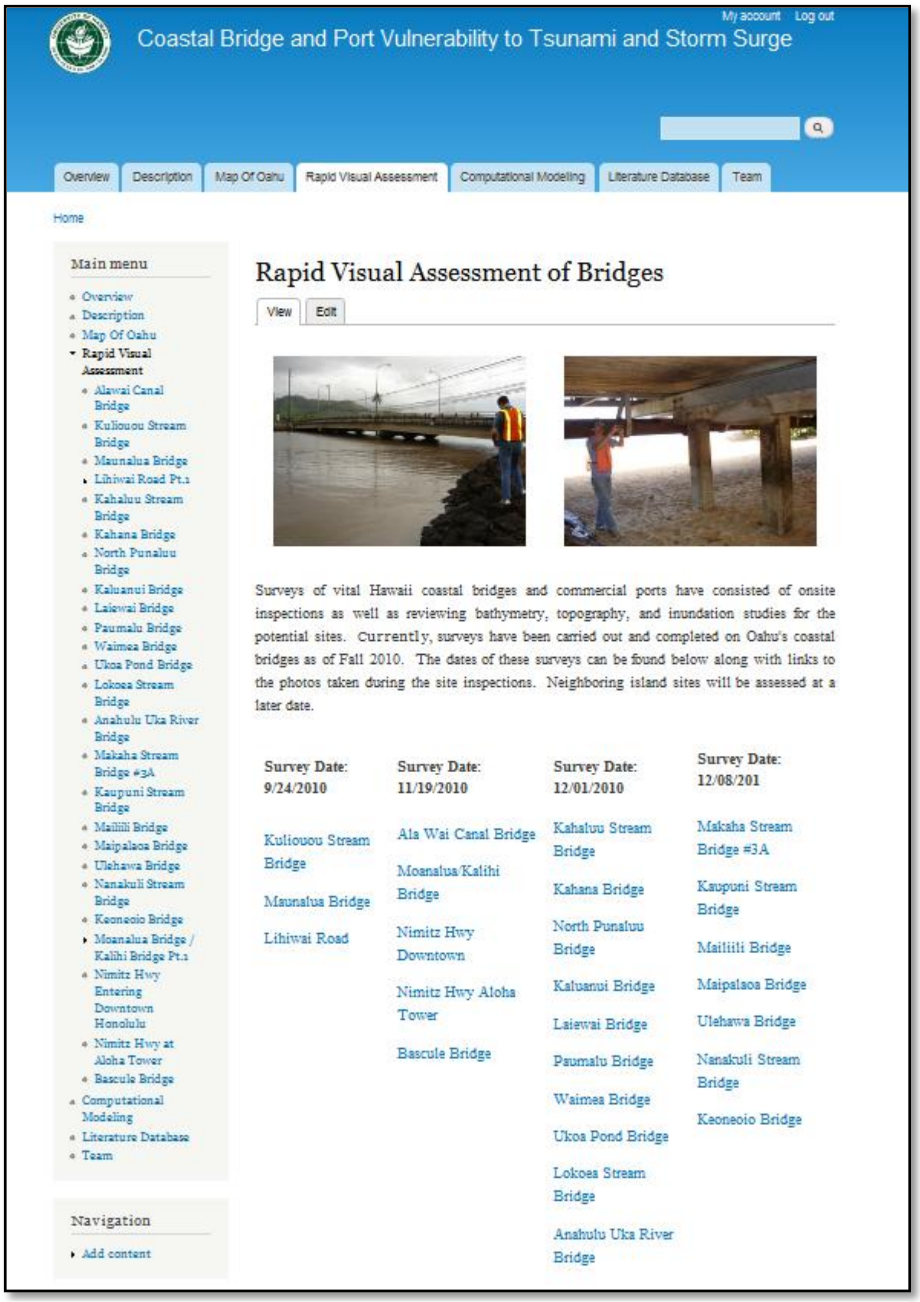


Figure 3-3: Rapid Visual Assessment Front Page

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.

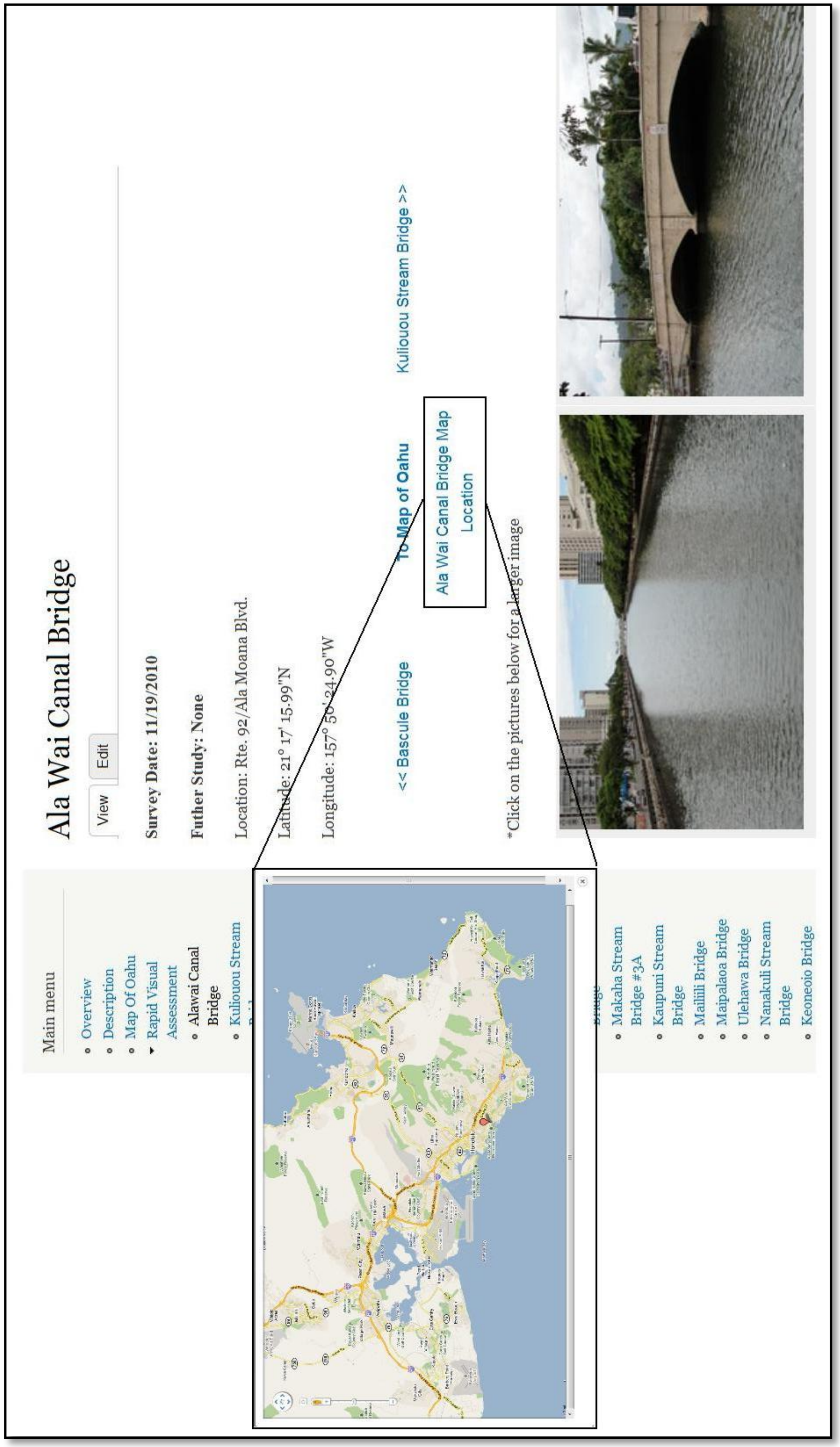


Figure 3-4: Mini Map Example

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.

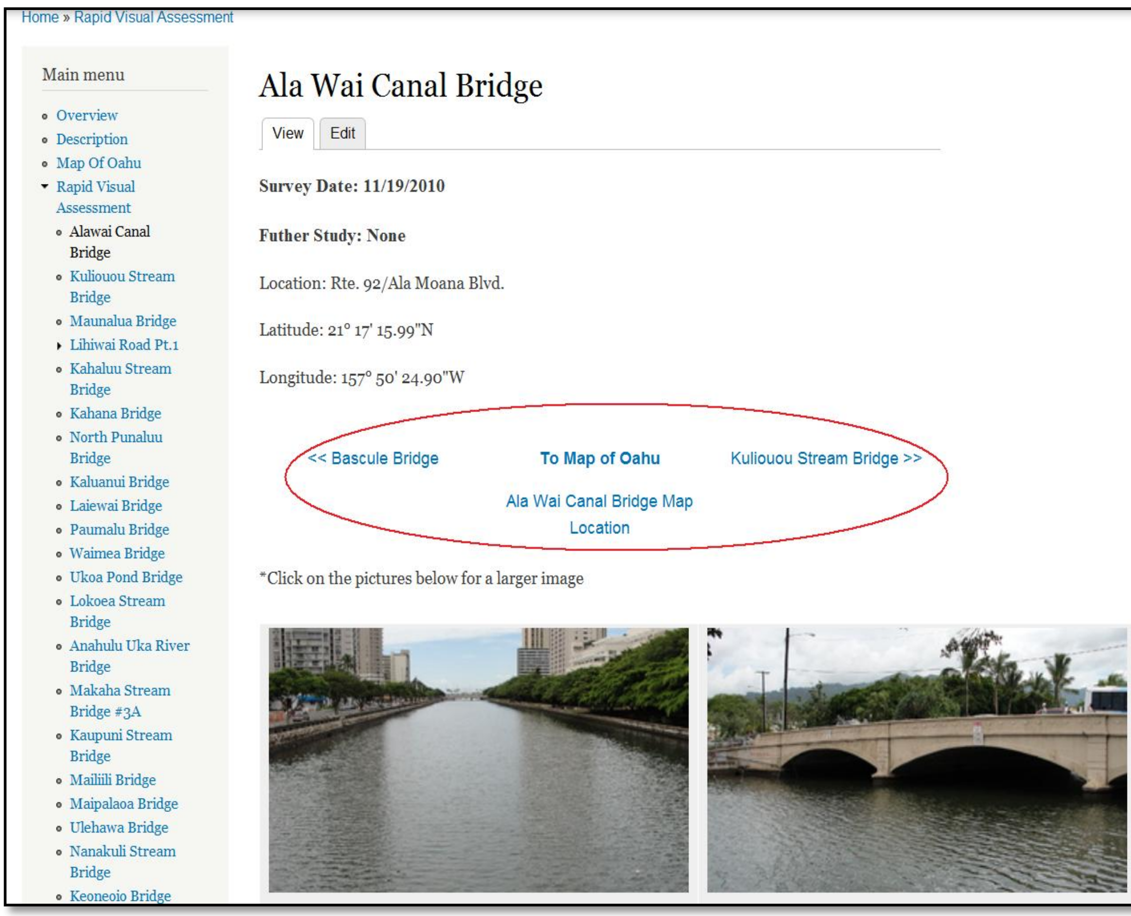


Figure 3-5: Navigation Menu

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.

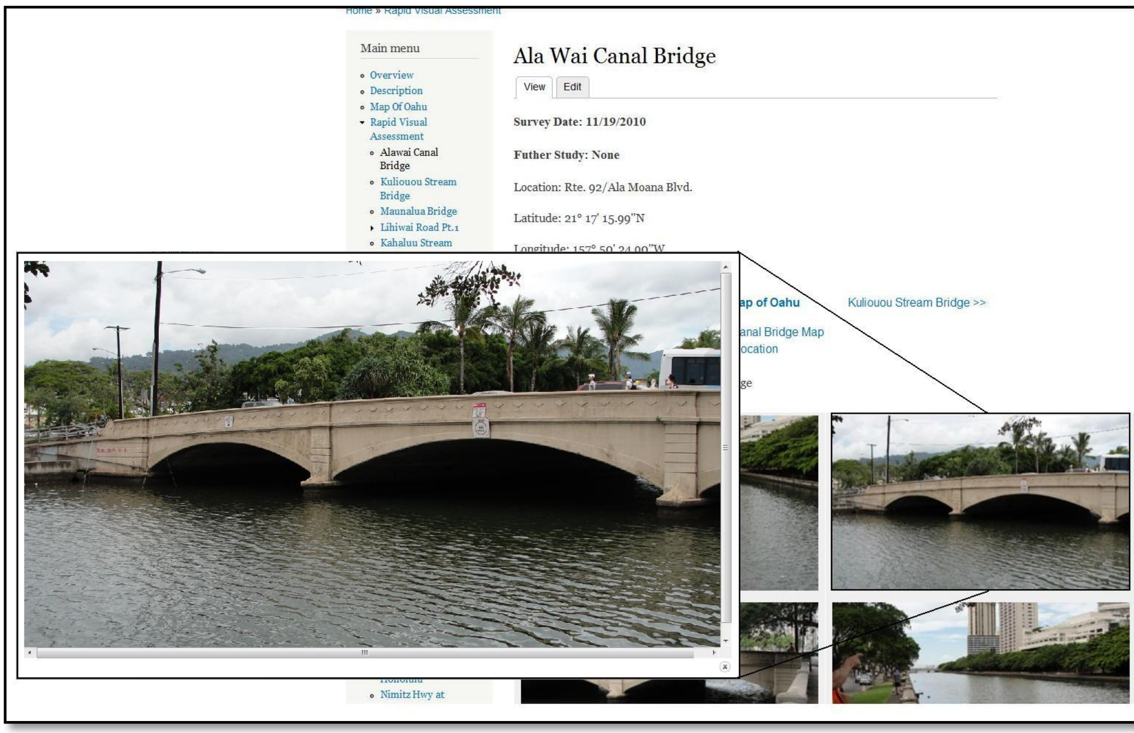


Figure 3-6: Thumbnail Picture Example

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.

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.

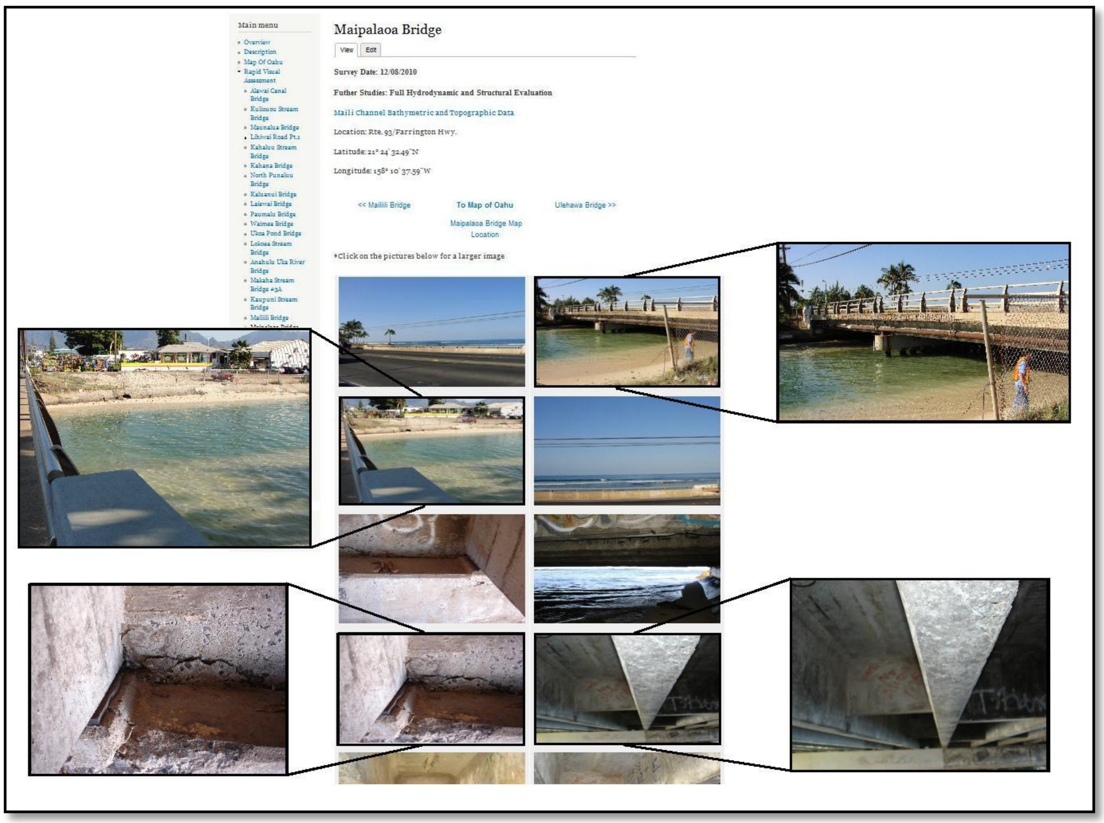


Figure 3-7: Survey Pictures

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.

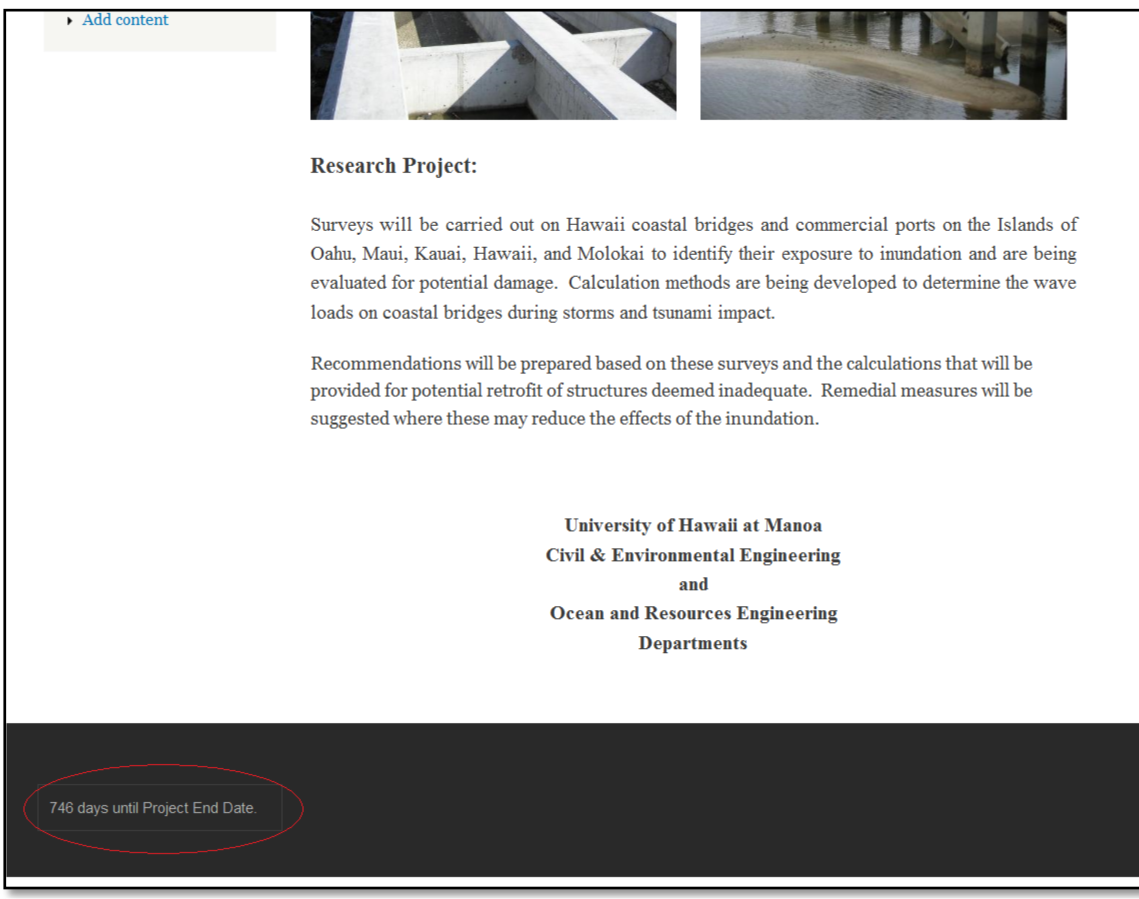


Figure 3-8: Project Timer

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 report. 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.

    - Typically this is placed on the left hand side of each page. See Figure 3-1 for an example.

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/ ft^3).

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

$$P2 = P1 + h * (\frac{\gamma}{144}) \qquad (4.2\text{-}1)$$

$$A2 = \frac{P1 * A1}{P2} \qquad (4.2\text{-}2)$$

in which,

h = height from bottom of air pockets to water elevation [ft]

γ = specific weight of water [64 lbs/ ft^3]

$P1$ = atmospheric pressure at the water surface [14.7 psi]

$P2$ = pressure at the bottom of air pocket [psi]

$A1$ = cross sectional area between the bridge girders (air pocket) [ft^2]

$A2$ = compressed area of air pocket [ft^2]

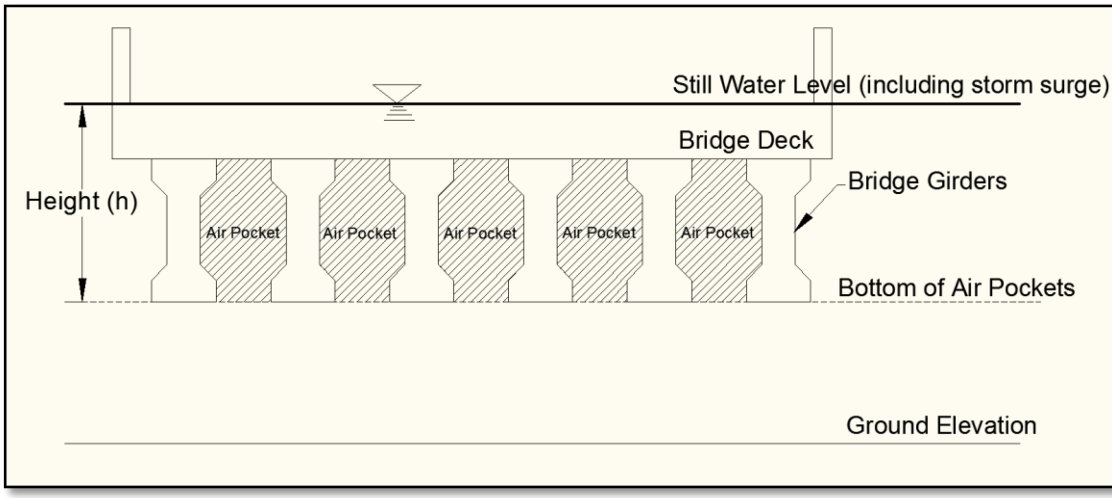


Figure 4-1: Air Pocket Diagram

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/ft^3). 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. 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. Table4.2-2 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 4.2-1: Bridge Dimensions

Bridge Dimensions:	Number of Girders	Bridge Width (ft)	Bridge Length (ft)	Girder Height (ft)	Deck Thickness (ft)	Railing Height (ft.)
Kuliouou Stream Bridge:	12	68.75	48.40	3.00	0.67	2.71
Kahaluu Stream Bridge:	8	46.00	318.00	4.50	0.50	2.13
New South Punaluu Bridge:	30	50.00	170.00	1.75	0.88	3.00
Ukoa Pond Bridge:	7	49.00	270.00	4.83	0.54	3.50
New Makaha Stream #3A Bridge:	1	46.83	70.00	2.33	0.46	1.17
Old Makaha Stream #3A Bridge:	12	32.83	78.83	1.50	0.50	0.00
Maipalaoa Bridge:	16	64.33	100.67	3.00	0.50	2.00
Moanalua Bridge:	9	64.33	215.00	1.83	0.67	3.73
Kalihi Bridge:	13	88.33	188.00	1.83	0.67	3.73
Nimitz Hwy. Slip Cover #2:	11	Variable	Variable	2.50	1.29	1.67
Nimitz Hwy. Slip Cover #3:	13	Variable	Variable	2.50	1.29	1.67

Table4.2-2: Calculated Bridge Volumes

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

*Note: Air pocket volumes are compressed values

Table 4.2-3: Buoyancy Force Results with 100% Air Volume

	Self Weight (kips)	Buoyancy Force (kips)	Residual Weight (kips)	% Retained Weight	Buoyant?
Kuliouou Bridge:	723.777	735.35134	-11.57434	-1.60%	Yes
Kahaluu Stream Bridge:	3811.5	3889.33	-77.83	-2.04%	Yes
New South Punaluu Bridge:	2336.99	1424.91	912.08	39.03%	No
Ukoa Pond Bridge:	2674.31	3306.57	-632.26	-23.64%	Yes
New Makaha Stream #3A Bridge:	1161.77	470.66	691.11	59.49%	No
Old Makaha Stream #3A Bridge:	279.83	576.19	-296.36	-105.91%	Yes
Maipalaoa Bridge:	1406.69	1319.76	86.93	6.18%	No
Moanalua Bridge:	3338.16	2775.17	562.99	16.87%	No
Kalihi Bridge:	3955.61	2484.28	1471.33	37.20%	No
Nimitz Hwy. Slip Cover #2:	2069.453	862.272	1207.181	58.33%	No
Nimitz Hwy. Slip Cover #3:	2180.023	900.288	1279.735	58.70%	No

Table 4.2-4: Recalculated Buoyancy Force Results with 50% Air Volume

Buoyancy Force Calculations: *(50% air pocket)*

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

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 report.

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.

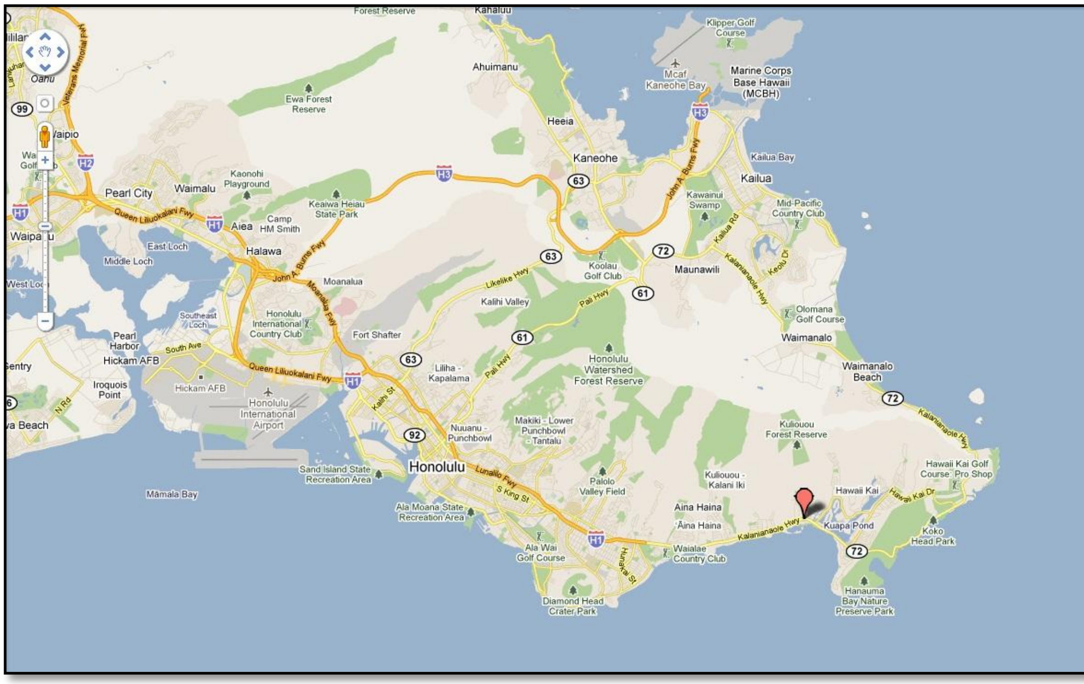


Figure 4-2: Kuliouou Stream Bridge map location

Figure 4-3: Picture of Kuliouou Stream Bridge looking north

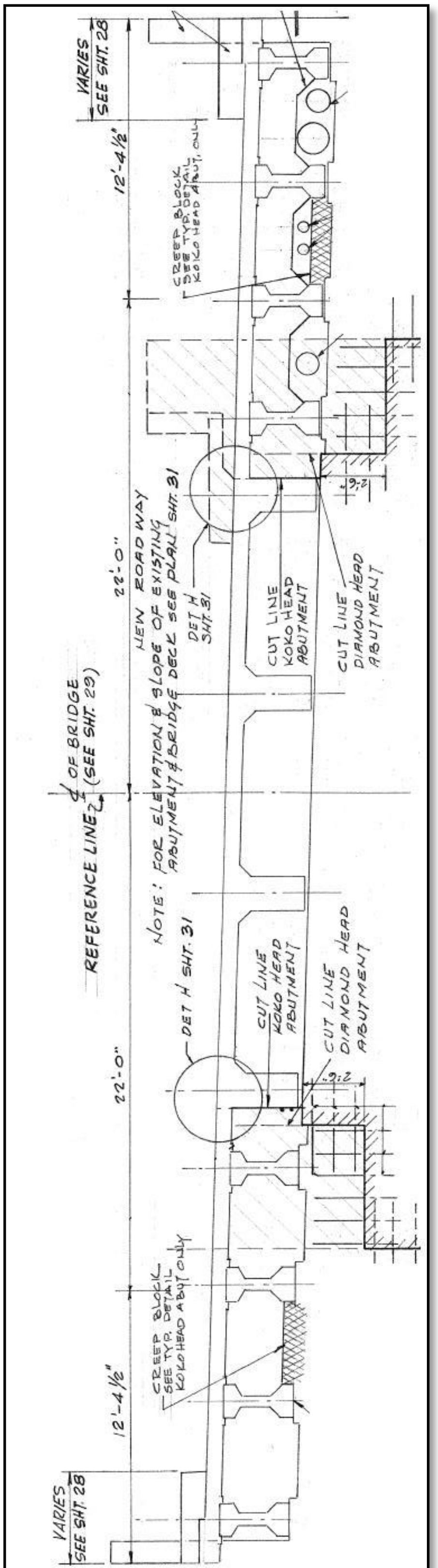


Figure 4-4: Kuliouou Bridge Profile View

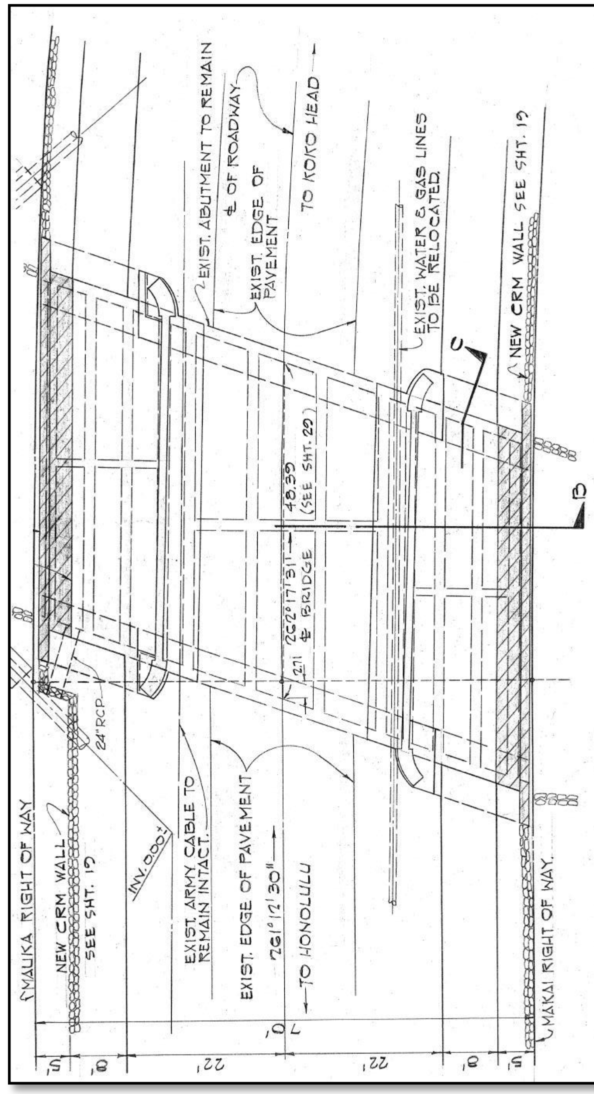


Figure 4-5: Kuliouou Bridge Plan View

4.4.2.1 Kuliouou Stream Bridge: Lateral Resistance

At the Diamond Head abutment, the girders are connected to the foundation shelves using ¾" thick galvanized steel plates (see Figure 4-6). A plate is attached directly to the bridge girder using two ¾" diameter x 6" long embedded steel studs. A similar plate is attached to the abutment with another pair of ¾" 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.

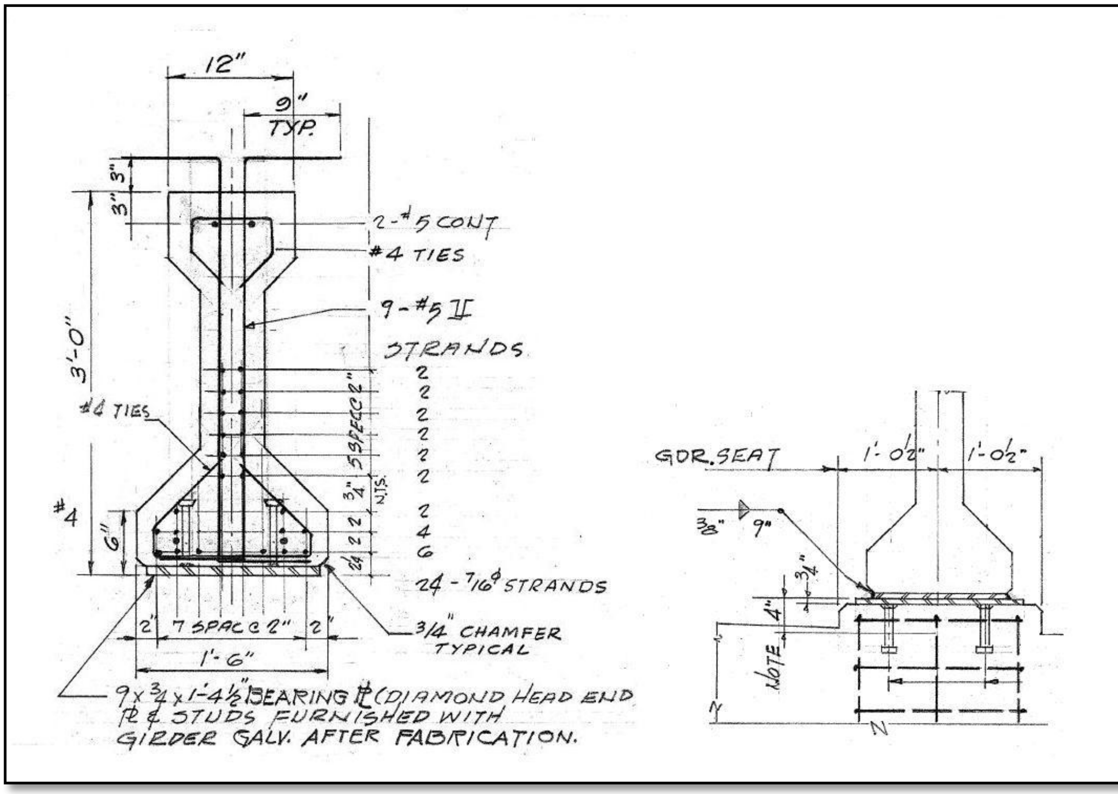


Figure 4-6: Diamond Head abutment bearing plates (profile view)

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.

Table 4.4-1: Bearing Plate Lateral Capacity

Capacity Calculation:	Capacity (kips)
Stud Shear Strength (*in abutment*)	**43.16**
Stud Shear Strength (*in girder*)	48.82
Stud Block Shear	572.34
Weld Strength	200.44
Weld Block Shear	877.50
Base Metal Shear Strength	394.00

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

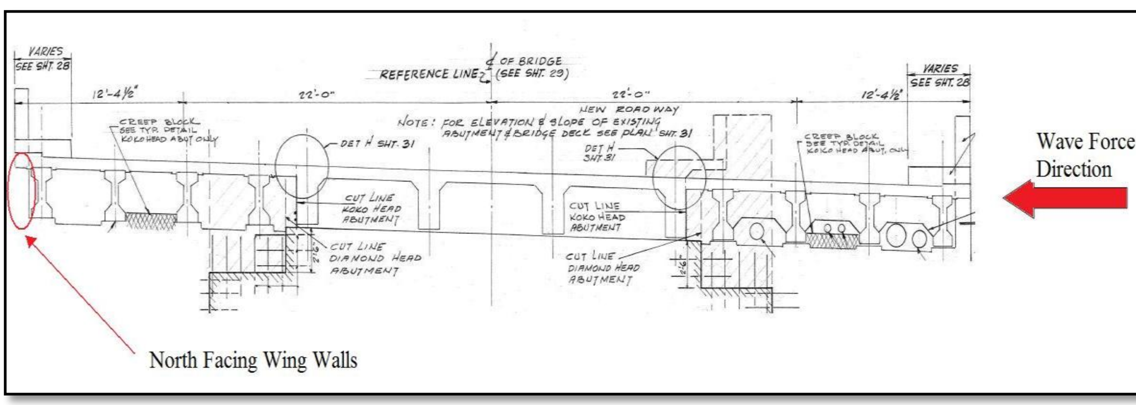


Figure 4-7: Lateral Wave Load Direction (profile view)

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.

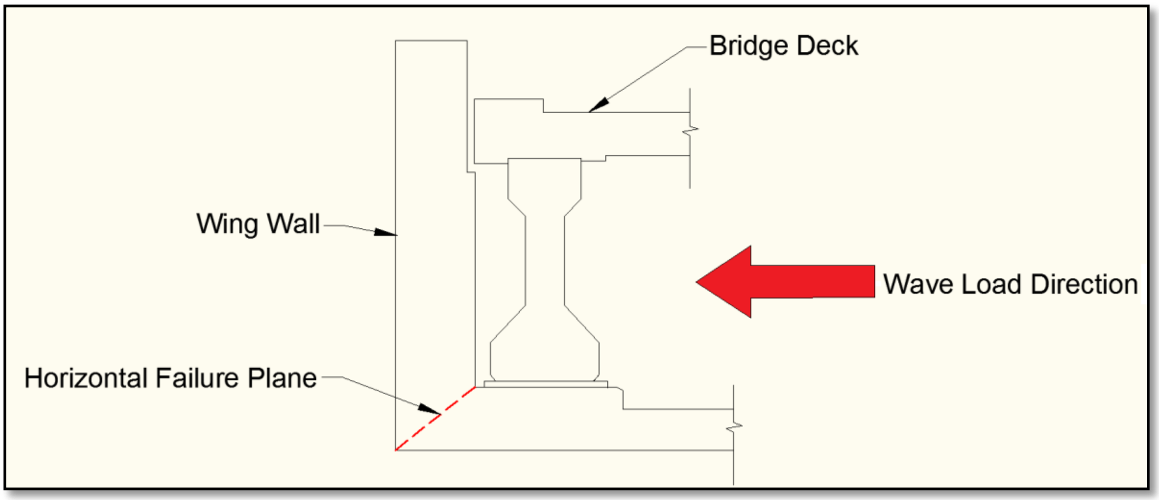


Figure 4-8: Wing Wall Horizontal Failure Plane (profile view)

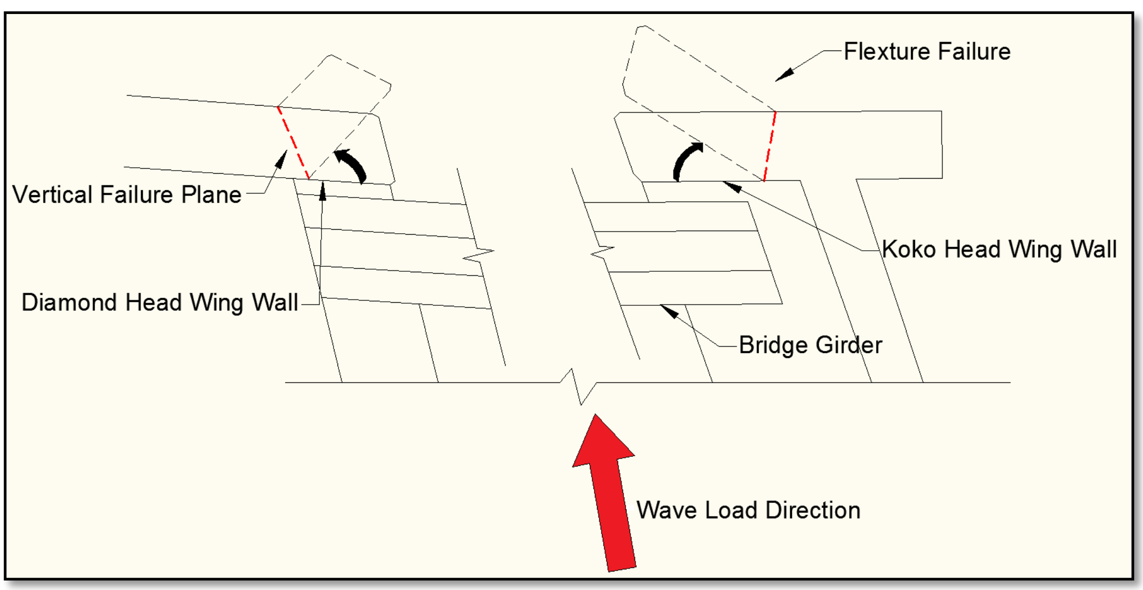


Figure 4-9: Wing Wall Vertical Failure Plane (plan view)

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

Wing Wall Capacity:	Capacity (kips)
Koko Head Horizontal Plane (Shear)	31.55
Koko Head Vertical Plane (Flexure)	27.85
Koko Head Total Capacity	**59.40**
Diamond Head Horizontal Plane (Shear)	24.20
Diamond Head Vertical Plane (Flexure)	41.76
Diamond Head Total Capacity	**65.96**

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.

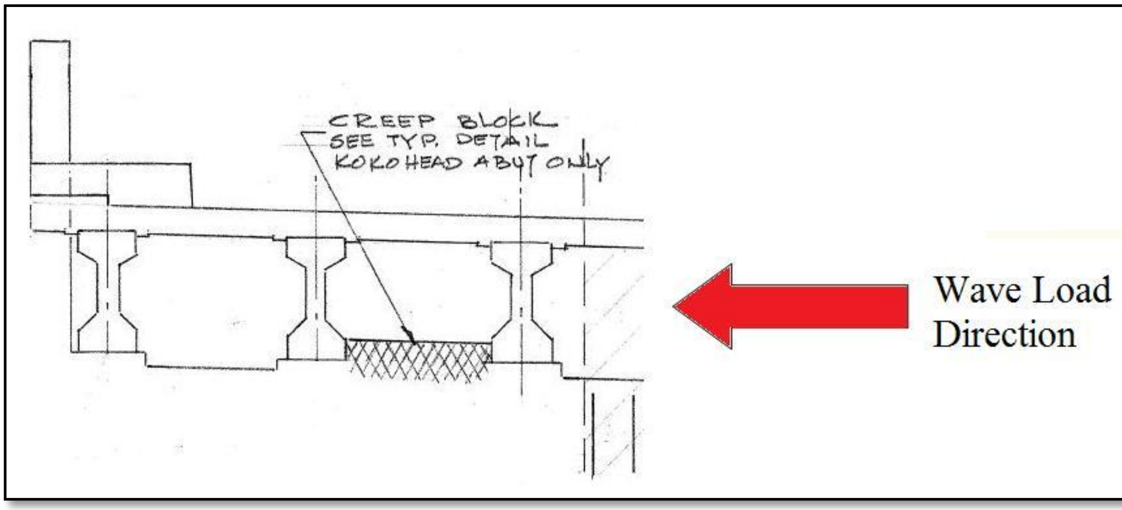


Figure 4-10: Creep Block Diagram (profile view)

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).

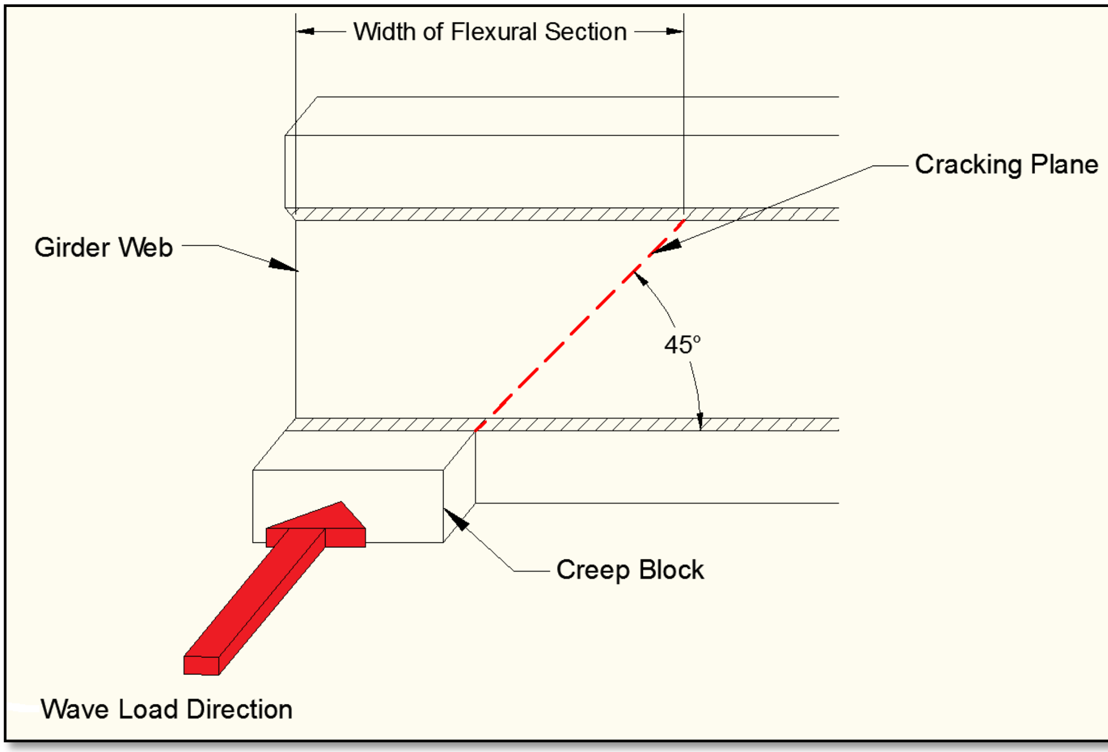


Figure 4-11: Concrete Cracking Plane

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.

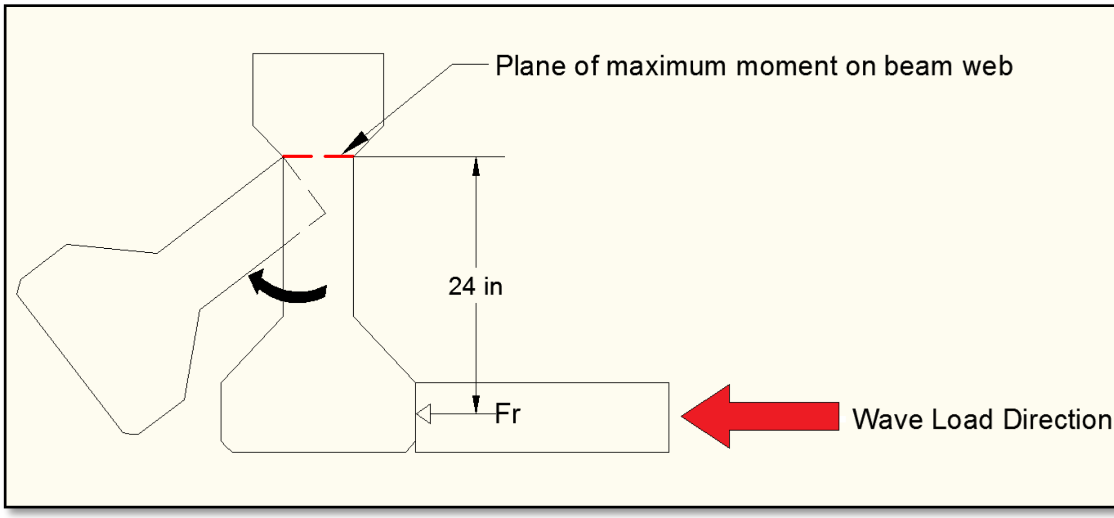


Figure 4-12: Web Flexure Failure

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

Capacity Calculation:	Capacity (kips)
Creep Block Shear Friction	134.40
Girder Web Flexure	**20.51**

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

Capacity Calculation:	Capacity (kips)
Concrete Break Out in Tension (girder)	67.62
Concrete Break Out in Tension (abutment)	**54.18**
Pull Out Strength of Stud (girder)	169.65
Pull Out Strength of Stud (abutment)	84.82
Steel Strength of Stud in Tension	57.43
Steel Plate Flexure Capacity	178.75

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).

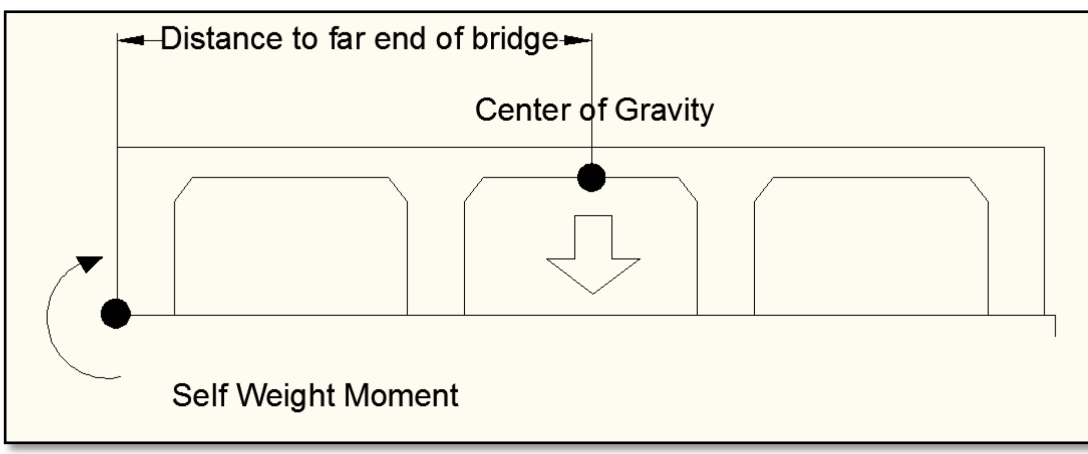


Figure 4-13: Stabilizing Moment Calculation Diagram

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

Bridge Resistance:	Capacity (kips)
Bearing Plates	345.31
Koko Head Wing Wall	59.40
Girder Web Flexure	41.01
Friction	36.19
Total Lateral Resistance	**481.91**
Total Vertical Resistance (self weight)	**723.78**
Overturning Moment Resistance	**24,879.9 kip-ft**

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.

Figure 4-14: Kahaluu Bridge Map Location

Figure 4-15: Photo of Kahaluu Bridge Looking East

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.

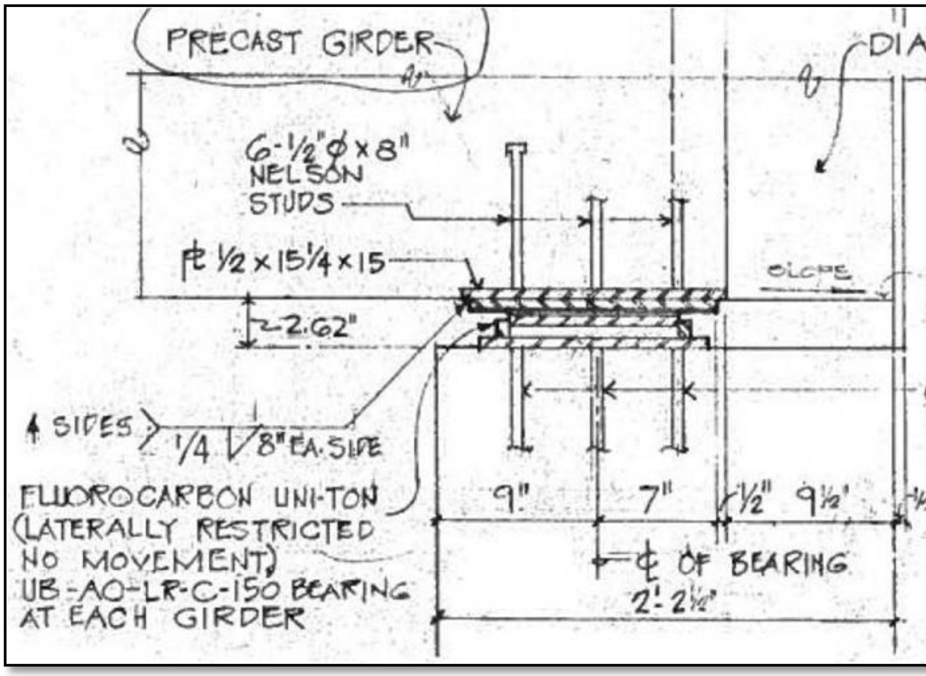


Figure 4-16: Flurocarbon Uni-Ton Bearing Pads (profile view)

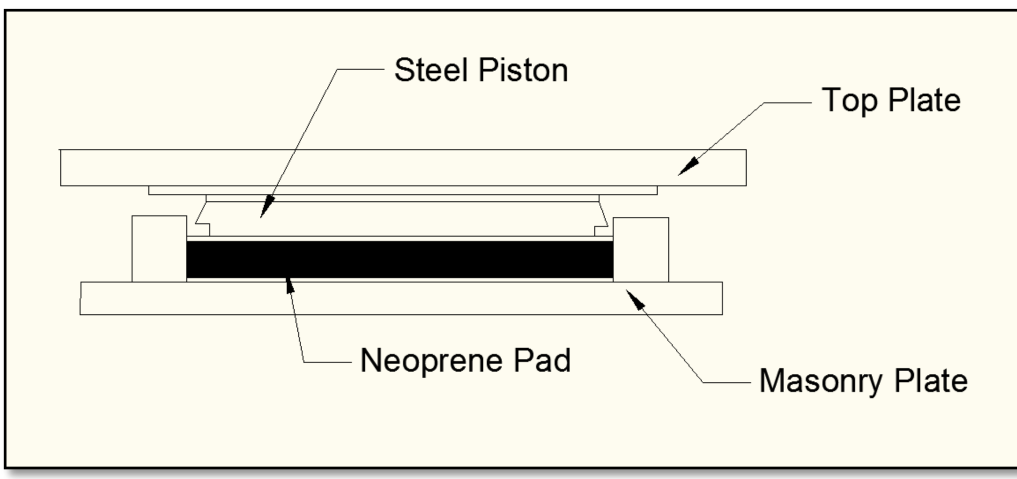


Figure 4-17: Flurocarbon Uni-Ton Bearing Pad (close up view)

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

Capacity Calculation:	Capacity (kips)
Steel Ring Capacity	91.2
Stud Shear Strength (*6 studs*)	**47.89**
Steel Plate Block Shear	1067.1
Weld Capacity	178.2
Girder Combination Capacity	62.99
Abutment Combination Capacity	**48.14**

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.

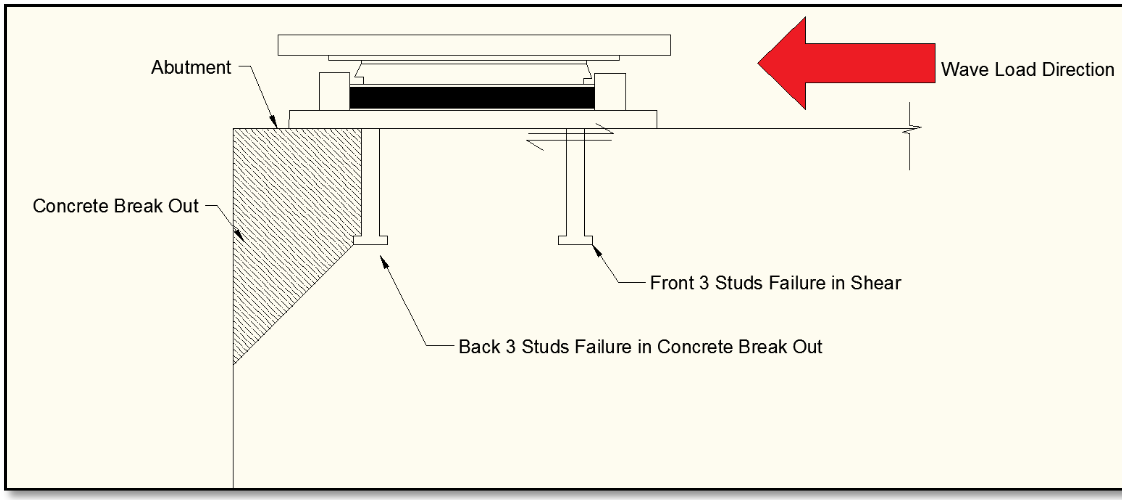


Figure 4-18: Bearing Pad Failure Mechanism

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

Wing Wall Capacity	Capacity (kips)
Horizontal Plane (Shear)	59.2
Vertical Plane (Flexure)	90.43
Total Capacity (per wing wall)	**149.63**

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

Bridge Resistance	Capacity (kips)
Wing Walls (2)	299.26
Friction	1,524.62
Total Lateral Resistance	**1,823.9**
Total Vertical Resistance (self weight)	**3,811.5**
Overturning Moment Resistance	**87,665.7 kip-ft**

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.

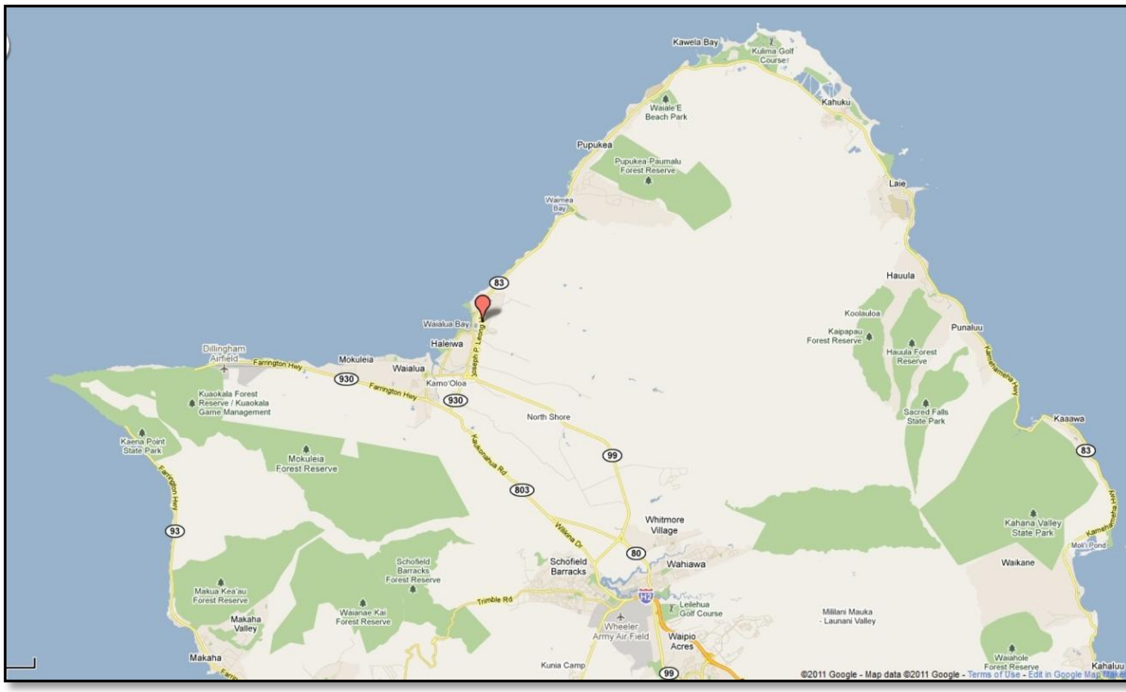


Figure 4-19: Ukoa Pond Bridge Map Location

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.

Figure 4-20: Photograph of Underside of Bridge Deck

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.

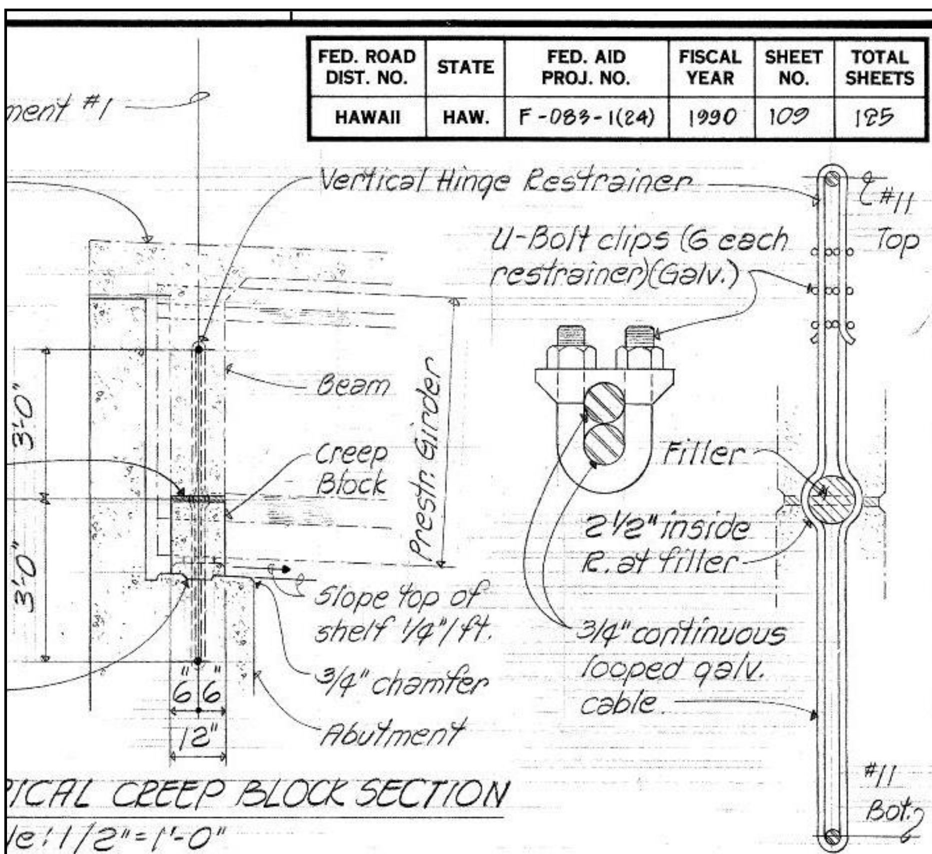


Figure 4-21: Vertical Hinge Restrainer

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).

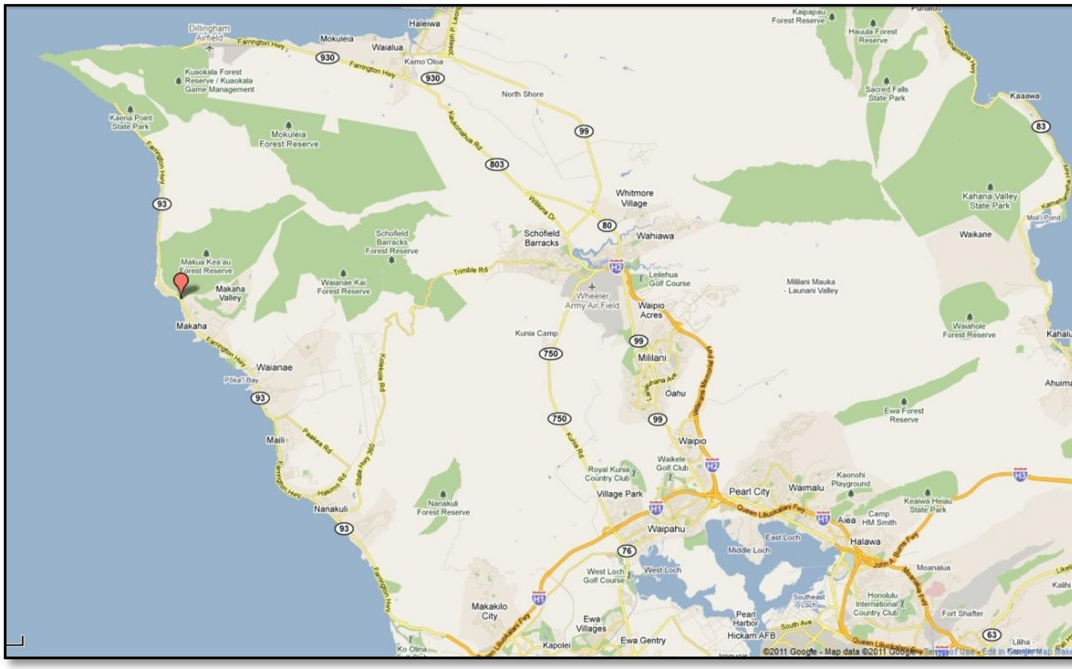


Figure 4-22: Old Makaha #3A Bridge Map Location

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.

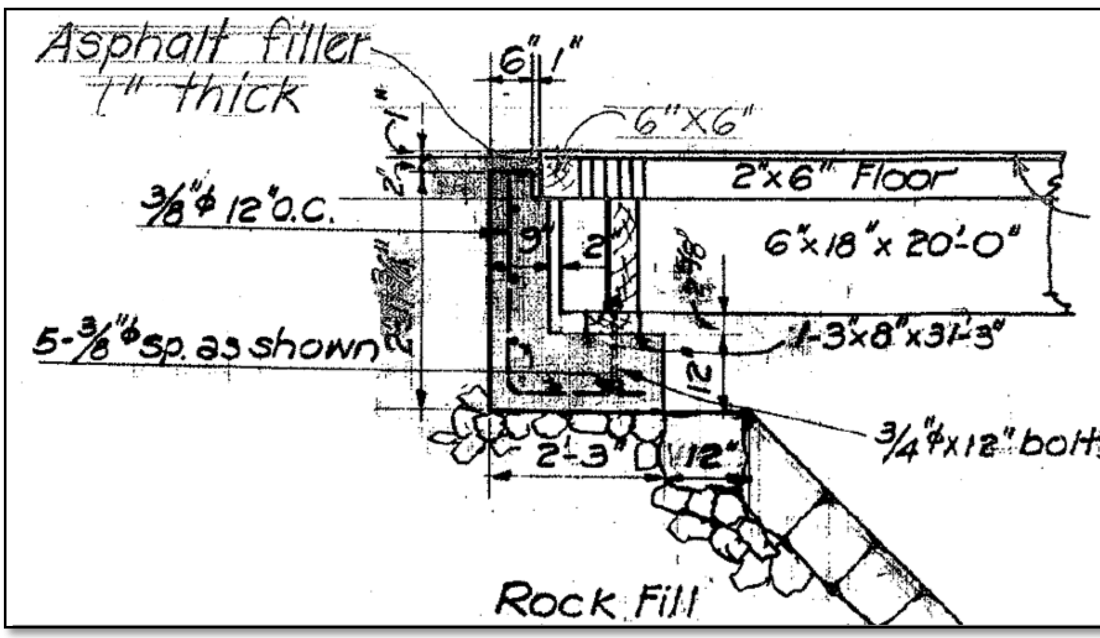


Figure 4-24: Concrete Abutment

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

Bridge Resistance:	Capacity (kips)
Bolt Shear Capacity	14.41
Friction	55.97
Total Lateral Resistance	**70.37**
Self Weight	279.83
Withdrawal Load	69.30
Total Vertical Resistance	**349.13**
Overturning Moment Resistance	**5,731.55 kip-ft**

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.

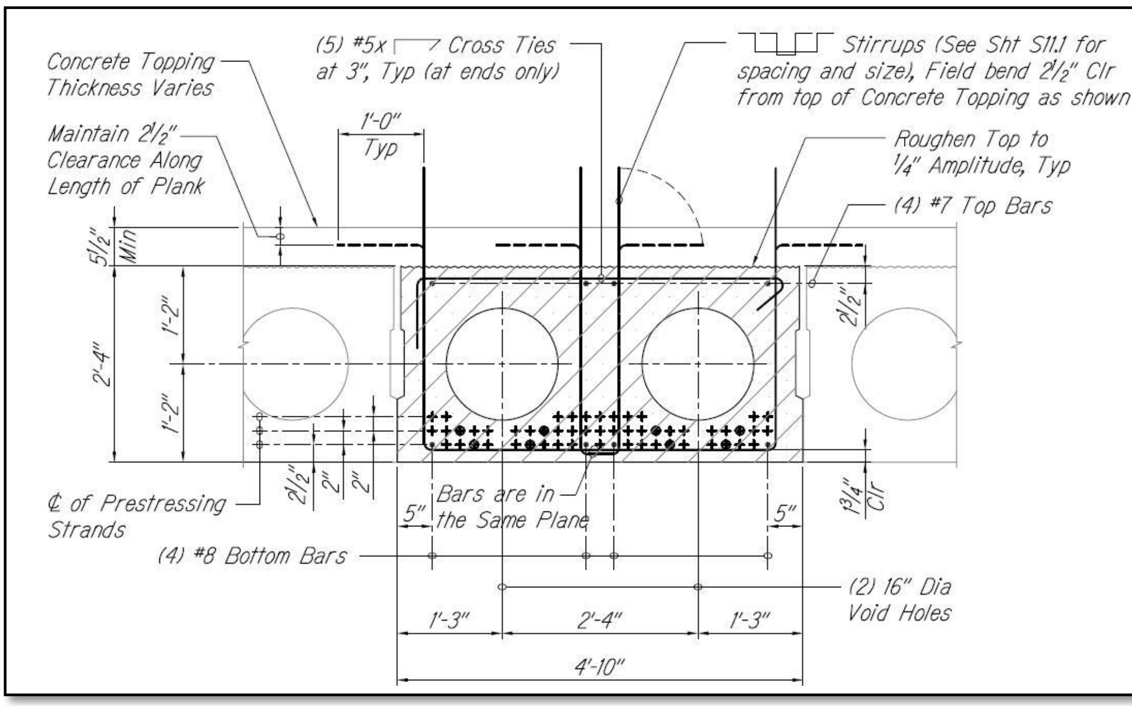


Figure 4-26: Hollow Core Plank Cross Section

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.

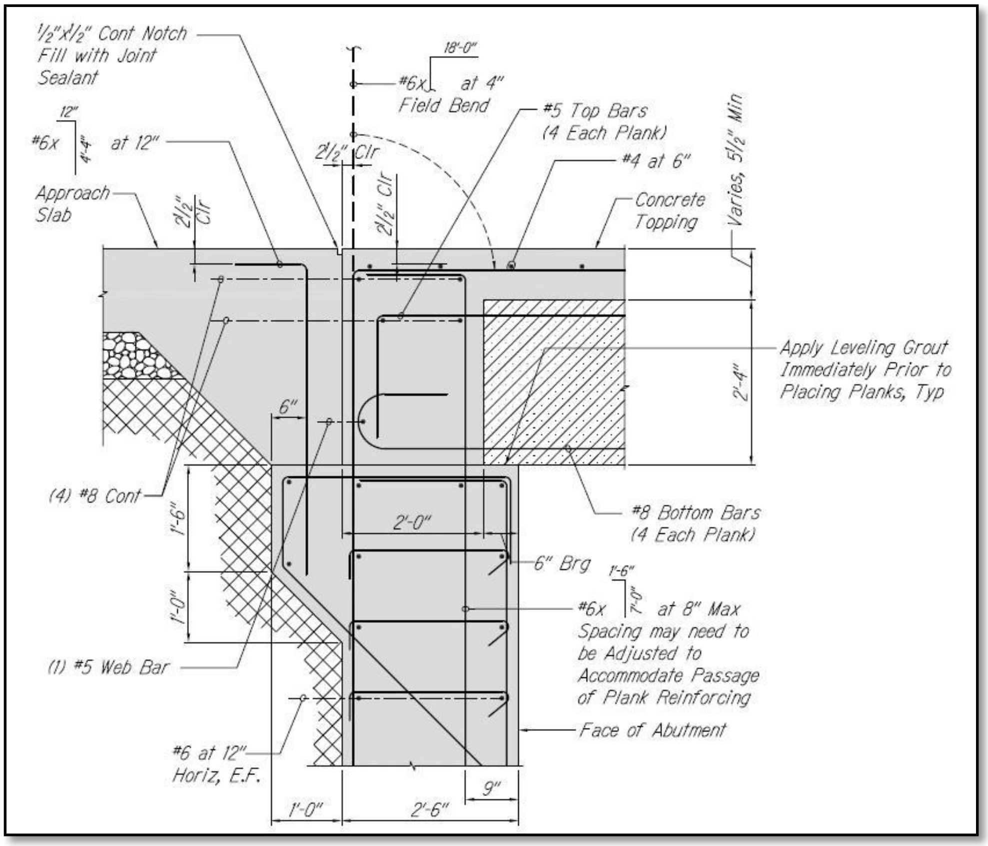


Figure 4-27: Abutment Detail

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].

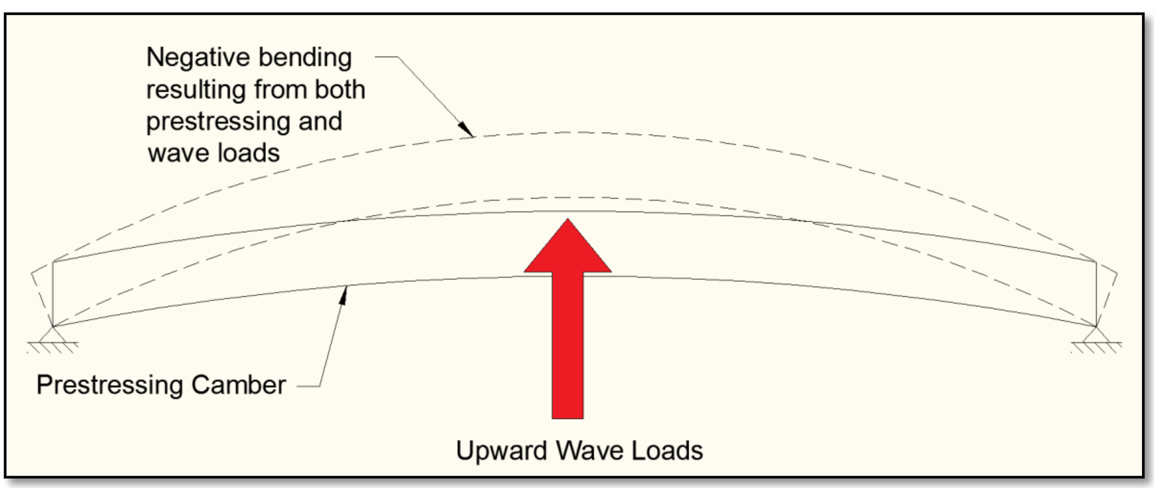


Figure 4-28: Negative Bending

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]:

$$f_{top} = -\frac{P_e}{A_c}\left(1 - \frac{e * c_t}{r^2}\right) - \frac{M_T}{S_t} \tag{4.4-1}$$

$$f_b = -\frac{P_e}{A_c}\left(1 + \frac{e * c_b}{r^2}\right) + \frac{M_T}{S_b} \tag{4.4-2}$$

In which

f_{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 [in^2]
e	=	distance from centroid of prestressing to centroid of the plank [in]
c_t	=	distance from top of plank to centroid (*rectangular section = ½ h*) [in]
c_b	=	distance from bottom of plank to centroid (*rectangular section = ½ h*) [in]
r^2	=	moment of inertia divided by the cross sectional area of concrete [in^2]
M_T	=	Total moment acting at the center of the plank span [lb-in]
S_t	=	moment of inertia divided by c_t [in^3]
S_b	=	moment of inertia divided by c_b [in^3]

The maximum permissible tensile stress is taken as:

$$f_t = 12 * \sqrt{f_c'}$$

The maximum permissible compressive stress is taken as:

$$f_c = -\,0.85 * f_c'$$

In which

f_c' = 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).

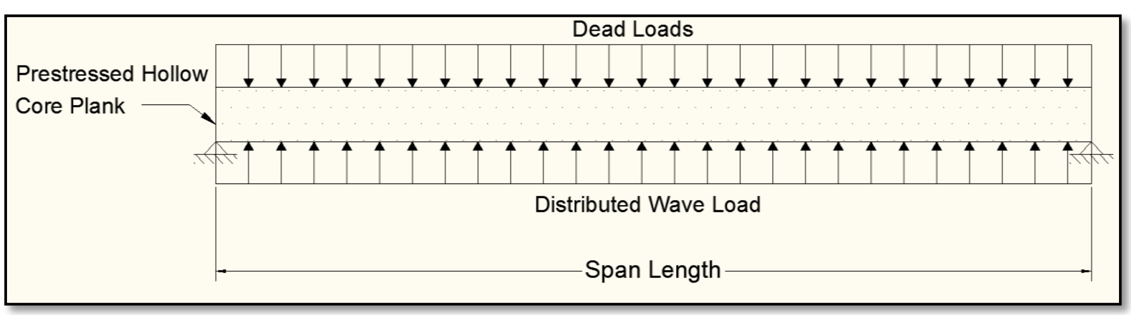


Figure 4-29: Distributed Loads Acting On Bridge Deck

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:

$$M_T = M_D - \frac{W_w * L^2}{8} \quad (4.4\text{-}3)$$

in which

M_D = positive moment due to dead loads [lb-in]

w_D = distributed dead load [lbs/in]

L = length of bridge [in]

W_w = 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:

$$W_{Ft} = \frac{8}{L^2} * [M_D + (S_t * f_t) - \left(\frac{S_t * P_e}{A_c}\right) * (\frac{e * c_t}{r^2} - 1)] \quad \text{(4.4-4: Tensile Limit)}$$

$$W_{Fc} = \frac{8}{L^2} * [M_D - (S_b * f_c) - \left(\frac{S_b * P_e}{A_c}\right) * (1 + \frac{e * c_b}{r^2})] \quad \text{(4.4-5: Compression Limit)}$$

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

Bridge Resistance	Capacity (kips)
Total Lateral Resistance (shear friction)	**8,870.40**
Self Weight	1,161.80
Tensile Strength of Rebar	16,632
Total Vertical Resistance	**17,793.80**
Overturning Moment Resistance	**416,671.5 kip-ft**

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

Deck Capacity	Capacity (kips)
Tensile Capacity	**1,127.13**
Compressive Capacity	3,625.89
Tensile Capacity (submerged)	**653.572**
Compressive Capacity (submerged)	2,898.44
Tensile % Loss once submerged	42.01%
Compressive % Loss once submerged	20.06%

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.

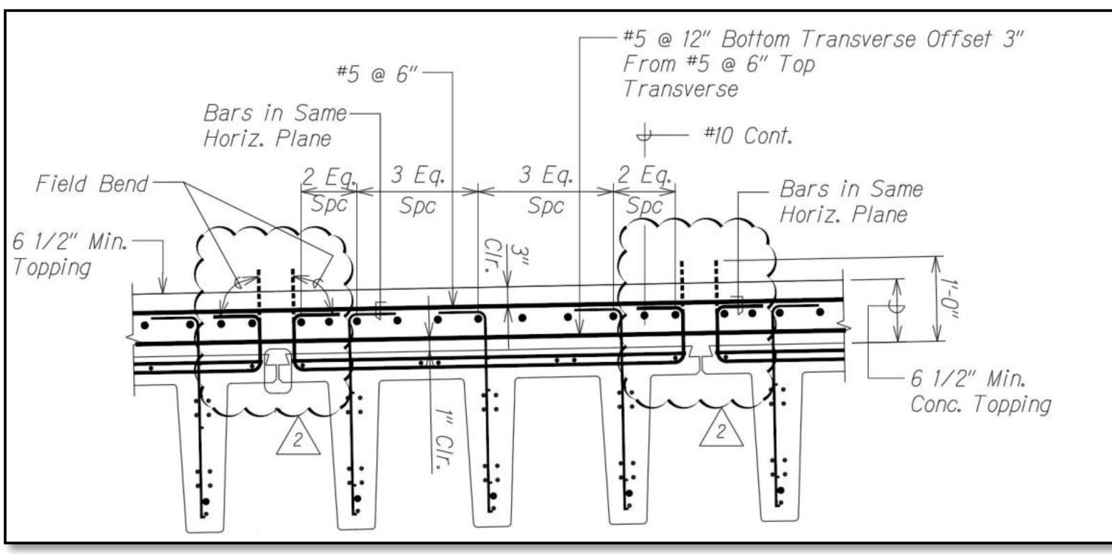


Figure 4-30: Typical Triple Tee (Trideck)

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)

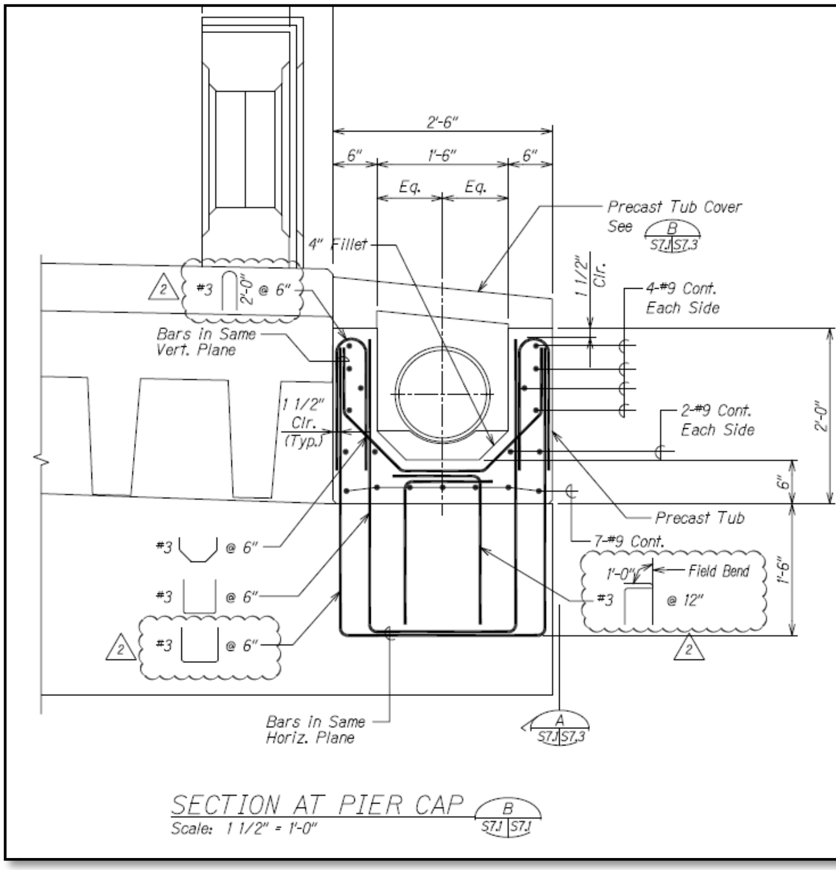


Figure 4-31: Precast Tub

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.

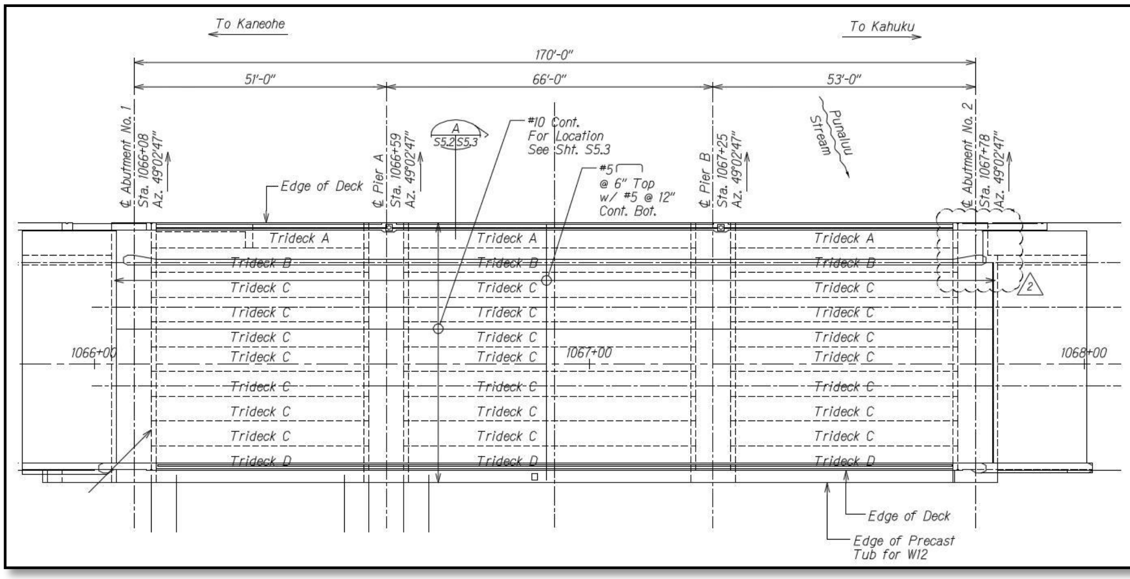


Figure 4-32: New South Punaluu Bridge Layout

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 Punaluu Bridge Negative Bending Capacity (Center Span)

Deck Capacity	Capacity (kips)
Tensile Capacity	**1,262.99**
Compressive Capacity	3,777.11
Tensile Capacity (submerged)	**812.81**
Compressive Capacity (submerged)	3,326.98
Tensile % Loss once submerged	35.64%
Compressive % Loss once submerged	11.92%

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 prestresssed 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).

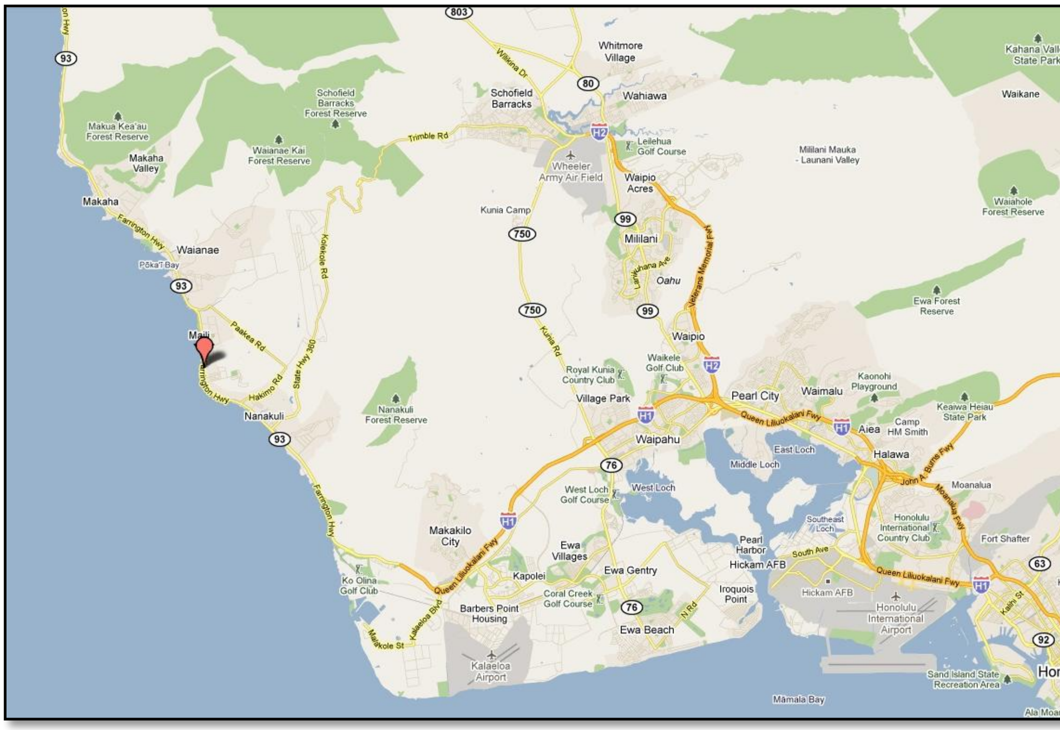


Figure 4-33: Maipalaoa Bridge Map Location

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).

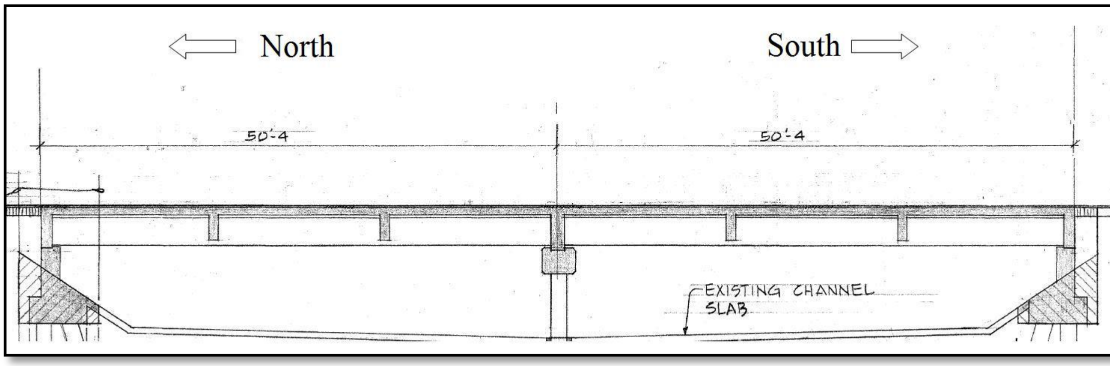


Figure 4-34: Maipalaoa Bridge Deck Span

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

Figure 4-35: Photograph of Maipalaoa Bridge Looking South East

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.

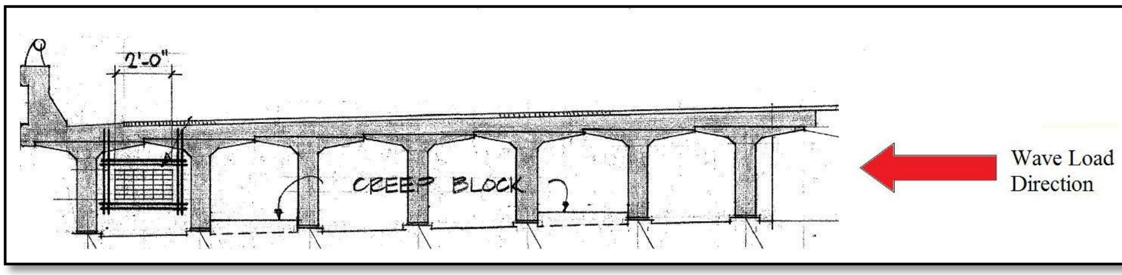


Figure 4-36: Maipalaoa Bridge Creep Blocks

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.

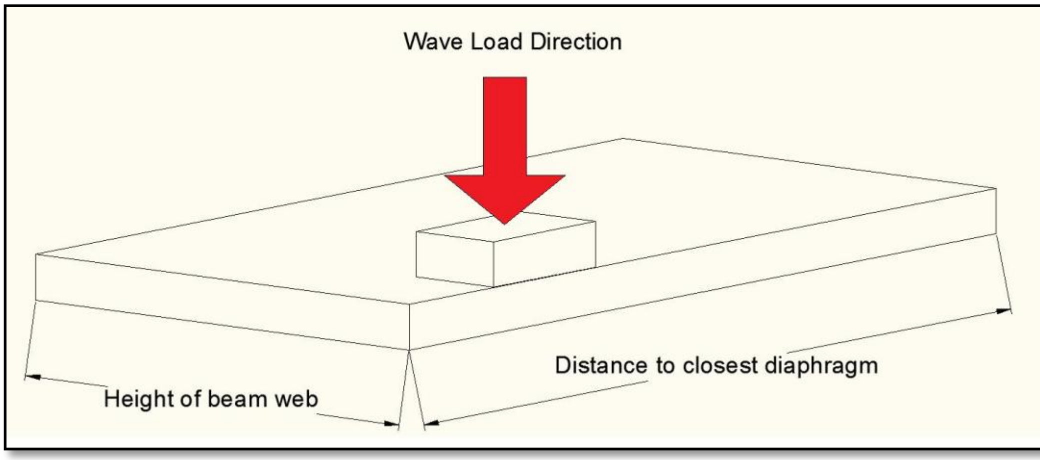


Figure 4-37: Beam Web Horizontal Orientation

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.

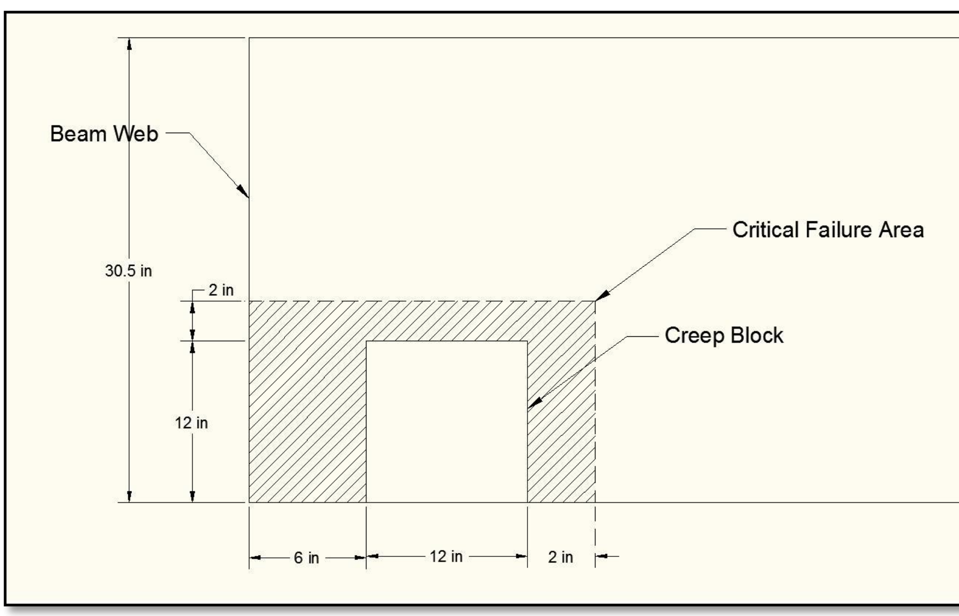


Figure 4-38: Beam Web Critical Failure Area

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.

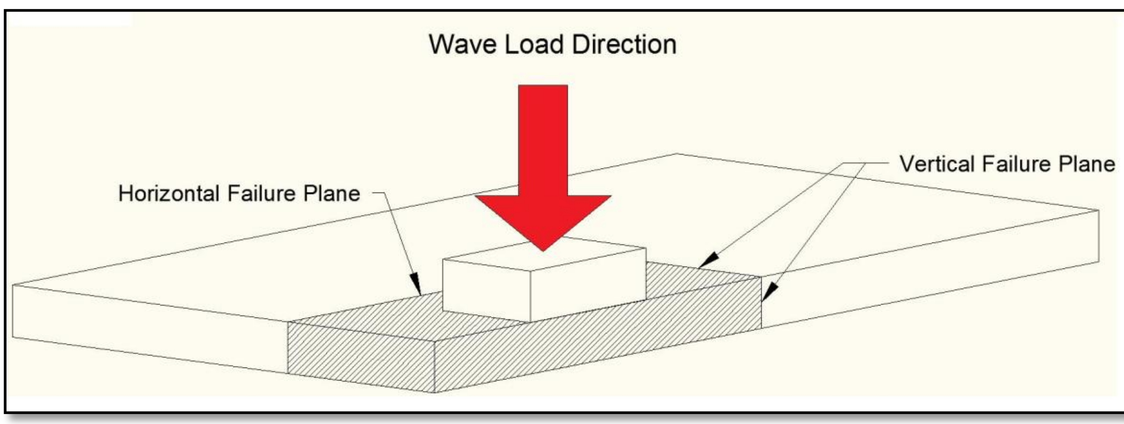


Figure 4-39: Failure Planes

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.

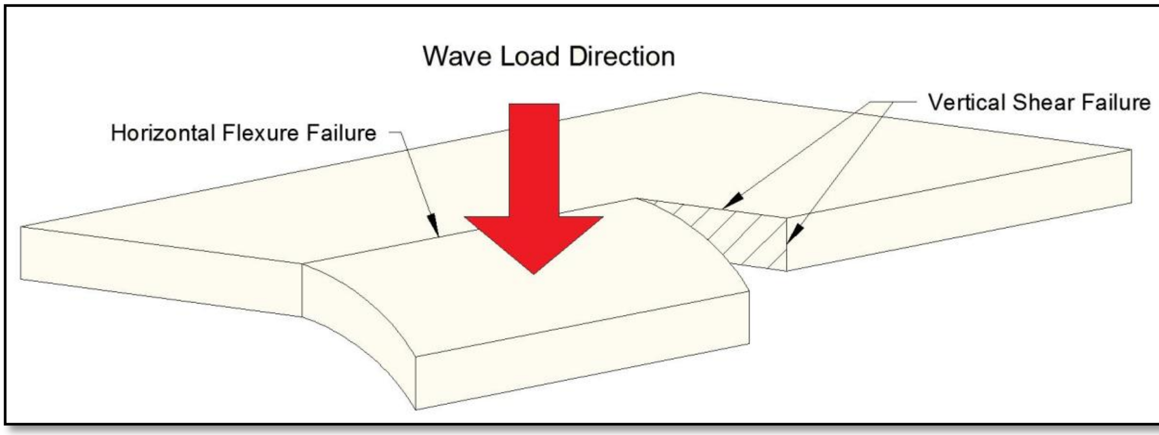


Figure 4-40: Horizontal Plane Flexure Failure

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

Capacity Calculation	Capacity (kips)
Creep Block Shear Friction	243.04
Beam Web Prestressed Slab Punch Out	137.3
Beam Web Non-reinforced Slab Punch Out	66.93
Beam Web Independent Failure Plane (shear-shear)	50.48
Beam Web Independent Failure Plane (shear-flexure)	86.35
Beam Web Flexure	**15.34**

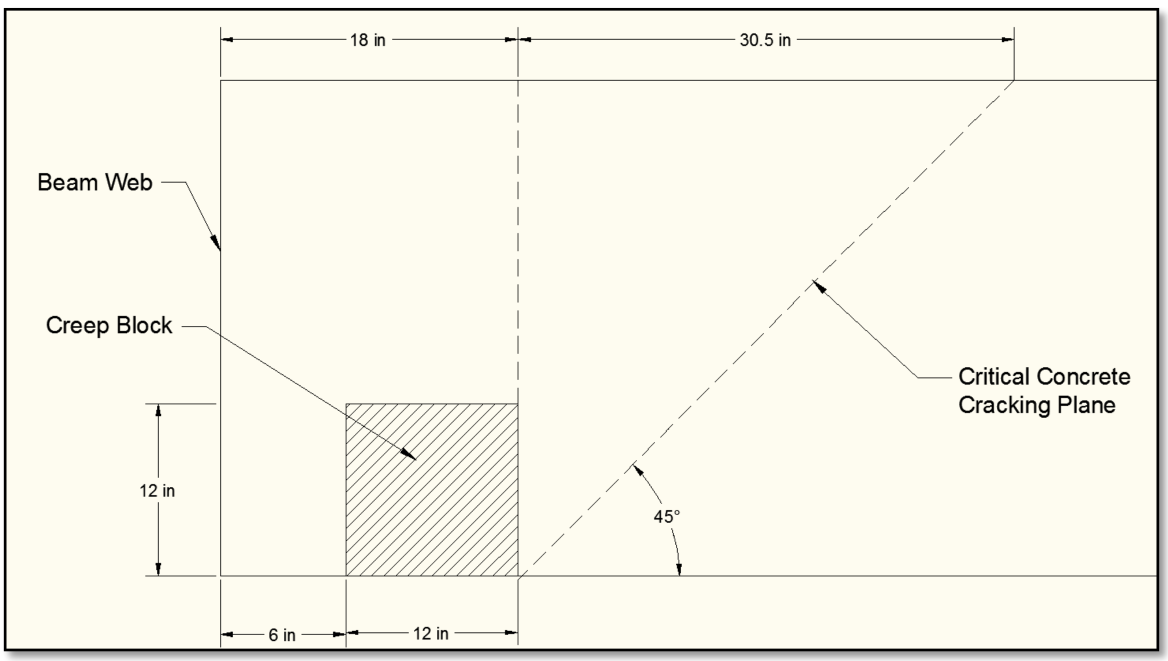


Figure 4-41: Beam Web Cracking Plane

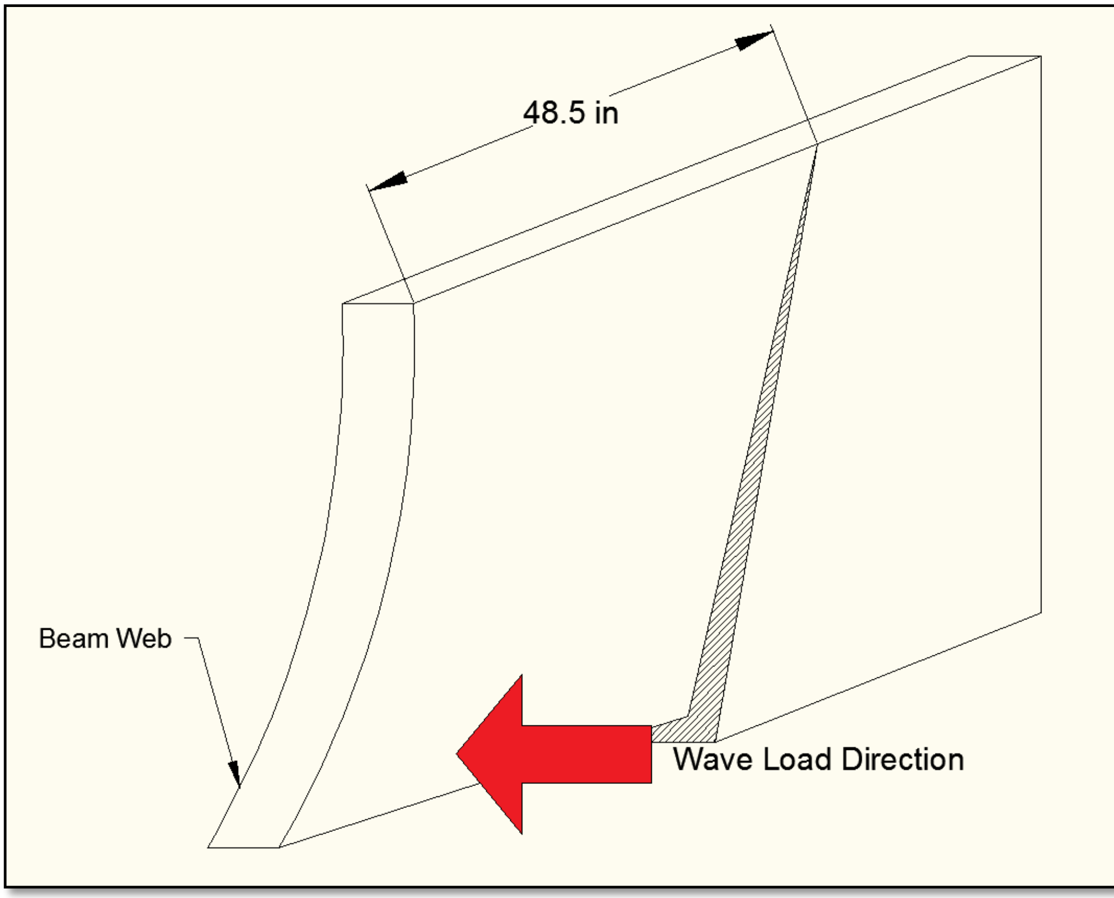


Figure 4-42: Beam Web Flexure

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

Bridge Resistance	Capacity (kips)
Total Vertical Resistance (self weight)	**1,406.70**
Beam Web Flexure Capacity	122.68
Gravity Induced Friction	140.67
Total Lateral Resistance	**263.35**
Overturning Moment Resistance	**45,248.85 kip - ft**

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.

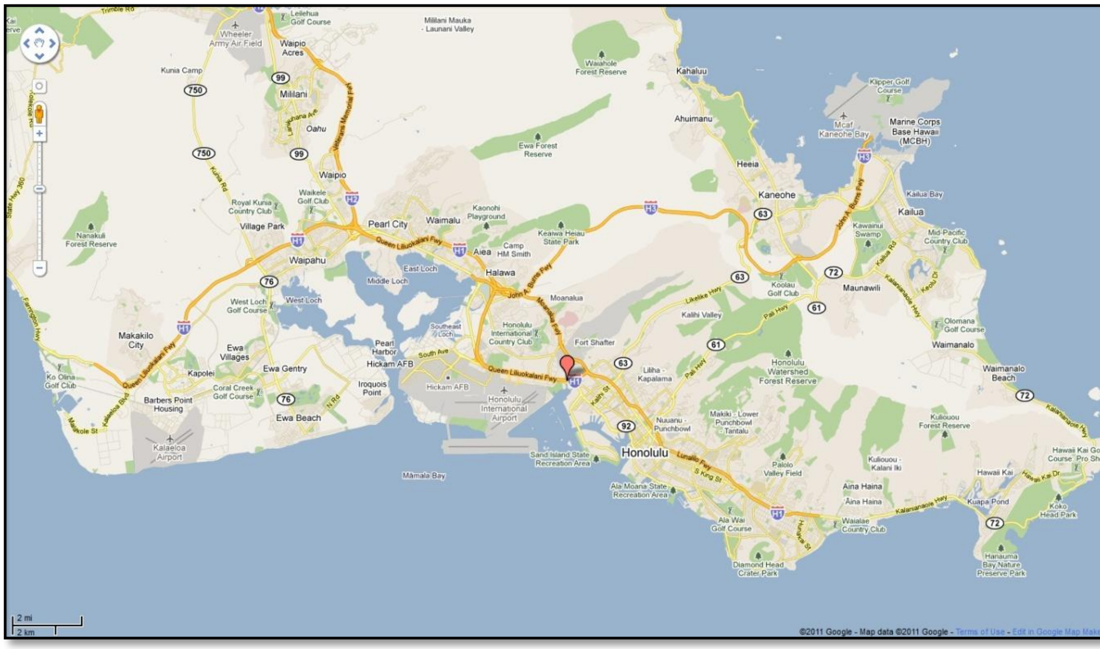


Figure 4-43: Map Location of Moanalua Bridge

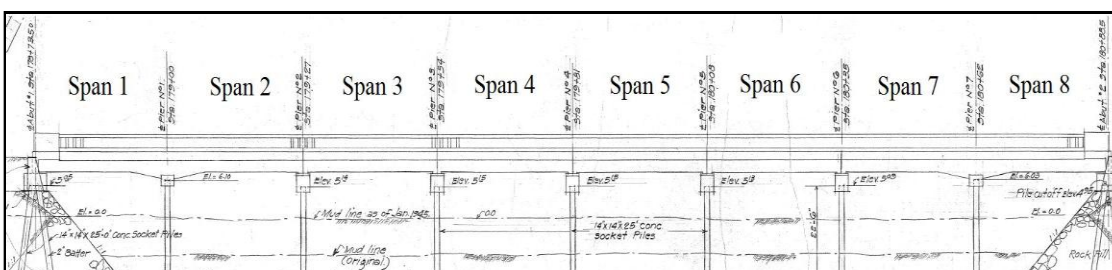


Figure 4-44: Bridge Spans

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

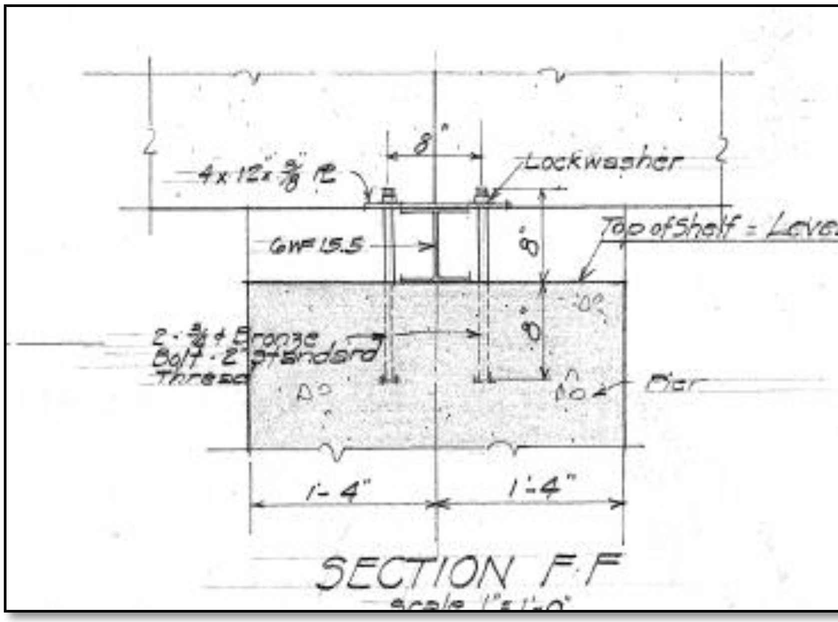


Figure 4-46: Water Main Support Connection Detail

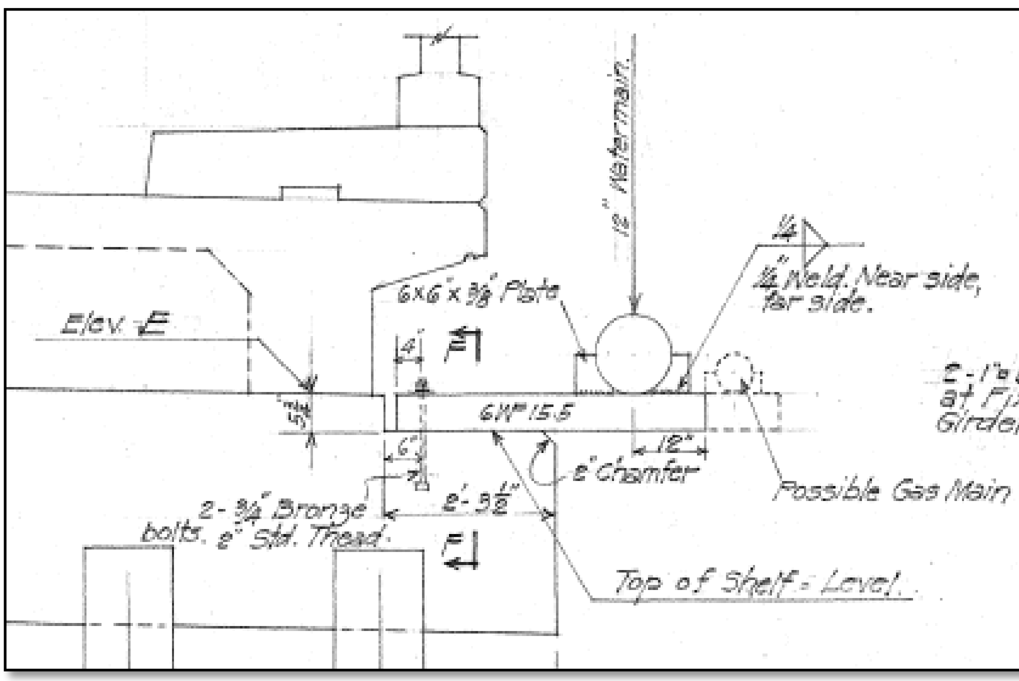


Figure 4-47: Water Main Support Profile View

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.

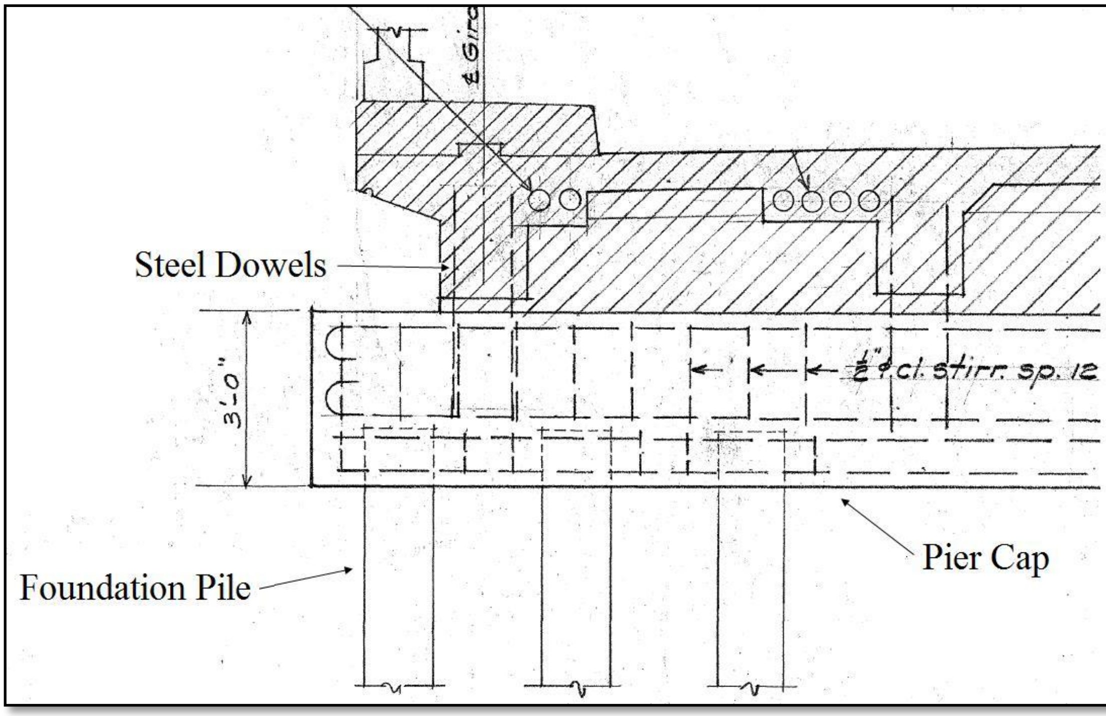


Figure 4-48: Bridge Pier Cap Diagram

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 334kips 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 Brides. 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

Spans 2 & 7 Resistance	Capacity (kips)
Total Vertical Resistance (self weight)	**417.27**
Total Lateral Resistance (friction)	**333.82**
Overturning Moment Resistance	**13,422 kip - ft**
Spans 3, 4, 5, & 6 Resistance	Capacity (kips)
Total Vertical Resistance (total self weight)	**575.67**
Total Lateral Resistance	**1926.46**
Overturning Moment Resistance	**18,517 kip - ft**

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.

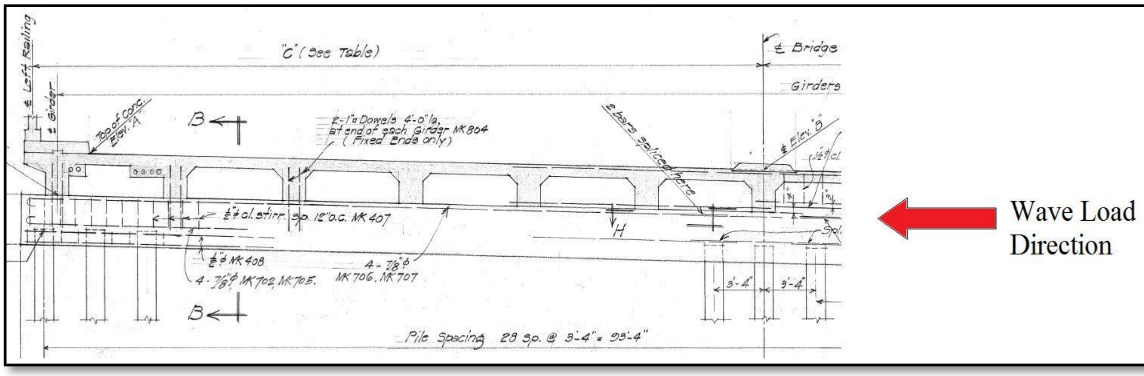


Figure 4-50: Kalihi Bridge Dowels

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

Typical Span Resistance	Capacity (kips)
Total Vertical Resistance (self weight)	**565.09**
Total Lateral Resistance (friction)	**452.07**
Overturning Moment Resistance	**24,958.14 kip-ft**

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.

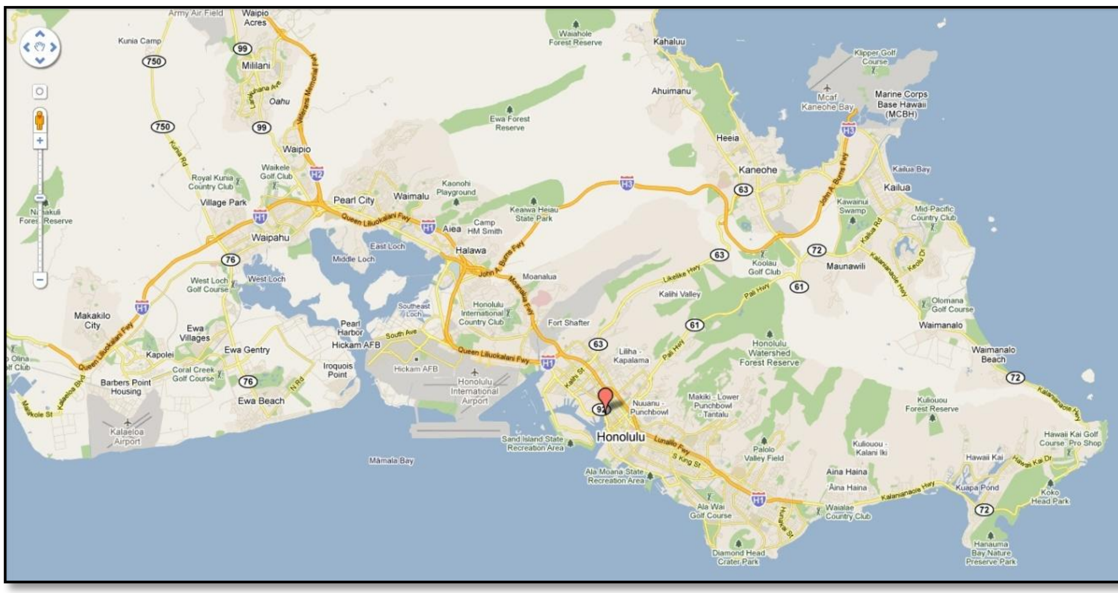


Figure 4-51: Nimitz Highway Slip Covers #2 & #3 Map Location

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.

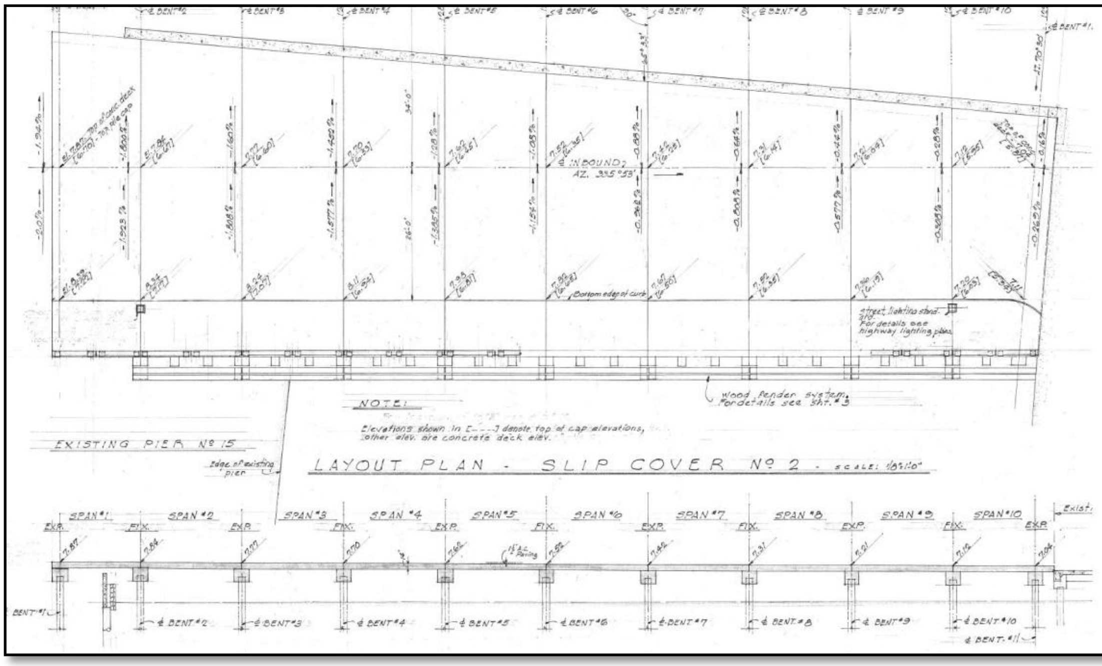


Figure 4-52: Nimitz Highway Slip Cover #2

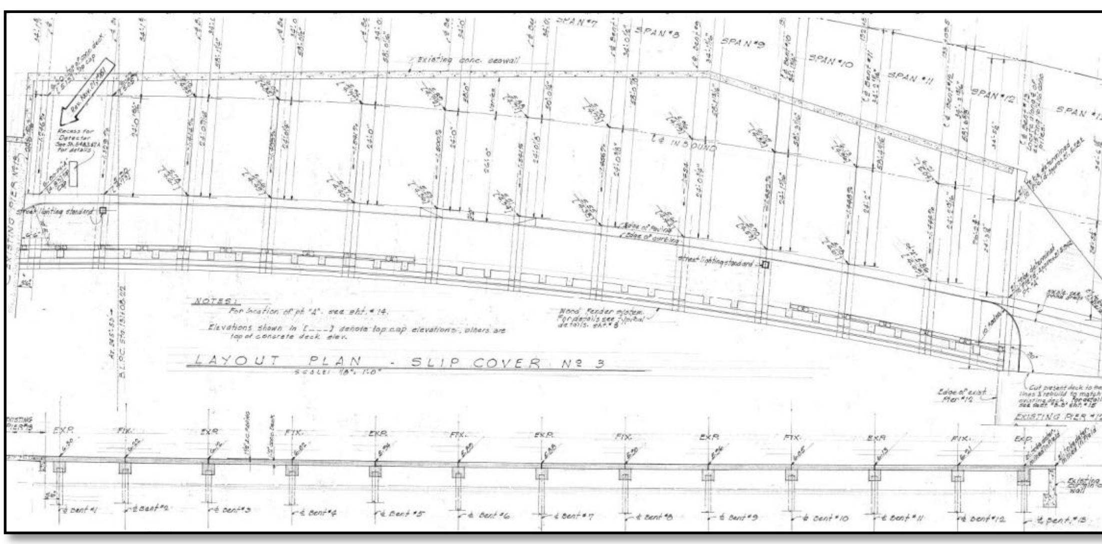


Figure 4-53: Nimitz Highway Slip Cover #3

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.

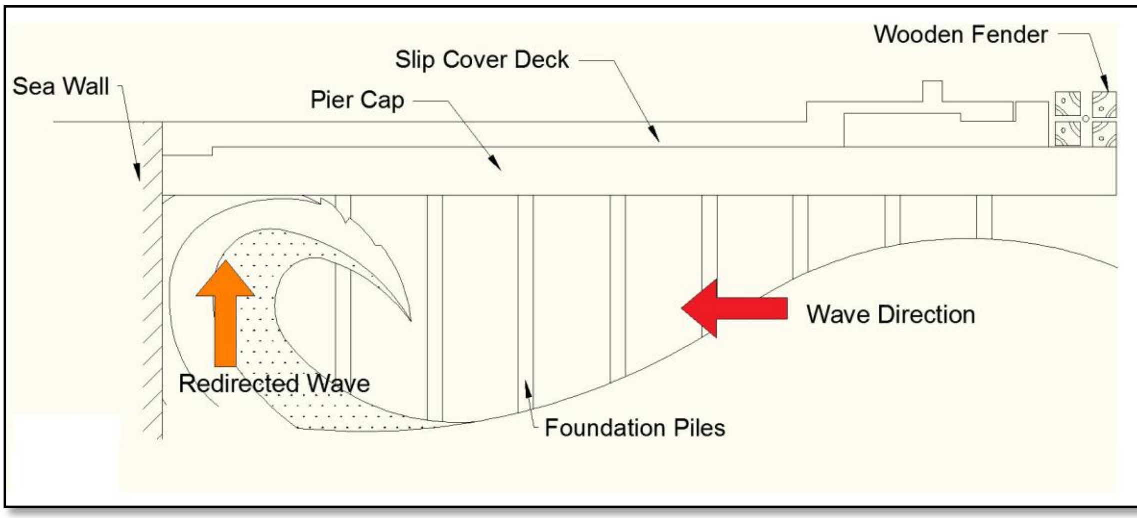


Figure 4-54: Wave Direction

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].

Figure 4-55: Photograph of failed access panels of port in Japan [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.

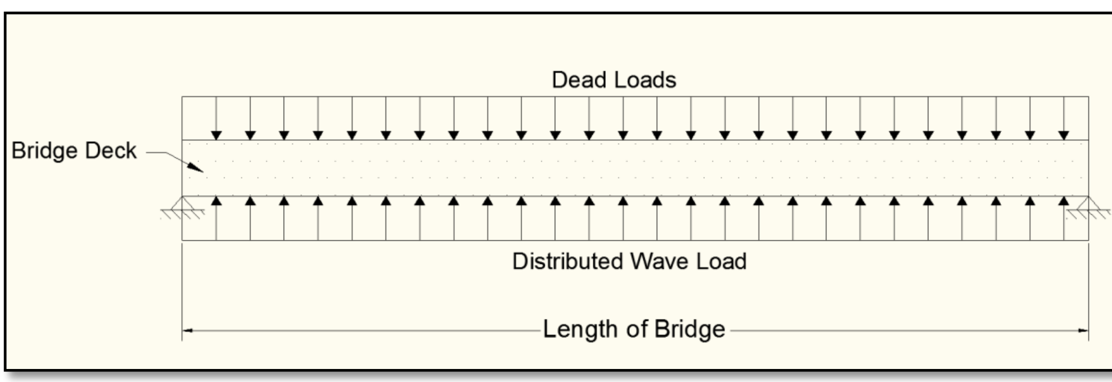


Figure 4-56: Loads Acting on Bridge Decks

Table 4.5-1: Bridge Deck Dimensional Properties

Bridge Deck Properties:	Deck Thickness (in.)	Span Width (in.)	Span Length (in.)	Top Concrete Cover (in.)	Bottom Concrete Cover (in.)
Kuliouou Bridge: (widening)	8.00	51.00	580.75	1.50	1.25
Kuliouou Bridge: (existing)	8.00	88.00	580.75	1.50	1.50
Kahaluu Stream Bridge:	6.00	49.00	3816.00	1.50	1.00
New South Punaluu Bridge:	6.50	59.63	792.00	3.00	1.00
Ukoa Pond Bridge:	6.50	60.00	1080.00	1.50	1.25
New Makaha Stream #3A Bridge:	5.50	58.00	840.00	2.50	1.75
Old Makaha Stream #3A Bridge: *(Wood Deck)*	6.00	394.50	946.00	NA	NA
Maipalaoa Bridge: (widening)	6.00	314.00	1208.00	1.50	0.75
Maipalaoa Bridge: (existing)	5.50	458.00	1208.00	1.50	1.00
Moanalua Bridge:	8.00	72.00	324.00	2.00	1.50
Kalihi Bridge:	8.00	66.00	324.00	2.00	1.50
Nimitz Hwy. Slip Cover #2:	14.00	206.00	720.00	2.00	2.00
Nimitz Hwy. Slip Cover #3:	14.00	206.00	410.40	2.00	2.00

Table 4.5-2: Bridge Deck Reinforcing Properties

	Concrete fc' (psi)	Reinforcing Steel fy (psi)	Top Reinforcement	Bottom Reinforcement	Stirrups
Kuliouou Bridge: (widening)	3000	40000	#4 @ 18" o.c.	#4 @ 8" o.c.	none
Kuliouou Bridge: (existing)	3000	40000	7 - #3	11 - #4	#4 @ 10 " o.c.
Kahaluu Stream Bridge:	4000	60000	3 - #6	3 - #6	none
New South Punaluu Bridge:	9000	270000	10 - #5	prestressing	#4 @ 12" o.c.
Ukoa Pond Bridge:	3750	60000	6 - #5, 2 - #10	8 - #5	none
New Makaha Stream #3A Bridge:	8000	270000	4 - #7	4 - #8, 53 - prestress	#5 at edge @ 3" o.c. #4 center @ 6", 12" o.c.
Old Makaha Stream #3A Bridge: *(Wood Deck)*	NA	NA	NA	NA	NA
Maipalaoa Bridge: (widening)	3000	40000	14 - #7, 12 - #4	42 - #4	none
Maipalaoa Bridge: (existing)	3000	40000	#4 @ 15" o.c.	#5 @ 12" o.c.	none
Moanalua Bridge:	3000	40000	3 - #4	5 - #5	#5 @ 12" o.c.
Kalihi Bridge:	3000	40000	3 - #4	5 - #5	#5 @ 12" o.c.
Nimitz Hwy. Slip Cover #2:	3500	40000	14 - 1/2" dia.	15 - 5/8" dia.	none
Nimitz Hwy. Slip Cover #3:	3500	40000	15 - 1/2" dia.	15 - 5/8" dia.	none

Table 4.5-3: Bridge Deck Capacities

	Positive Moment Capacity Mn (k-ft)	Negative Moment Capacity Mn (k-ft)	Positive load capacity (kips)	Negative load capacity (kips)	Bridge Vertical Resistance (kips)	Critical Capacity
Kuliouou Bridge: (widening)	25.26	12.32	67.55	756.71	342.68	displacement
Kuliouou Bridge: (existing)	44.40	15.87	68.80	740.23	471.88	displacement
Kahaluu Stream Bridge:	28.96	25.65	8.21	3819.40	3811.55	displacement
New South Punaluu Bridge: *(Prestressed)*	NA	NA	NA	NA	NA	NA
Ukoa Pond Bridge:	59.50	93.45	51.83	2752.11	3042.3	deck failure
New Makaha Stream #3A Bridge: *(Prestressed)*	NA	NA	NA	NA	NA	NA
Old Makaha Stream #3A Bridge: *(Wood deck)*	118.56	118.56	12.02	303.37	349.13	deck failure
Maipalaoa Bridge: (widening)	134.13	145.09	26.21	600.50	572.15	displacement
Maipalaoa Bridge: (existing)	156.51	72.95	30.58	848.79	834.54	displacement
Moanalua Bridge:	31.10	11.37	98.79	453.39	417.27	displacement
Kalihi Bridge:	31.02	11.36	147.60	619.13	565.09	displacement
Nimitz Hwy. Slip Cover #2:	178.80	108.80	23.84	221.01	206.5	displacement
Nimitz Hwy. Slip Cover #3:	178.80	116.52	41.82	245.26	218	displacement

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 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 4.5-4: Girder Properties

	Concrete fc' (psi)	Girder Length (ft)	Number of Girders	Prestressing Strands	Pe (lbs)	Total Dead Load (unsbumerged) (lb/in)	Total Dead Load (submerged) (lb/in)
Kuliouou Bridge: (widening)	3000	48.39	8	24 - 7/16" dia	317520.00	75.78	11.98
Kahaluu Stream Bridge:	4000	106.00	24	40 - 1/2" dia	1156680.00	113.54	18.06
Ukoa Pond Bridge: (span 1) Keehi Type IV Girder	4500	90.00	7	1/2" dia	793400	121.10	22.55
Ukoa Pond Bridge: (span 2, 3, & 4) AASHTO Type III Girder	4000	60.00	21	1/2" dia	636200	109.96	16.16
Maipalaoa Bridge: (widening)	6000	50.00	14	12 - 1/2" dia	211680.00	62.29	35.71
Maipalaoa Bridge: (existing)	6000	50.00	18	12 - 1/2" dia	211680.00	62.29	35.71
Moanalua Bridge:	3000	8.00	9	none	NA	NA	NA
Kalihi Bridge:	3000	8.00	13	none	NA	NA	NA
Nimitz Hwy. Slip Cover #2:	3500	60.00	11	none	NA	NA	NA
Nimitz Hwy. Slip Cover #3:	3500	32.50	13	none	NA	NA	NA

Table 4.5-5: Girder Negative Bending Capacity

	Negative Bending (unsubmerged)		Negative Bending (submerged)		Loss in capacity once submerged	
	Tensile	Compression	Tensile	Compression	Tensile	Compression
Kuliouou Bridge:	63.49	194.71	26.45	157.66	58.34%	19.03%
Kahaluu Stream Bridge:	131.13	236.24	9.67	114.78	92.62%	51.41%
Ukoa Pond Bridge: (span 1) Keehi Type IV Girder	136.20	230.82	11.25	105.87	91.74%	54.13%
Ukoa Pond Bridge: (span 2, 3, & 4) AASHTO Type III Girder	66.46	112.81	-1.07	45.28	101.61%	59.86%
Maipalaoa Bridge: (widening)	80.70	147.75	64.76	131.80	19.76%	10.80%
Maipalaoa Bridge: (existing)	80.70	147.75	64.76	131.80	19.76%	10.80%
Moanalua Bridge:	68.43	68.43	39.08	39.08	42.89%	42.89%
Kalihi Bridge:	65.54	65.54	42.58	42.58	35.04%	35.04%
Nimitz Hwy. Slip Cover #2:	55.86	55.86	47.83	47.83	14.37%	14.37%
Nimitz Hwy. Slip Cover #3:	82.36	82.36	76.41	76.41	7.22%	7.22%

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_t*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 report.

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 4.6-1: Summary of Structural Capacities

	Lateral Resistance (kips)	Vertical Resistance (kips)	Overturning Resistance (kip-ft)	Negative Moment Deck Capacity (kips)	Bridge Negative Bending Capacity (kips)
Kuliouou Bridge:	518.1	1157.2	24879.9	740.2	507.9
Kahaluu Stream Bridge:	1823.9	3811.6	87665.7	3819.4	3147.1
New South Punaluu Bridge: (span #2)	725.8	1263.0	NA	NA	1263.0
Ukoa Pond Bridge:	NA	3042.3	NA	2752.1	1395.7
New Makaha Stream #3A Bridge:	9799.8	1127.1	NA	NA	1127.1
Old Makaha Stream #3A Bridge:	70.4	349.1	5731.6	303.4	NA
Maipalaoa Bridge:	263.4	1406.7	45248.9	1449.3	2582.5
Moanalua Bridge: (single span)	333.8	417.3	13422.2	453.4	615.9
Kalihi Bridge: (single span)	452.1	565.1	24958.1	619.1	852.0
Nimitz Hwy. Slip Cover #2:	1655.6	206.9	12390.0	221.0	614.4
Nimitz Hwy. Slip Cover #3:	1744.0	181.7	7448.3	245.3	1070.7

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 report 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 (η). 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:

$$d_s = \frac{base\ flood\ elevation}{(1+.65*.70)} \tag{5.1-1}$$

$$\eta = base\ flood\ elevation - \ d_s \tag{5.1-2}$$

in which

d_s = still water level (including storm surge)

η = 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.

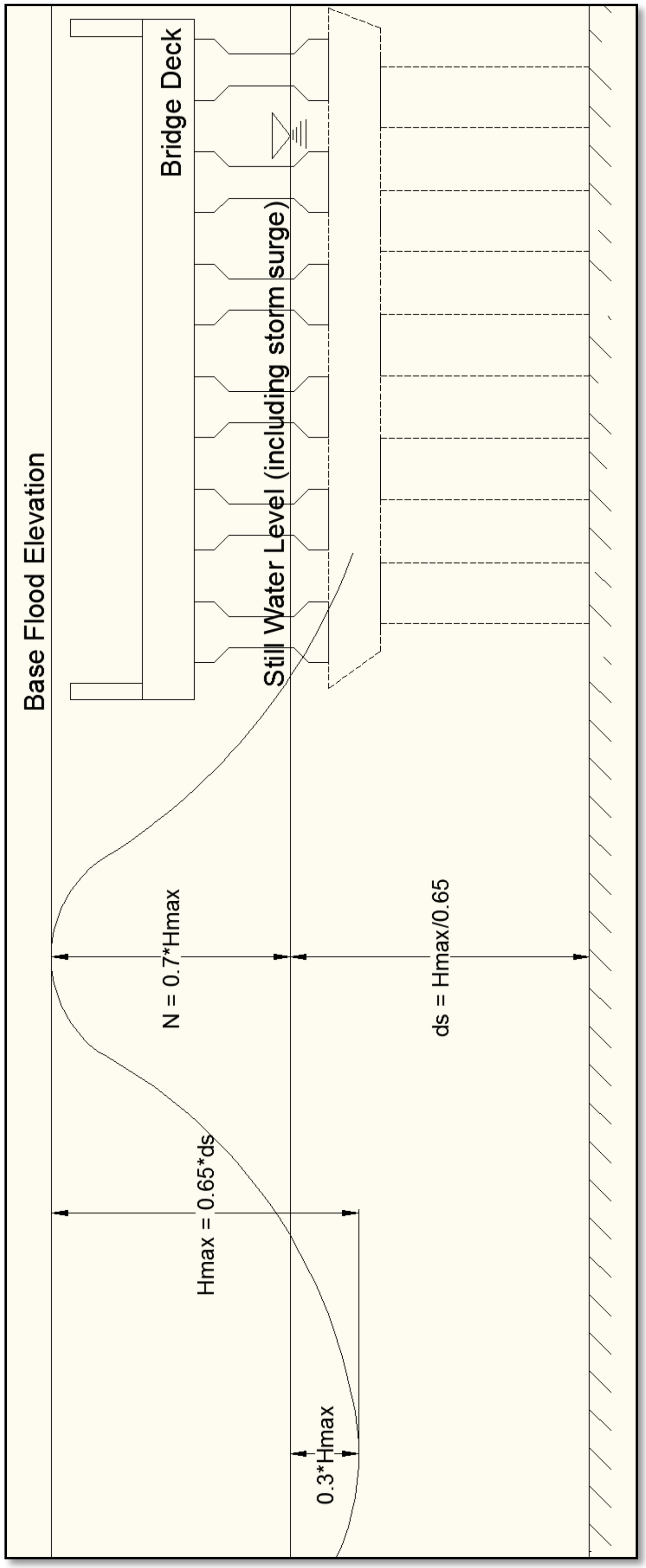


Figure 5-1: Wave Profile

Table 5.1-1: Data from FEMA Flood Insurance Study for a 100-year Storm

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

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

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

* 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 report 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

$$F_v = c_{v-va} * F_v^* \tag{2.3-1}$$

$$F_h = [1 + C_r * (N - 1)] * c_{h-va} * F_h^* \tag{2.3-2}$$

$$F_v^* = \gamma * (\Delta z_v) * A_v \tag{2.3-3}$$

$$F_h^* = \gamma * (\Delta z_h) * A_h \tag{2.3-4}$$

in which

$\mathrm{F_v}$	=	vertical wave-induced load
$\mathrm{F_h}$	=	horizontal wave-induced load
$\mathrm{F_v^*}$	=	a “reference” vertical load
$\mathrm{F_h^*}$	=	a “reference” horizontal load
$\mathrm{c_{v-va}}$	=	an empirical coefficient for the vertical “varying” load (recommended value is $c_{v\text{-}va}$ = 1 for non conservative, 2 for conservative design)
$\mathrm{c_{h-va}}$	=	an empirical coefficient for the horizontal “varying” load (recommended value is $c_{h\text{-}va}$ = 1 for non conservative, 2 for conservative design)
$\mathrm{C_r}$	=	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
γ	=	unit weight of salt water (10.06 kN/m^3 or 64 lb/ft^3)

Δz_v = 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_v = the area of the bridge contributing to vertical uplift, i.e. the projection of the bridge deck onto the horizontal plane

Δz_h = difference between the elevation of the crest of the maximum wave and the elevation of the centroid of A_h

A_h = the area of the projection of the bridge deck onto the vertical plane

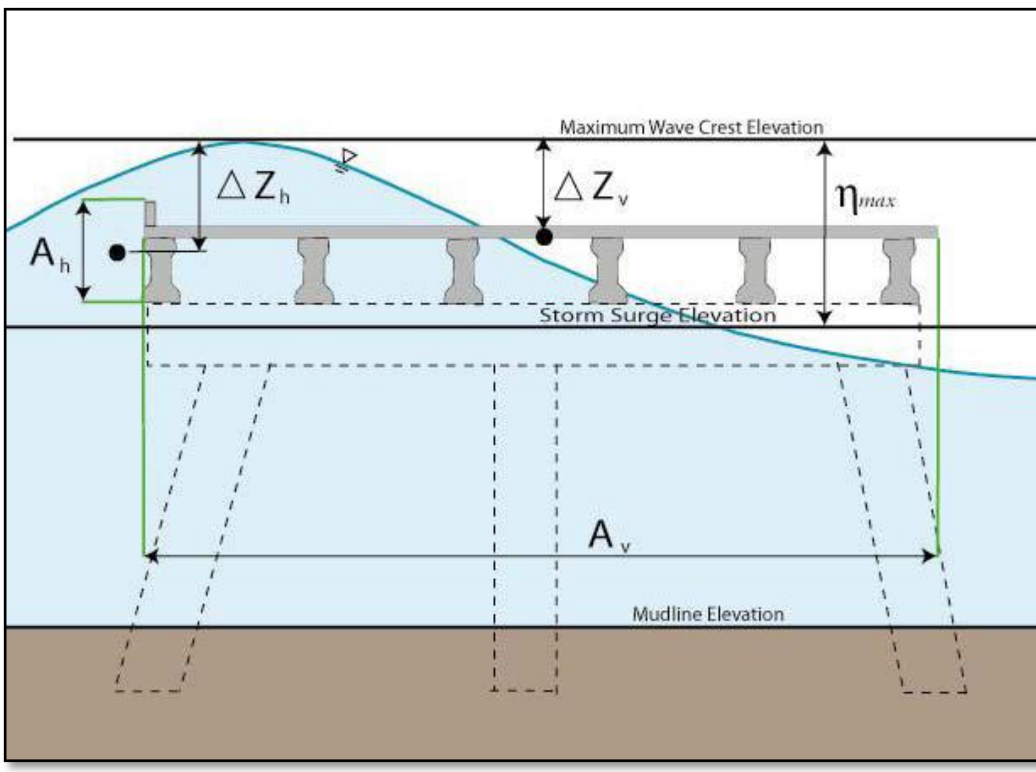


Figure 5-2: Douglass Wave Estimation Method Diagram [10]

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 5.2-1: Bridge Dimensions

	Number of Girders	Bridge Width: (ft)	Bridge Length: (ft)	Bridge Area Impacted by Vertical Force (sq ft)	Bridge Area Impacted by Lateral Force (sq ft)
Kuliouou Bridge:	12	68.75	48.40	3327.21	308.52
Kahaluu Stream Bridge:	8	46.00	318.00	14628.00	2265.75
New South Punaluu Bridge: (span	30	50.00	66.00	3300.00	371.25
Ukoa Pond Bridge:	7	48.00	270	12960.00	2396.25
New Makaha Stream #3A Bridge	1	46.83	70	3278.33	277.08
Old Makaha Stream #3A Bridge:	12	32.88	78.83	2591.65	157.67
Maipalaoa Bridge:	16	64.33	100.67	6476.22	553.67
Moanalua Bridge: (single span) *(spans 3, 4, 5, & 6)*	9	64.33	27.00	1737.00	168.19
Kalihi Bridge: (single span) (typical span)	13	88.33	27.00	2385.00	168.19
Nimitz Hwy. Slip Cover #2:	11	Variable	Variable	10704.60	973.82
Nimitz Hwy. Slip Cover #3:	13	Variable	Variable	8536.50	1310.00

Table 5.2-2: Bridge Dimensions Continued

	Height of girders: (ft)	Height of Deck: (ft)	Height of Railing: (ft)
Kuliouou Bridge:	3.00	0.67	2.71
Kahaluu Stream Bridge:	4.50	0.50	2.13
New South Punaluu Bridge:	1.75	0.88	3.00
Ukoa Pond Bridge:	4.83	0.54	3.50
New Makaha Stream #3A Bridge:	2.33	0.46	1.17
Old Makaha Stream #3A Bridge:	1.50	0.50	0.00
Maipalaoa Bridge:	3.00	0.50	2.00
Moanalua Bridge: (single span)	1.83	0.67	3.73
Kalihi Bridge: (single span) (typical span)	1.83	0.67	3.73
Nimitz Hwy. Slip Cover #2:	2.50	1.29	1.67
Nimitz Hwy. Slip Cover #3:	2.50	1.29	1.67

Table 5.2-3: Douglass Estimated Wave Forces

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

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.

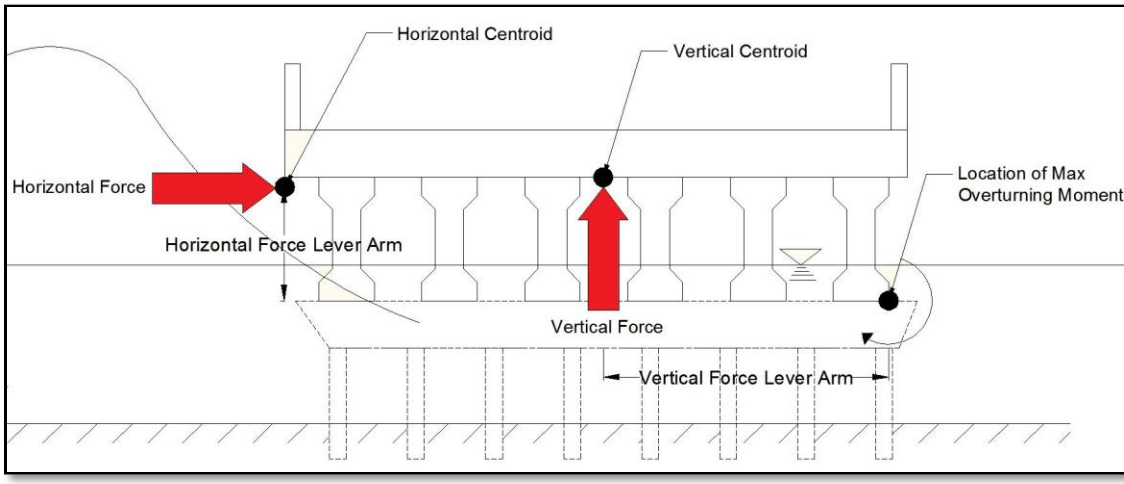


Figure 5-3: Douglass Method Resultant Force Locations

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 Δz_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.

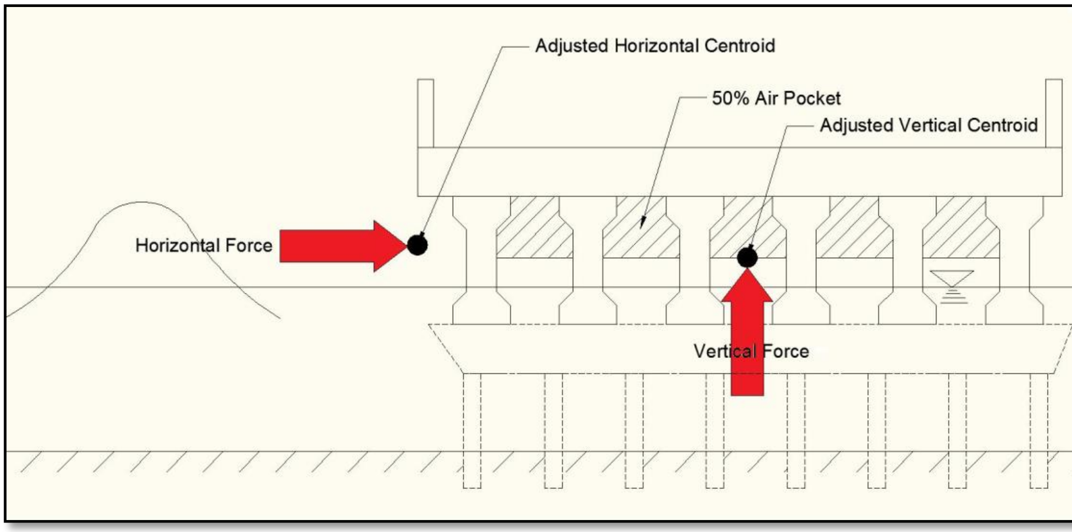


Figure 5-4: Adjusted Bridge Centroids for Douglass' Method

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 $SWL < h_{girders}$,

$$F_{Hydrostatic\_back} = 0 \tag{2.4-9}$$

and if $SWL > h_{girders}$,

$$F_{Hydrostatic\_back} = 0.5(h - h_{girder})^2 L_{bridge}\gamma \tag{2.4-10}$$

In the above equations,

γ	=	unit weight of salt water [64 lb/ft^3]
δ_z	=	distance from the top of the bridge deck to the top of the wave [ft]
δ_G	=	height of the bridge girders [ft]
δ	=	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_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]
η_{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_{bridge}	=	height of bridge impacted by lateral wave forces [ft]

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

SWL = still water level including storm surge [ft]

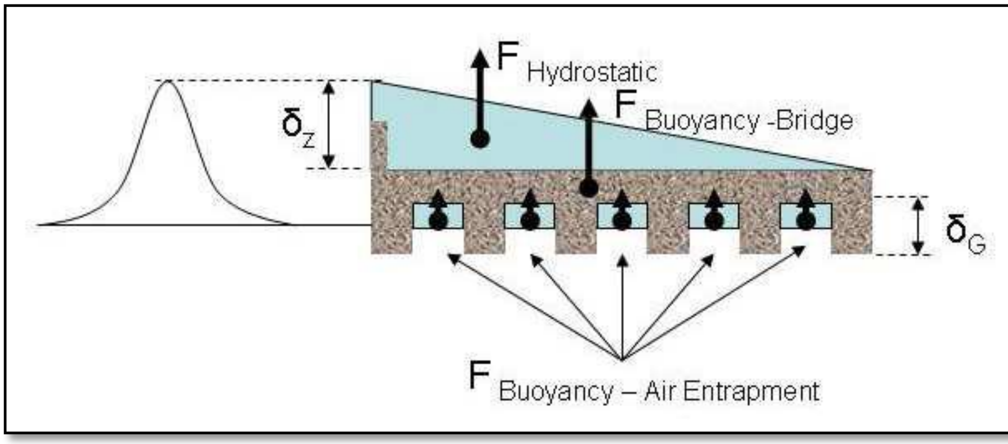


Figure 5-5: McPherson Vertical Wave Estimation Method Diagram [11]

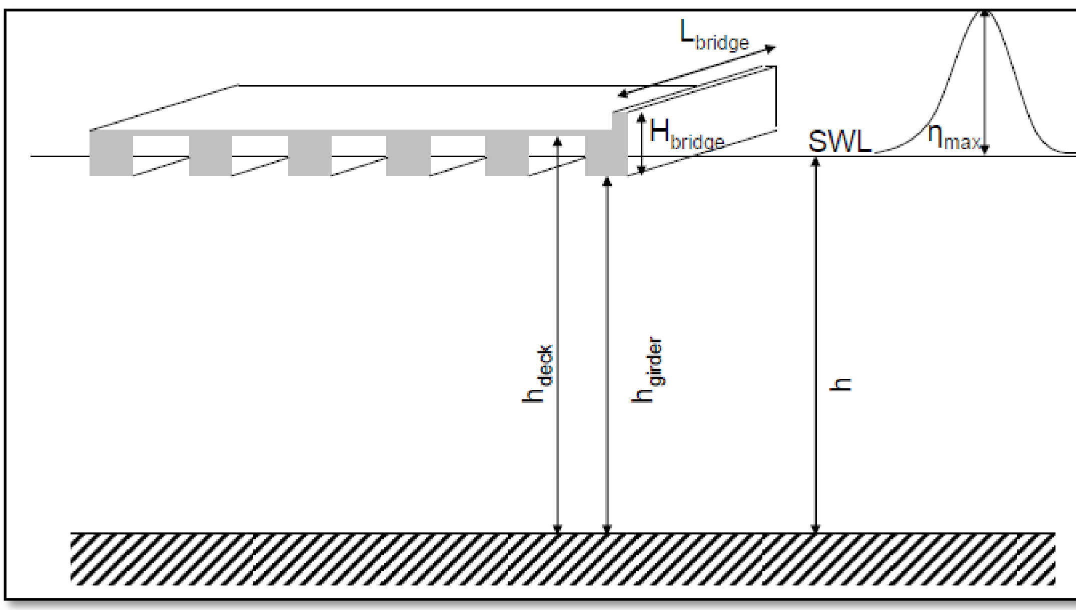


Figure 5-6: McPherson Horizontal Wave Estimation Method Diagram [11]

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

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

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

$$H_{mo} = \frac{0.21 * W^2}{g} \qquad \text{(5.4-1: } \textit{Significant Wave Height}\text{)}$$

$$W_A = 0.71 * W^{1.23} \qquad \text{(5.4-2: } \textit{Adjusted Wind Speed}\text{)}$$

$$F = \frac{g * H_{mo}}{W_A^2} * \frac{3.281}{(2.56 * 10^{-6})} \qquad \text{(5.4-3: } \textit{Fetch Length}\text{)}$$

in which

g	=	gravity [ft/sec^2]
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 report, 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 5.4-1: Design Wave Parameters

	Fetch Length (miles)	Average Water Depth over Fetch Length (ft)	Surge Depth at Bridge: (ft)	Wave Length (ft)	Hmax (ft)	η max (ft)
Kuliouou Bridge:	902.96	25.63	5.50	132.28	3.58	2.50
Kahaluu Stream Bridge:	902.96	16.38	11.00	156.93	7.15	5.01
New South Punaluu Bridge:	902.96	61.22	8.25	222.87	5.36	3.75
Ukoa Pond Bridge:	NA	NA	NA	NA	NA	NA
New Makaha Stream #3A Bridge:	902.96	27.10	8.93	171.95	5.80	4.06
Old Makaha Stream #3A Bridge:	902.96	27.10	8.93	171.95	5.80	4.06
Maipalaoa Bridge:	902.96	20.59	8.25	148.88	5.36	3.75
Moanalua Bridge: (single span) *(spans 3, 4, 5, & 6)*	902.96	7.42	6.87	80.54	4.47	3.13
Kalihi Bridge: (single span) (typical span)	902.96	7.42	6.87	80.54	4.47	3.13
Nimitz Hwy. Slip Cover #2:	902.96	6.88	5.50	70.34	3.58	2.50
Nimitz Hwy. Slip Cover #3:	902.96	6.88	5.50	70.34	3.58	2.50

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

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

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

	Douglass Vertical Force (kips)	McPherson Vertical Force (kips)	AASHTO Vertical Force (kips)	Vertical Bridge Capacity (kips)	Bridge Deck Capacity (kips)
Kuliouou Bridge:	332.19	531.23	288.66	1157.23	740.23
Kahaluu Stream Bridge:	2808.58	2876.79	3737.39	3811.55	3819.40
New South Punaluu Bridge: (span #2)	1125.70	1119.70	1054.21	NA	1262.99
Ukoa Pond Bridge:	NA	NA	NA	NA	2752.11
New Makaha Stream #3A Bridge:	505.65	495.69	402.79	NA	1127.13
Old Makaha Stream #3A Bridge:	43.07	518.46	175.75	349.13	303.37
Maipalaoa Bridge:	1036.20	1309.70	1312.27	1406.69	1449.29
Moanalua Bridge: (single span)	185.28	319.76	129.35	417.27	453.39
Kalihi Bridge: (single span) (typical span)	254.40	374.83	127.50	565.087	619.13
Nimitz Hwy. Slip Cover #2:	0.00	0.00	0.00	206.95	221.01
Nimitz Hwy. Slip Cover #3:	0.00	0.00	0.00	181.67	245.26

Table 6.1-2: Horizontal Wave Comparison

	Douglass Horizontal Force (kips)	McPherson Horizontal Force (kips)	AASHTO Horizontal Force (kips)	Horizontal Bridge Capacity (kips)
Kuliouou Bridge:	52.34	29.73	27.61	518.10
Kahaluu Stream Bridge:	929.86	380.03	647.40	1823.88
New South Punaluu Bridge: (span #2)	1277.59	54.14	75.47	725.84
Ukoa Pond Bridge:	NA	NA	NA	NA
New Makaha Stream #3A Bridge:	7.64	21.37	53.80	9799.82
Old Makaha Stream #3A Bridge:	41.41	8.88	34.95	70.37
Maipalaoa Bridge:	682.12	87.61	128.30	263.35
Moanalua Bridge: (single span)	17.42	18.72	20.59	333.82
Kalihi Bridge: (single span)	24.06	24.63	20.59	452.07
Nimitz Hwy. Slip Cover #2:	0.00	0.00	0.00	1655.56
Nimitz Hwy. Slip Cover #3:	0.00	0.00	0.00	1744.02

Table 6.1-3: Overturning Moment Comparison

	Douglass Overturning Moment (kip-ft)	McPherson Overturning Moment (kip-ft)	AASHTO Overturning Moment (kip-ft)	Overturning Moment Capacity (kip-ft)
Kuliouou Bridge:	11585.83	18355.91	15296.27	24879.90
Kahaluu Stream Bridge:	67909.89	67519.95	112176.33	87665.70
New South Punaluu Bridge: (span #2)	31735.61	28144.71	14407.44	NA
Ukoa Pond Bridge:	NA	NA	NA	NA
New Makaha Stream #3A Bridge:	11855.76	11649.65	13197.16	NA
Old Makaha Stream #3A Bridge:	749.38	8531.07	4371.66	5731.55
Maipalaoa Bridge:	35206.78	42369.57	38712.95	45248.85
Moanalua Bridge: (single span)	6014.11	10343.86	3348.62	13422.19
Kalihi Bridge: (single span)	11310.94	16631.87	4505.59	24958.14
Nimitz Hwy. Slip Cover #2:	0.00	0.00	0.00	12390.00
Nimitz Hwy. Slip Cover #3:	0.00	0.00	0.00	7448.30

Table 6.1-4: Girder Negative Bending Capacity Comparison

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

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.

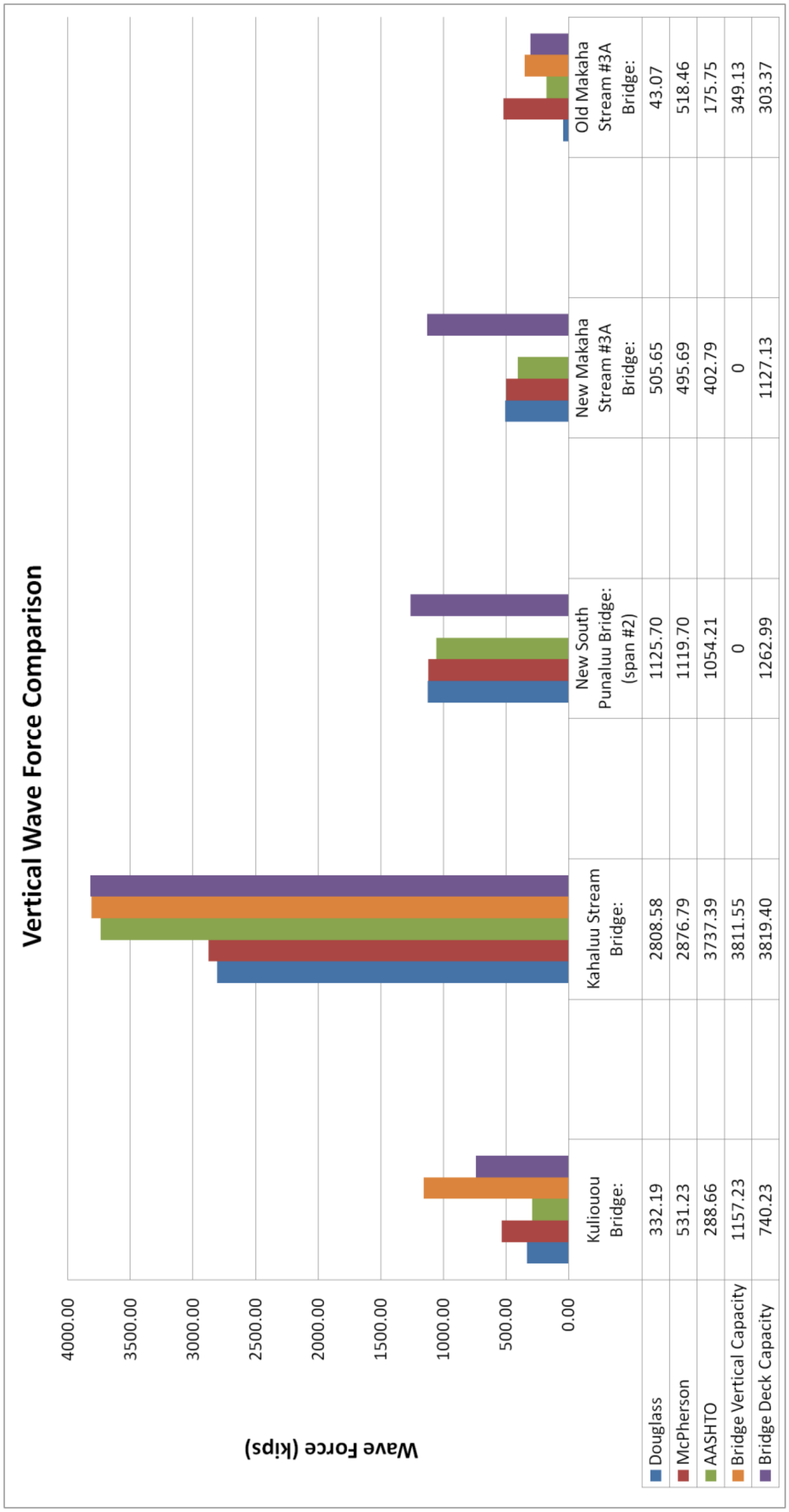


Figure 6-1: Graphical Comparison of Vertical Wave Forces

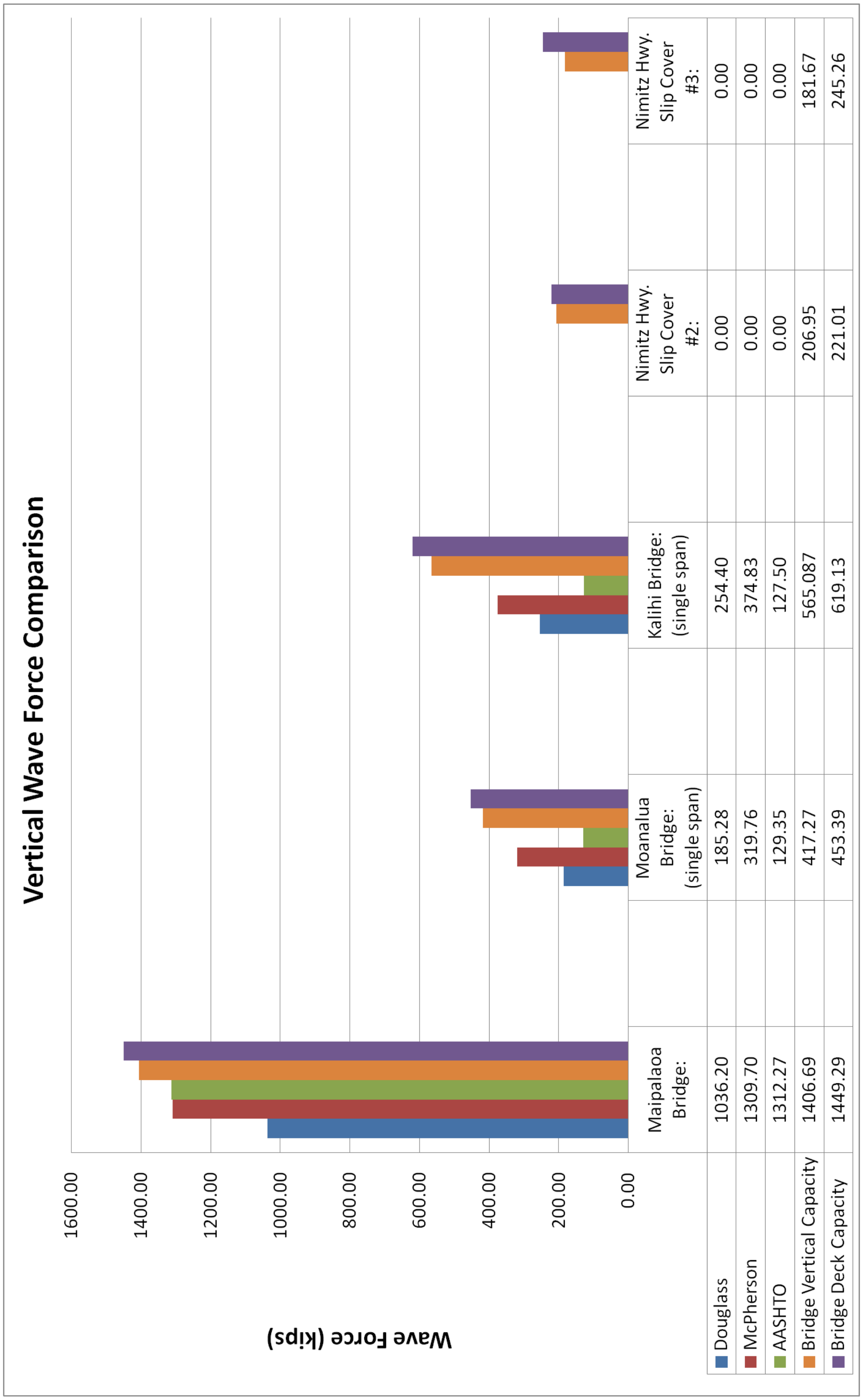
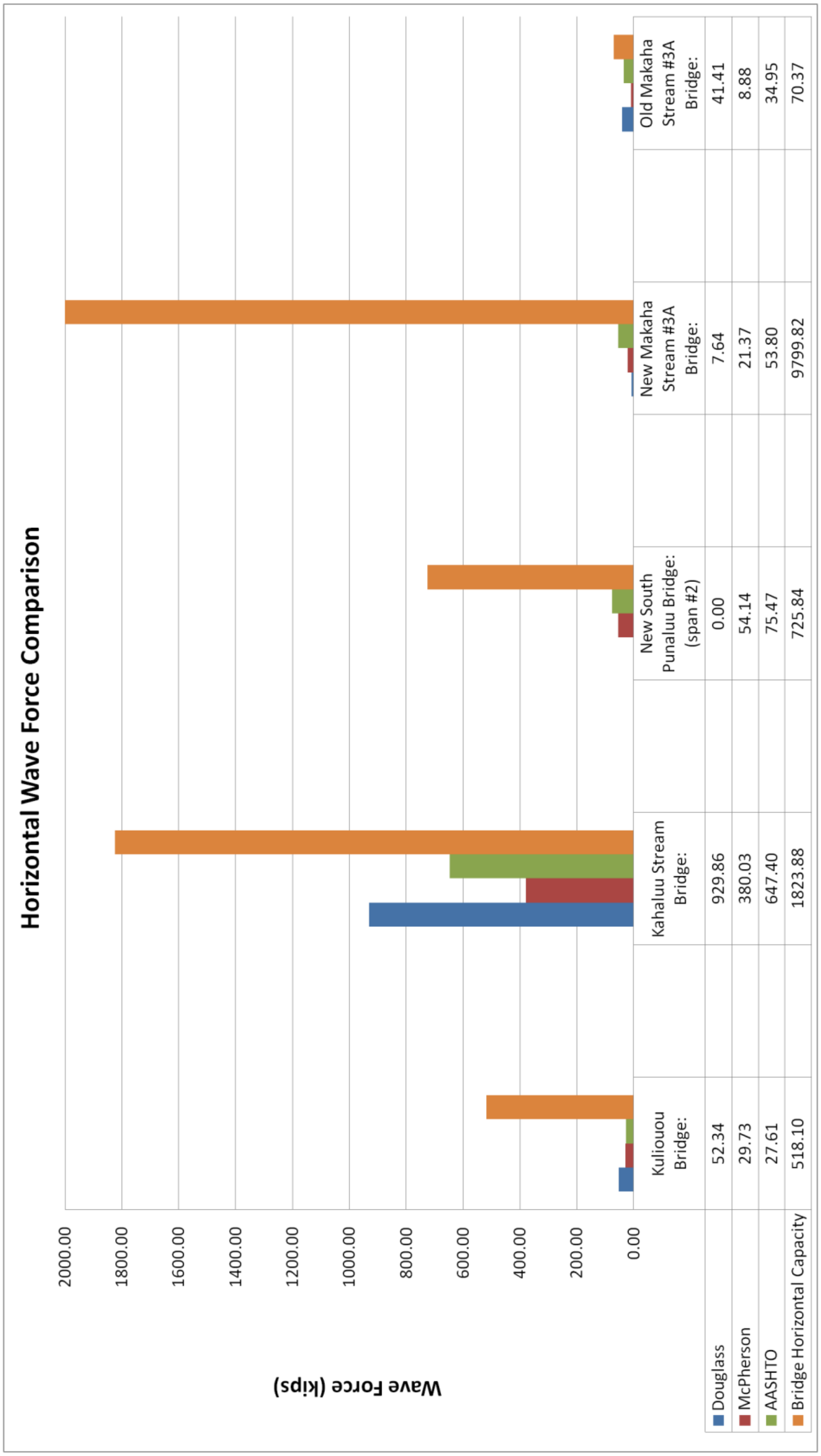


Figure 6-2: Graphical Comparison of Vertical Wave Forces cont.



	Kuliouou Bridge:	Kahaluu Stream Bridge:	New South Punaluu Bridge: (span #2)	New Makaha Stream #3A Bridge:	Old Makaha Stream #3A Bridge:
Douglass	52.34	929.86	0.00	7.64	41.41
McPherson	29.73	380.03	54.14	21.37	8.88
AASHTO	27.61	647.40	75.47	53.80	34.95
Bridge Horizontal Capacity	518.10	1823.88	725.84	9799.82	70.37

Figure 6-3: Graphical Comparison of Horizontal Wave Forces

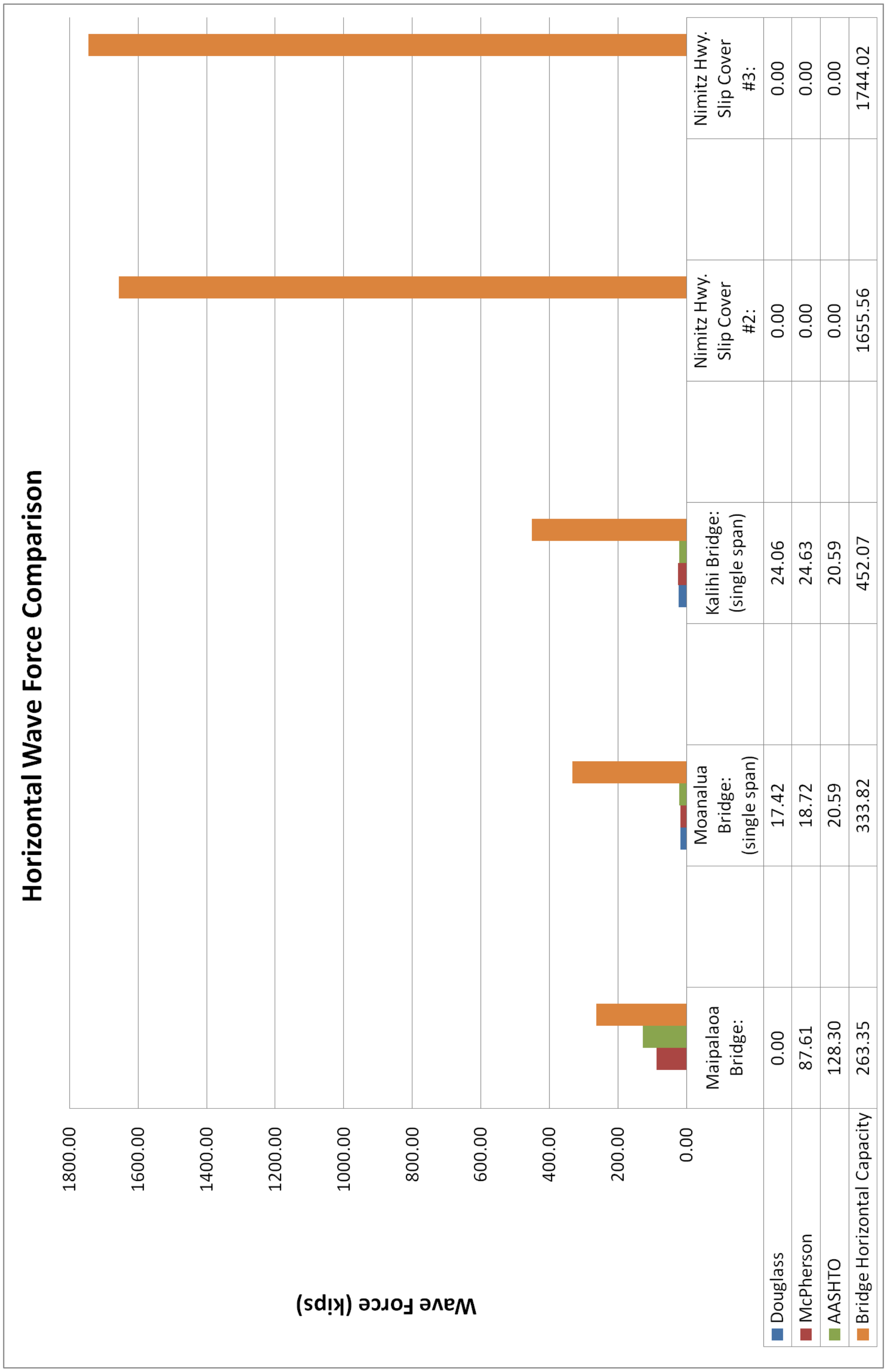


Figure 6-4: Graphical Comparison of Horizontal Wave Forces cont.

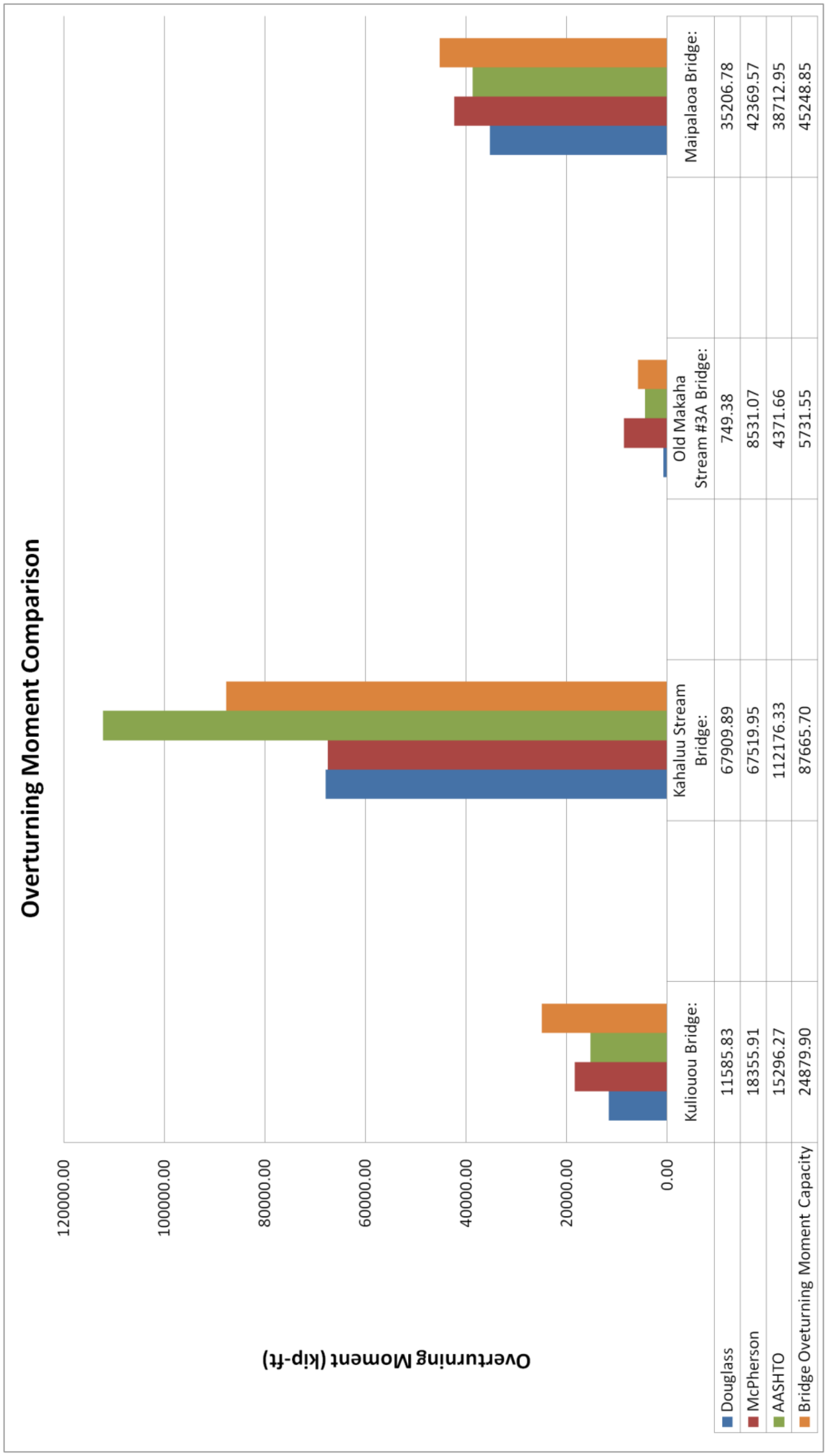
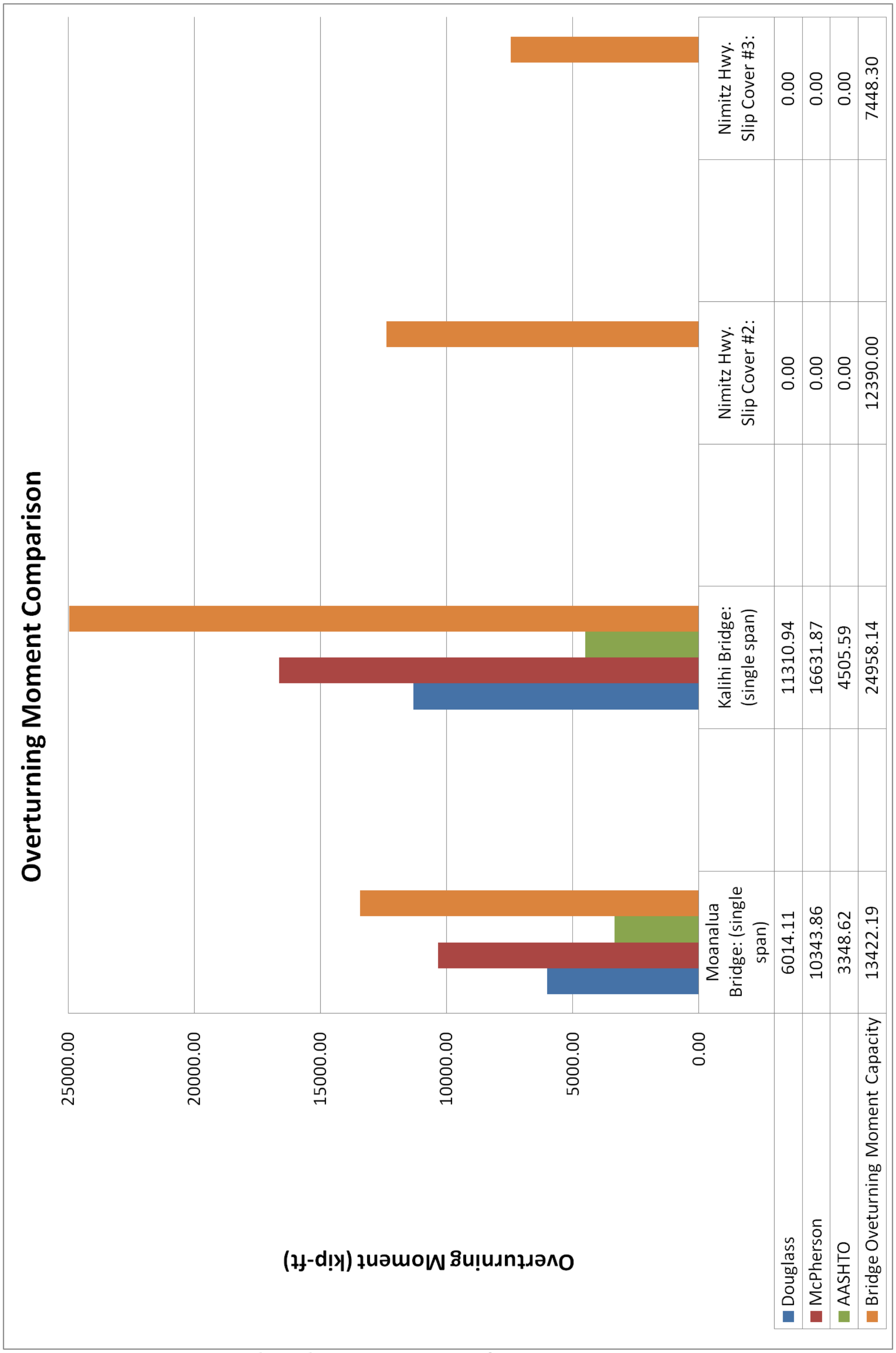


Figure 6-5: Graphical Comparison of Overturning Moments



	Moanalua Bridge: (single span)	Kalihi Bridge: (single span)	Nimitz Hwy. Slip Cover #2:	Nimitz Hwy. Slip Cover #3:
Douglass	6014.11	11310.94	0.00	0.00
McPherson	10343.86	16631.87	0.00	0.00
AASHTO	3348.62	4505.59	0.00	0.00
Bridge Oveturning Moment Capacity	13422.19	24958.14	12390.00	7448.30

Figure 6-6: Graphical Comparison of Overturning Moments cont.

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 over predicts 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 report.

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 report.

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 6.3-1: Bridge Factor of Safeties above AASHTO Wave Forces

	Vertical Factor of Safety	Horizontal Factor of Safety	Overturning Moment FS	Bridge Condition
Kuliouou Bridge:	4.01	18.76	1.63	OK
Kahaluu Stream Bridge:	1.02	2.82	0.78	NG
New South Punaluu Bridge:	1.20	9.62	NA	OK
Ukoa Pond Bridge:	NA	NA	NA	OK
New Makaha Stream #3A Bridge:	2.80	182.16	NA	OK
Old Makaha Stream #3A Bridge:	1.99	2.01	1.31	OK
Maipalaoa Bridge:	1.07	1.81	1.17	OK
Moanalua Bridge: (single span) *(spans 3, 4, 5, & 6)*	3.23	16.21	4.01	OK
Kalihi Bridge: (single span) (typical span)	4.43	21.96	5.54	OK
Nimitz Hwy. Slip Cover #2:	NA	NA	NA	OK
Nimitz Hwy. Slip Cover #3:	NA	NA	NA	OK

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. Colum 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 6.4-1: Method Which Estimates the Largest Force

	Largest Vertical Force Estimated By	Largest Horizontal Force Estimated By	Largest Overturning Moment Estimated By
Kuliouou Bridge:	McPherson	Douglass	McPherson
Kahaluu Stream Bridge:	AASHTO	Douglass	AASHTO
New South Punaluu Bridge:	Douglass	AASHTO	–
Ukoa Pond Bridge:	–	–	–
New Makaha Stream #3A Bridge:	Douglass	AASHTO	–
Old Makaha Stream #3A Bridge:	McPherson	Douglass	McPherson
Maipalaoa Bridge:	AASHTO	AASHTO	McPherson
Moanalua Bridge:	McPherson	AASHTO	McPherson
Kalihi Bridge:	McPherson	McPherson	McPherson
Nimitz Hwy. Slip Cover #2:	–	–	–
Nimitz Hwy. Slip Cover #3:	–	–	–

Table 6.4-2: Calculated Factors of Safety

	Displacement			Superstructure Failure	
	Vertical	Lateral	Overturning	Deck Capacity	Bridge Negative Bending
Kuliouou Bridge:	2.18	9.90	1.36	1.39	1.43
Kahaluu Stream Bridge:	1.02	1.96	0.78	1.02	0.84
New South Punaluu Bridge:	—	—	—	—	1.12
Ukoa Pond Bridge:	—	—	—	—	—
New Makaha Stream #3A Bridge:	—	—	—	—	2.23
Old Makaha Stream #3A Bridge:	0.67	1.70	0.67	0.59	NA
Maipalaoa Bridge:	1.07	2.05	1.07	1.10	1.97
Moanalua Bridge:	1.30	16.21	1.30	1.42	1.93
Kalihi Bridge:	1.51	18.36	1.50	1.65	2.27
Nimitz Hwy. Slip Cover #2:	—	—	—	—	—
Nimitz Hwy. Slip Cover #3:	—	—	—	—	—

Table 6.4-3: Summary of Bridges

	Estimated Potential Failure Mode	Notes	Bridge Vulnerability
Kuliouou Bridge:	● None	The estimated wave forces are less than the calculated capacities.	OK
Kahaluu Stream Bridge:	● Displacement - Overturning ● Bridge Negative Bending Failure	The estimated wave forces by McPherson's method are greater than the overturning resistance and the bridge negative bending capacity.	At Risk
New South Punaluu Bridge:	● None	The estimated vertical wave loads are below the negative bending capacity of the bridge deck.	OK
Ukoa Pond Bridge:	● Buoyancy	An 84% air pocket will cause the buoyancy force to be greater than the total vertical capacity of the bridge.	At Risk
New Makaha Stream #3A Bridge:	● None	The estimated vertical wave forces are less than the calculated negative bending capacity of the bridge deck.	OK
Old Makaha Stream #3A Bridge:	● Displacement - Vertical ● Displacement - Overturning	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.	At Risk
Maipalaoa Bridge:	● Displacement - Vertical ● Displacement - Overturning	The estimated vertical wave force by AASHTO and overturning moment estimated by McPherson nearly exceed the bridge capacities. Repeated wave impacts may cause failure.	At Risk
Moanalua Bridge:	● None	The estimated wave forces are less than the calculate capacities.	OK
Kalihi Bridge:	● None	The estimated wave forces are less than the calculate capacities.	OK
Nimitz Hwy. Slip Cover #2:	● None	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.	OK
Nimitz Hwy. Slip Cover #3:	● None	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.	OK

7 Conclusions/ Recommendations

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 prestresssed 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

1. "Japan’s 2011 Earthquake and Tsunami: Economic Effects and Implications for the United States." *International Trade*. Web. 09 Aug. 2011. <http://international-trade-reports.blogspot.com/2011/06/japans-2011-earthquake-and-tsunami.html>.

2. "Indian Ocean Tsunami." *Global Education*. Web. 09 Aug. 2011. <http://www.globaleducation.edna.edu.au/globaled/go/pid/2258>.

3. "Hurricane Katrina Facts." *Hurricane-Facts.com*. Web. 09 Aug. 2011. <http://www.hurricane-facts.com/Hurricane-Katrina-Facts.php>.

4. *Steel Construction Manual*. (2005). (Thirteenth.). American Institute of Steel Construction.

5. *PCI Design Handbook Precast and Prestressed Concrete*. (1999). (Fifth.). Precast/Prestressed Concrete Institute.

6. *Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary*. (2008). American Concrete Institute.

7. Robertson, I. N., Yim, S., Riggs, H. R., & Young, Y. L. (2007). Coastal bridge performance during Hurricane Katrina. *Structural Engineering, Mechanics, and Computation 3* (pp. 1864-1870). South Africa: Millpress, The Netherlands.

8. Hayes, M. B., & Mertz, D. R. (2008). ASSESSING THE VULNERABILITY OF DELAWARE’S COASTAL BRIDGES TO HURRICANE FORCES. University of Delaware.

9. AASHTO, & Officials, A. A. O. S. H. A. T. (2008). *Guide Specifications for Bridges Vulnerable to Coastal Storms* (p. 55). Federal Highway Administration.

10. Douglass, Scott; Chen, Qin; Olsen, J. (2006). Wave Forces on Bridge Decks during Hurricanes and Impact on the Foundation, (June), 13-13. Asce. doi:10.1061/40962(325)13

11. McPherson, R. L. (2008). *Hurricane Induced Wave and Surge Forces on Bridge Decks*. Texas A&M University.

12. McConnell, K. (2004). Piers, Jetties, and Related Structures Exposed to Waves: Guidelines for Hydraulic Loadings. *Thomas Telford Press, London*.

13. Boon-intra, S., Engineering, C., Forces, E. T., & Superstructures, B. (2010). Development of a Guideline for Estimating Tsunami Forces on Bridge Superstructures.

14. Bea, R.G.; Fellow; ASCE; Xu, T.; Stear, J.; Ramos, R. (1999). Wave Forces on Decks of Offshore Platforms. *Manager*, (June), 136-144.

15. Robertson, I. N., Riggs, H. R., Yim, S. C. S., & Young, Y. L. (2007). Lessons from Hurricane Katrina Storm Surge on Bridges and Buildings. *Journal of Waterway, Port, Coastal, and Ocean Engineering*, *133*(6), 463. doi:10.1061/(ASCE)0733-950X(2007)133:6(463)

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

17. Lehrman, J. (2010). *Laboratory Performance Of Highway Bridge Girder Anchorages Under Hurricane Induced Wave Loading*. Oregon State University.

18. "Storm Surge Overview." *National Hurricane Center*. Web. 01 Oct. 2011. <http://www.nhc.noaa.gov/surge/>.

19. Cox, D. (2011). The Tohoku , Japan , Tsunami of March 11 , 2011 : Effects on Structures. *Earthquake*, (September), 1-14.

20. Federal Emergency Management Agency. (n.d.). *Flood Insurance Study*.

21. Sorensen, R. M. (2006). *Basic Coastal Engineering* (Third.). Springer.

22. Laboratory, F. P. (1999). *Wood handbook—Wood as an engineering material*. U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.

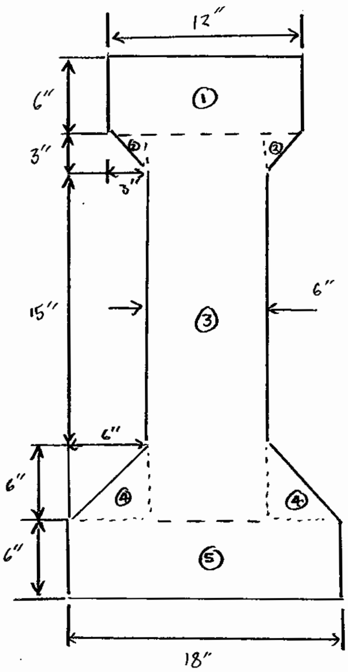
Appendices

Appendix A: Hand Calculations

Calculations By: Daniel Lum

Buoyancy Calculations:

- Typical Girder Section: (Widening)
  - Sheet 32 section L



$A_1 = 6 \times 12 = 72\ in^2$

$A_2 = 2\left(\frac{1}{2}(3)(3)\right) = 9\ in^2$

$A_3 = 6 \times 24 = 144\ in^2$

$A_4 = 2\left[\frac{1}{2}(6)(6)\right] = 36\ in^2$

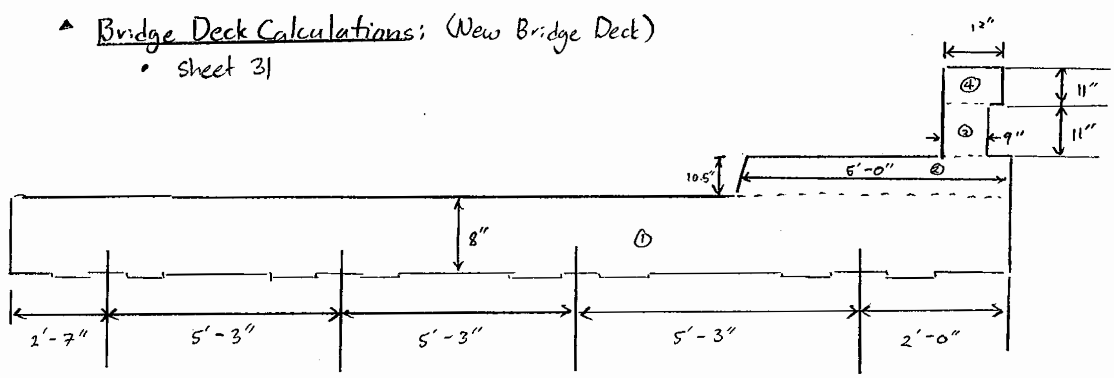
$A_5 = 6 \times 18 = 108\ in^2$

- $A_T = \sum_{i=1}^{5} A_i = 369\ in^2$

$\boxed{A_T = 2.5625\ ft^2}$ Girder

Length ≈ 53 ft

- Bridge Deck Calculations: (New Bridge Deck)
  - Sheet 31



* Guardrail detail sheet 42

$A_1 = (8/12)\left([2 + 7/12] + 5.25 + 5.25 + 5.25 + 2.0\right) = 13.56\ ft^2$

$A_2 = (5)(10.5/12) = 4.375\ ft^2$

$A_3 = (9/12)(11/12) = 0.6875\ ft^2$

$A_4 = (12/12)(11/12) = 0.9167\ ft^2$

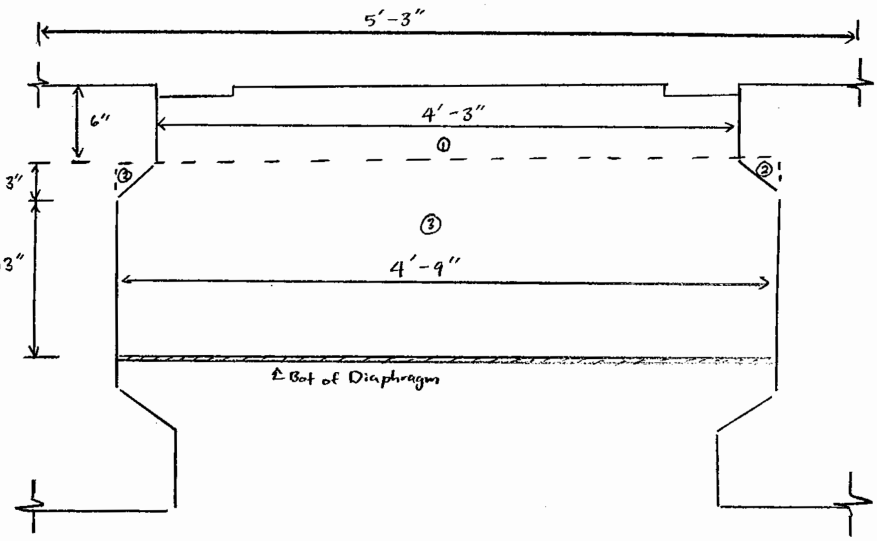
- $A_T = A_1 + A_2 = 17.935\ ft^2$
- $A_{guardrails} = A_3 + A_4 = 1.604\ ft^2$

$\boxed{A_{gr} = 1.604\ ft^2}$ Guard Rails

$\boxed{A_T = 17.94\ ft^2}$ New Deck no girders

▲ Concrete Diaphragm: (New Section)

- Sheet 31 for geometry
  sheet 32 for Diaphragm



$A_1 = (4 + 3/12)(6/12) = 2.125\ ft^2$

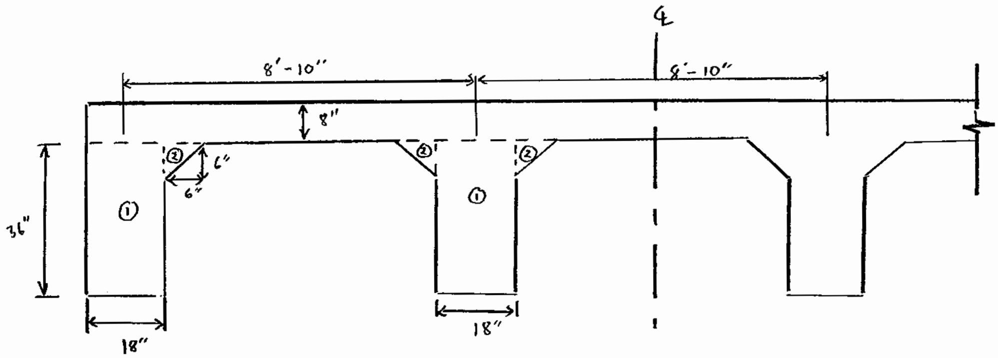
$A_2 = 2\left[\tfrac{1}{2}(3/12)(3/12)\right] = .0625\ ft^2\ (-)$

$A_3 = (4 + 9/12)(16/12) = 6.333\ ft^2$

- $A_T = A_1 - A_2 + A_3 = 8.39583\ ft^2$

$\boxed{A_T = 8.396\ ft^2}$ Concrete Diaphragm
(New Bridge Section)
Thickness = 1.25'
Amount = 6

▲ Original Bridge Section:



○ Side Girders:

$$A_1 = (18/12)(36/12) = 4.5\ ft^2$$

$$A_2 = \tfrac{1}{2}(6/12)(6/12) = 0.125\ ft^2$$

• $A_T = A_1 + A_2 = 4.625\ ft^2$

$\boxed{A_T = 4.625\ ft^2}$ Side Girder

○ Inner Girders:

$$A_1 = (18/12)(36/12) = 4.5$$

$$A_2 = 2\left[\tfrac{1}{2}(6/12)(6/12)\right] = 0.25\ ft^2$$

• $A_T = A_1 + A_2 = 4.75\ ft^2$

$\boxed{A_T = 4.75\ ft^2}$ Inner Girder

○ Deck:

$$\text{Width} = (8 + 10/12) + (8 + 10/12) + (8 + 10/12) + (9/12) + (9/12)$$
$$= 28\ ft$$

$$A_T = (28)(8/12) = 18.67\ ft^2$$

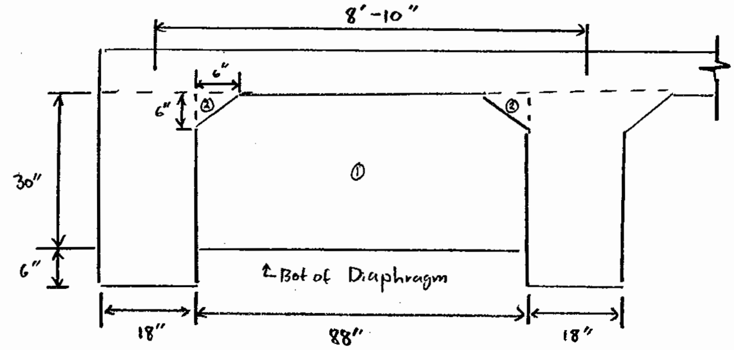
$\boxed{A_T = 18.67\ ft^2}$ Deck

○ Total Area:

$$A_T = (2)\,\text{Side} + (2)\,\text{Inner} + \text{Deck}$$
$$= (2)(4.625) + (2)(4.75) + 18.67$$
$$A_T = 37.4167\ ft^2$$

$\boxed{A_T = 37.42\ ft^2}$ Total Area Original Section

▲ Concrete Diaphragm: (original Section)



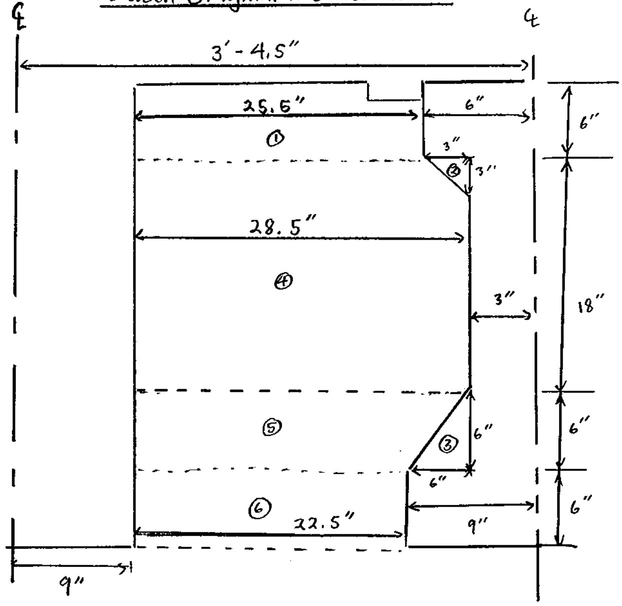
$$A_1 = (88/12)(30/12) = 18.333\ ft^2$$

$$A_2 = 2[\tfrac{1}{2}(6/12)^2] = 0.25\ ft^2\ (-)$$

$$\bullet\ A_T = A_1 - A_2 = 18.083\ ft^2$$

$\boxed{A_T = 18.08\ ft^2}$ Concrete Diaphragm (original Section)
Thickness = 1.25'
Amount = 3

- Air Pocket Calculations: (New Bridge Section)
  - Sheet 31:
    - Between Original and New Section:



★ Note: Air is assumed to fill the entire area between old and new section girders.

$$A_1 = (25.5/12)(6/12) = 1.0625\ ft^2$$

$$A_2 = \tfrac{1}{2}(3/12)(3/12) = .03125\ ft^2\ (-)$$

$$A_3 = \tfrac{1}{2}(6/12)(6/12) = 0.125\ ft^2\ (-)$$

$$A_4 = (28.5/12)(18/12) = 3.5625\ ft^2$$

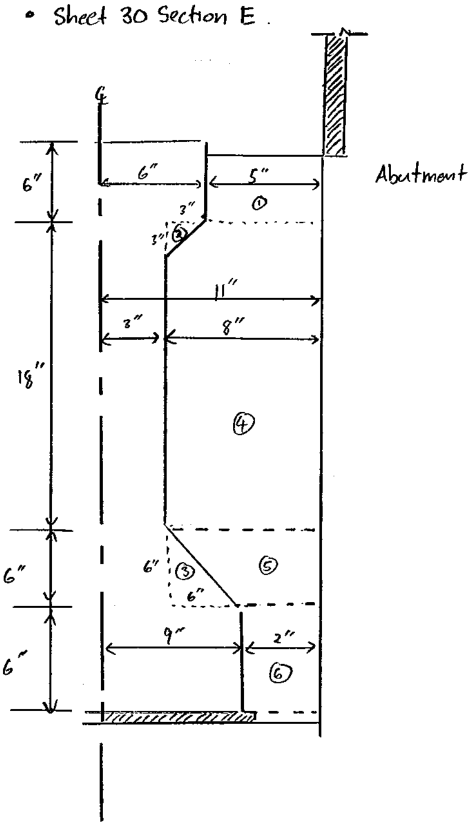
$$A_5 = (28.5/12)(6/12) = 1.1875\ ft^2$$

$$A_6 = (22.5/12)(6/12) = 0.9375\ ft^2$$

- $A_T = A_1 - A_2 - A_3 + A_4 + A_5 + A_6 = 6.594\ ft^2$

$\boxed{A_T = 6.594\ ft^2}$ Air Pocket between old & new section

- Between Abutment and Girder:
  - Sheet 30 Section E



$$A_1 = (5/12)(6/12) = 0.2083\ ft^2$$

$$A_2 = \tfrac{1}{2}(3/12)(3/12) = 0.0313\ ft^2\ (-)$$

$$A_3 = \tfrac{1}{2}(6/12)(6/12) = 0.125\ ft^2\ (-)$$

$$A_4 = (8/12)(18/12) = 1.0\ ft^2$$

$$A_5 = (8/12)(6/12) = 0.333\ ft^2$$

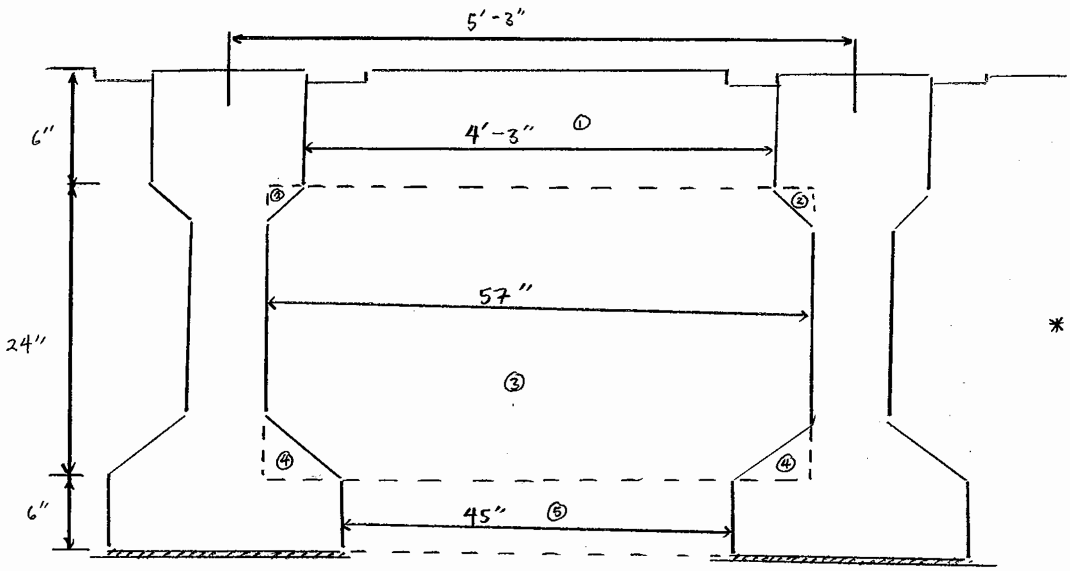
$$A_6 = (2/12)(6/12) = 0.833\ ft^2$$

$$A_T = A_1 - A_2 - A_3 + A_4 + A_5 + A_6$$
$$= 0.2083 - 0.0313 - 0.125 + 1.0 + 0.333 + 0.833$$
$$A_T = 1.4687\ ft^2$$

$\boxed{A_T = 1.47\ ft^2}$ Air pocket between abut. & girder

▲ Air Pocket Calculations : (New Bridge Section)

○ Between Girders: (No creep block)



* Note: Air fills entire area between girders

$A_1 = (4+3/12)(6/12) = 2.125\ ft^2$

$A_2 = 2[1/2(3/12)(3/12)] = .0625\ ft^2\ (-)$

$A_3 = (57/12)(24/12) = 9.5\ ft^2$

$A_4 = 2[1/2(6/12)(6/12)] = 0.25\ ft^2\ (-)$

$A_5 = (45/12)(6/12) = 1.875\ ft^2$

- $A_T = A_1 - A_2 + A_3 - A_4 + A_5 = 13.1875\ ft^2$

$\boxed{A_T = 13.19\ ft^2}$ Air Pocket

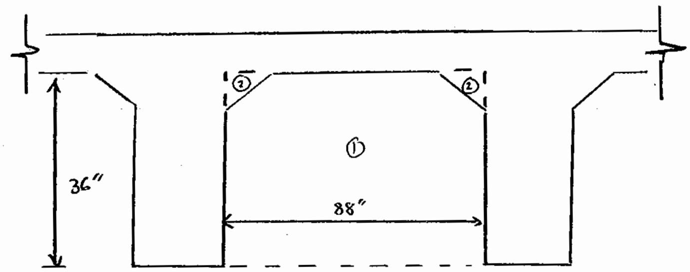
○ Between Girders: (creep block)

- $A_T = A_1 - A_2 + A_3 - A_4 = 11.313\ ft^2$

$\boxed{A_T = 11.31\ ft^2}$

▲ <u>Air Pocket Calculations</u>: (Original Section)

○ <u>Between Girders</u>:



* Note: Air fills entire area between girders

$$A_1 = (88/12)(36/12) = 22\ ft^2$$

$$A_2 = 2\,[1/2\,(6/12)(6/12)] = .25\ ft^2\ (-)$$

• $A_T = A_1 - A_2 = 21.75\ ft^2$

$\boxed{A_T = 21.75\ ft^2}$ Air Pocket

▲ <u>Reduction of air pocket area</u>:

* Note: The bridge is assumed to be submerged to the top of deck.

○ <u>For all air pockets</u>:

• $h = (8/12) + (36/12) = 3.667\ ft$

• $P_2 = 14.7 + h(64/144)$
$= 14.7 + (3.667)(64/144)$
$P_2 = 16.3296\ psi$

○ <u>Between Girders</u>: (New Section) – creep block

• $A_2 = \dfrac{P_1 A_1}{P_2} = \dfrac{(14.7)(11.3125)}{(16.3296)}$

$A_2 = 10.1836\ ft^2$

$\boxed{A_T = 10.184\ ft^2}$

○ <u>Between Original and New Section</u>:

• $A_2 = \dfrac{P_1 A_1}{P_2} = \dfrac{(14.7)(6.594)}{16.3296}$

$A_2 = 5.93595\ ft^2$

$\boxed{A_T = 5.936\ ft^2}$

○ <u>Between Girders</u>: (New Section) – no creep block

• $A_2 = \dfrac{P_1 A_1}{P_2} = \dfrac{(14.7)(13.19)}{16.3296}$

$A_2 = 11.8714\ ft^2$

$\boxed{A_T = 11.871\ ft^2}$

○ <u>Between Abutment and Girder</u>:

• $A_2 = \dfrac{P_1 A_1}{P_2} = \dfrac{(14.7)(1.4687)}{16.3296}$

$A_2 = 1.32213\ ft^2$

$\boxed{A_T = 1.322\ ft^2}$

○ <u>Between Girders</u>: (Original Section)

• $A_2 = \dfrac{P_1 A_1}{P_2} = \dfrac{(14.7)(21.75)}{16.3296}$

$A_2 = 19.5794\ ft^2$

$\boxed{A_T = 19.58\ ft^2}$

- Collection of Data:
  - Concrete: ($\gamma_{RC} = 150\ lb/ft^3$)
    - New Section Girder:
      - Area = $2.563\ ft^2$
      - Amount = 8
      - Length = 48.39′
    - Diaphragm: (New Section)
      - Area = $8.396\ ft^2$
      - Amount = 6
      - Length = 1.25′
    - New Section Deck:
      - Area = $17.94\ ft^2$
      - Amount = 2
      - Length = 48.39′
    - Diaphragm: (Original Section)
      - Area = $18.08\ ft^2$
      - Amount = 3
      - Length = 1.25′
    - Railing:
      - Area = $1.604\ ft^2$
      - Amount = 2
      - Length = 48.39′
    - Original Section:
      - Area = $37.42\ ft^2$
      - Amount = 1
      - Length = 48.39′
  - Air Pocket: (Compressed) ($\gamma_{seawater} = 64\ lb/ft^3$)
    - Between Original and New Section:
      - Area = $5.936\ ft^2$
      - Amount = 2
      - Length = 48.39′
    - Between Girders: (New Section) - Creep block
      - Area = $10.184\ ft^2$
      - Amount = 2
      - Length = 12.098 ft
    - Between Abutment and Girder:
      - Area = $1.322\ ft^2$
      - Amount = 2
      - Length = 48.39′
    - Between Girders: (New Section) - No Creep block
      - Area = $11.871\ ft^2$
      - Amount = 4 | Amount = 2
      - Length = 48.39′ | Length = 36.293 ft
    - Between Girders: (original Section)
      - Area = $19.58\ ft^2$
      - Amount = 3
      - Length = 48.39′

- Buoyancy Calculations:
  - Self Weight:

$$\text{Self Weight} = \gamma_{RC}\{8(\text{New Girder}) + 2\,\text{Deck} + 2(\text{Railing}) + \text{Original Section}\}(48.39) + \{6(\text{New Dia})(1.25) + 3(\text{Original Dia})(1.25)\}\gamma_{RC}$$

$$= (150)(48.39)\{8(2.563) + (2)(17.94) + 2(1.604) + 37.42\} + (150)(1.25)\{6(8.396) + 3(18.08)\}$$

$$\boxed{SW = 723777\ lbs}$$

  - Buoyant Force:

$$\text{Submerged Volume} = \text{Submerged Concrete} + \text{Air Pocket}$$

$$= [(8(2.563) + (2)(17.94) + (37.42) + (2)(5.936) + 2(1.322) + 4(11.871) + 3(19.58)](48.39) + 2(10.184)(12.098) + 2(11.871)(36.293)$$

$$SV = 11489.9\ ft^3$$

- Buoyancy Calculations: cont.

  - Buoyant Force:

$$\text{Buoyant Force} = S\forall \times \gamma_{sw}$$
$$= (11489.9)(64)$$
$$\boxed{BF = 735351.34 \text{ lbs.}}$$

  - Residual Weight:

$$\text{Residual Weight} = SW - BF$$
$$= (723777 - 735351.34)$$
$$\boxed{RW = -11574.2 \text{ lbs}}$$

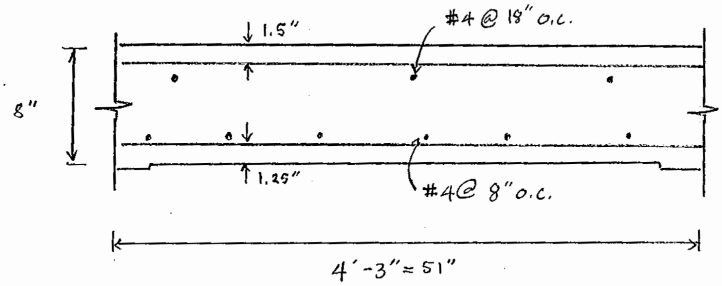
% Retained Weight = -1.6%

- Summary of Results:

  - Self Weight = 723.8 kips
  - Buoyant Force = 735.4 kips
  - Residual Weight = -11.57 kips
  - % Retained Weight = -1.6%

∴ Bridge is Buoyant

■ Deck Capacity: (Widened Section)



$f_c' = 3000psi$
$f_y = 40{,}000psi$

Top: 3 - #4
Bot: 6 - #4

○ Positive Bending:

I) $A_s = 6(0.20) = 1.2\ in^2$

II) $a = \frac{A_s f_y}{0.85 f_c' b} = \frac{(1.2)(40)}{0.85(3)(51)}$

$a = 0.369089\ in$

III) $d = 8 - 1.25 - (\frac{1}{2})(0.5) = 6.5\ in$

IV) $\varepsilon_s = .0419 \geq \varepsilon_y = .0014$ (o.k.)

V) $M_n = A_s f_y (d - a/2)$

$= (1.2)(40)(6.5 - .369/2)$

$M_n = 303.142\ k\text{-}in = 25.26\ k\text{-}ft$

$\phi M_n = 0.9(25.26) = 22.74\ k\text{-}ft$

Positive Bending:
$M_n = 25.26\ k\text{-}ft$
$\phi M_n = 22.74\ k\text{-}ft$

○ Negative Bending:

I) $A_s = 3(0.20) = 0.6\ in^2$

II) $a = \frac{A_s f_y}{0.85 f_c' b} = \frac{(0.60)(40)}{0.85(3)(51)}$

$a = .184544\ in$

III) $d = 8 - 1.5 - (\frac{1}{2})(0.5) = 6.25\ in$

IV) $\varepsilon_s > \varepsilon_y$ (o.k.)

V) $M_n = A_s f_y (d - a/2)$

$= (0.6)(40)(6.25 - .1845/2)$

$M_n = 147.785\ k\text{-}in = 12.3155\ k\text{-}ft$

$\phi M_n = 0.9(12.3155) = 11.0839\ k\text{-}ft$

Negative Bending:
$M_n = 12.32\ k\text{-}ft$
$\phi M_n = 11.08\ k\text{-}ft$

○ Negative Shear:

I) $V_c = 2\lambda\sqrt{f_c'}\, b_w d$

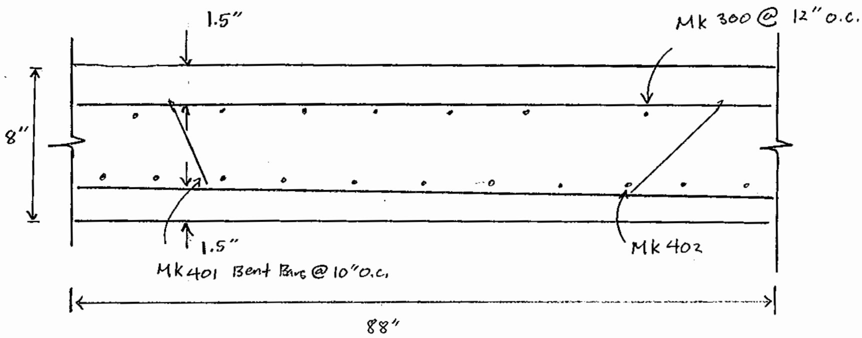
$= 2(1.0)(3000)^{1/2}(51)(6.25)$

$V_c = 34917.3\ lbs = 34.917\ kips$

II) $\phi V_c = 0.75(34.917) = 26.188\ kips$

Negative Shear:
$V_n = 34.92\ kips$
$\phi V_n = 26.19\ kips$

Deck Capacity: (Existing Section)



f_y = 40,000 psi
f_c' = 3000 psi

Top: 7 - #3
Bot: 11 - #4

o **Positive Bending:**

I.) $A_s = (11)(0.20) = 2.2 \text{ in}^2$

II.) $a = \dfrac{A_s f_y}{0.85 f_c' b} = \dfrac{(2.2)(40)}{0.85(3)(88)}$

$a = .392157$ in

III.) $d = 8 - 1.5 - (1/2)(1/2) = 6.25$ in

IV.) $\varepsilon_s = .0376 \geq \varepsilon_y = .0014$ (o.k.)

V.) $M_n = A_s f_y (d - a/2)$
$= (2.2)(40)(6.25 - .392157/2)$
$M_n = 532.745$ k-in $= 44.3954$ k-ft

$\phi M_n = 0.90(44.3954) = 39.956$ k-ft

$\boxed{\begin{array}{l}\text{Positive Bending:} \\ M_n = 44.40 \text{ k-ft} \\ \phi M_n = 39.96 \text{ k-ft}\end{array}}$

o **Negative Bending:**

I.) $A_s = (7)(0.11) = 0.77 \text{ in}^2$

II.) $a = \dfrac{A_s f_y}{0.85 f_c' b} = \dfrac{(0.77)(40)}{0.85(3)(88)}$

$a = 0.13726$ in

III.) $d = 6.25$ in

IV.) $\varepsilon_s \geq \varepsilon_y$ (o.k.)

V.) $M_n = A_s f_y (d - a/2)$
$= (0.77)(40)(6.25 - .13726/2)$
$M_n = 190.386$ k-in $= 15.866$ k-ft

$\phi M_n = 0.90(15.866) = 14.279$ k-ft

$\boxed{\begin{array}{l}\text{Negative Bending:} \\ M_n = 15.87 \text{ k-ft} \\ \phi M_n = 14.28 \text{ k-ft}\end{array}}$

o **Negative Shear:**

I.) $V_c = 2\lambda\sqrt{f_c'}\, b_w d$
$= 2(1.0)(3000)^{1/2}(88)(6.25)$
$V_c = 60249.5$ lbs $= 60.2495$ kips

II.) $V_s = \dfrac{A_v f_y d}{s}$

$= \dfrac{(2)(0.20)(40)(6.25)}{10}$

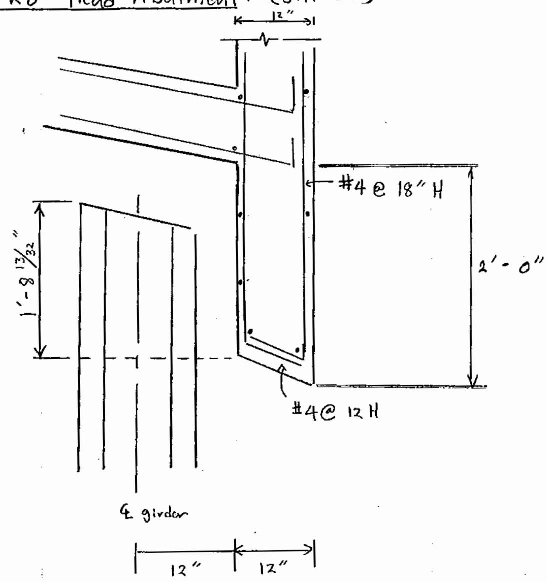
$V_s = 10$ kips

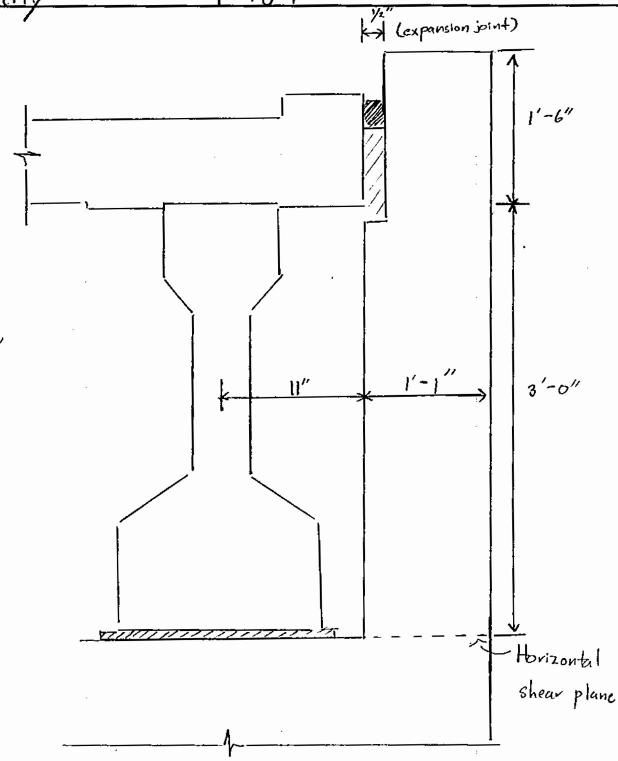
III.) $V_n = V_c + V_s = 70.2495$ kips

$\phi V_n = 0.75(70.2495) = 52.6871$ kips

$\boxed{\begin{array}{l}\text{Negative Shear:} \\ V_n = 70.25 \text{ kips} \\ \phi V_n = 52.69 \text{ kips}\end{array}}$

■ Koko Head Abutment: (Sht 30)





$f_y = 40{,}000$ psi

$f_c' = 3{,}000$ psi

○ Shear Horizontal Plane:

* Reinforcement will not provide shear resistance.

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

I.) $V_c = 2\lambda\sqrt{f_c'}\,bd$

$= 2(1.0)(3000)^{1/2}(12)(24)$

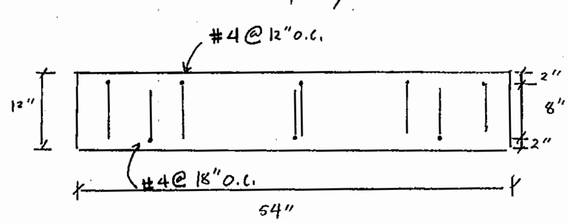
$V_c = 31548.8$ lbs

II.) $V_c = 31.5488$ kips

$\boxed{\Rightarrow V_n = 31.55 \text{ kips}}$

Horizontal Shear Plane

○ Vertical Flexure Capacity:



* Note: Only top reinforcement will provide flextural resistance.

I.) $A_s = (5)(\#4) = (5)(0.20) = 1.0\,in^2$

II.) $a = \dfrac{A_s f_y}{0.85 f_c' b} = \dfrac{(1.0)(40)}{0.85(3)(54)}$

$a = 0.290487$ in

III.) $d = 12 - 2 - \frac{1}{2}(0.5) = 9.75$ in

IV.) $\varepsilon_s = .0826 \geq \varepsilon_y = .0014$ (o.k.)

V.) $M_n = A_s f_y (d - a/2)$

$= (1.0)(40)(9.75 - .29/2)$

$M_n = 384.19$ k-in

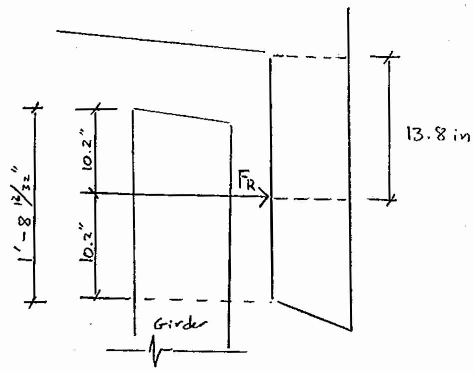
* Next page *

- **Koko Head Abutment: cont.**

  - Vertical Flextural Capacity: cont.

    VI.) $M_n = 384.19$ k-in

    VII.)



* ∴ Resultant force of the girder acts 13.8 in away from plane of flextural capacity

$M_n = F_R \times 13.8$

$F_R = M_n / 13.8 = (384.19 \text{ k-in}) / 13.8$

$F_R = 27.8462$ kips

$\boxed{\Rightarrow F_R = 27.85 \text{ kips}}$

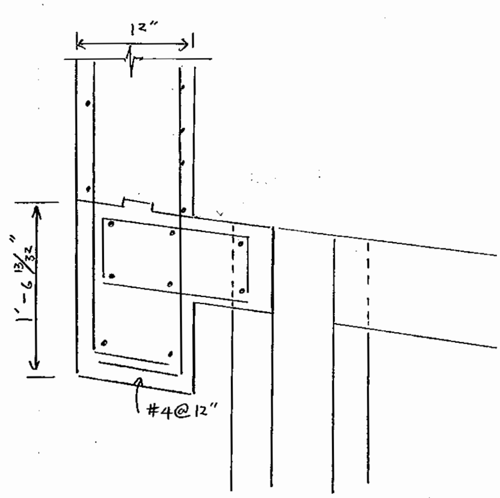
Flextural capacity

- Total Capacity: (Koko head Wing Wall)

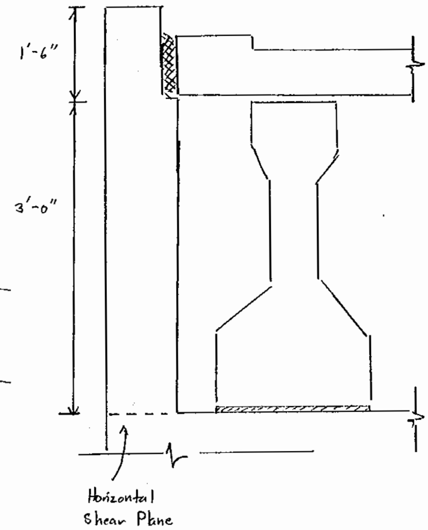
  - Horizontal Shear = 31.55 kips
  - Vertical Flexture = 27.85 kips

$\boxed{\text{Total} = 59.395 \text{ kips}}$

■ <u>Diamond Head Abutment</u>: (sht 30)



Girder



$f_y = 40{,}000$ psi
$f_c' = 3000$ psi

* Note: The most likely failure will be due to horizontal shear and vertical bending

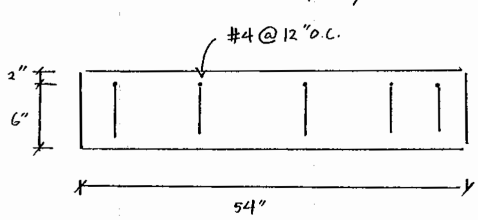
○ <u>Horizontal Shear Plane</u>:

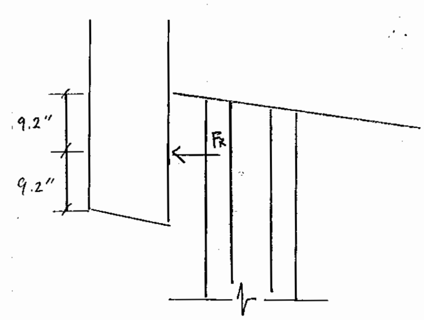
I.) $V_c = 2\lambda\sqrt{f_c'}\, b\, d$
$= 2(1.0)(3000)^{1/2}(12)(12+6.40625)$
$V_c = 24195.6$ lbs

II.) $V_c = 24.1956$ kips

$\boxed{\Rightarrow V_n = 24.20 \text{ kips}}$
Horizontal Shear Plane

○ <u>Vertical Flexture Capacity</u>:





* Note: only top reinforcement will provide flextural capacity

I.) $M_n = 384.19$ k-in
(same as koko head abutment)

II.) $F_R = M_n / 9.2$
$= 384.19 / 9.2$
$F_R = 41.7598$ kips

$\boxed{\Rightarrow F_R = 41.76 \text{ kips}}$
Flextural Capacity

- **Diamond Head Abutment: cont.**
  - Total Capacity: (Diamon Head Wing Wall)
    - Horizontal Shear = 24.20 kips
    - Vertical Flexture = 41.76 kips
    - $\boxed{\text{Total} = 65.9554 \text{ kips}}$

- **Both Wing Walls:**

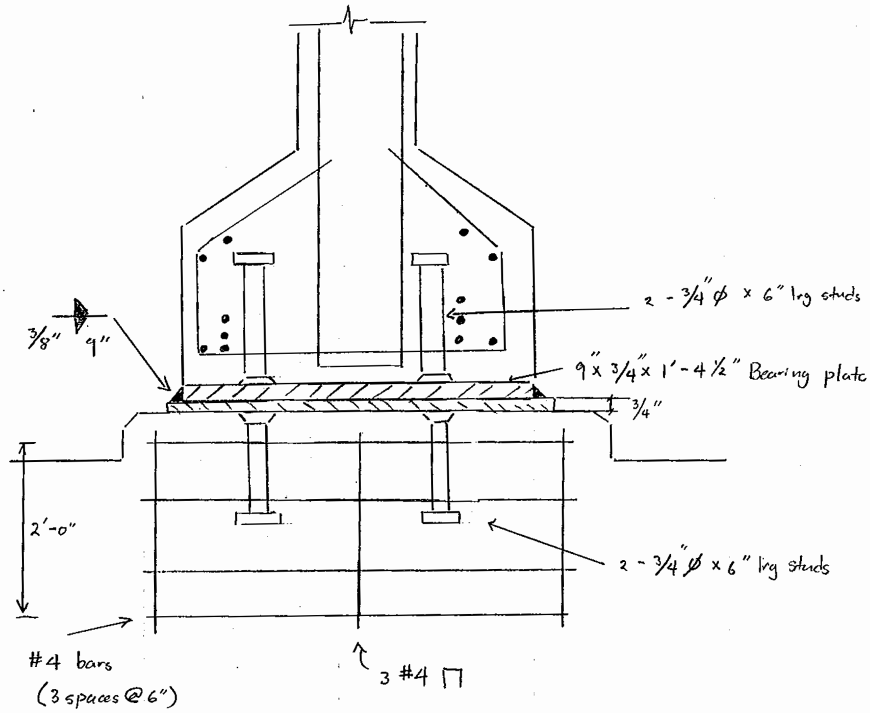
$$\text{Total Capacity} = \text{Koko} + \text{Diamond}$$
$$= 59.395 + 65.9554$$
$$\text{Total Capacity} = 125.35 \text{ kips}$$

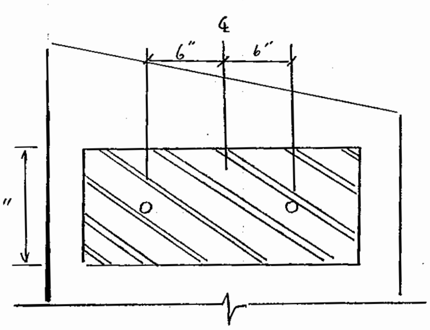
$$\boxed{\text{Total Lateral Capacity} = 125.35 \text{ kips}}$$

Wing Walls

National® Brand 42-182 100 SHEETS

- Girder Seat Bearing Pad: (Sheet 30 & 32)





Properties: (Based on year of plans)

Studs: $F_y = 40$ ksi $F_u = 65$ ksi

Steel: $F_y = 50$ ksi

Welds: Assume E70

* Note: The studs 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)

- **Bearing Lateral Capacity:**

  - Stud Shear Strength: (2 - 3/4" ϕ)

    $f_c' = 6000$ psi girder

    $f_c' = 3000$ psi bottom

  - For Girder:

    I.) $Q_n = 0.5\, A_{sc} \sqrt{f_c' E_c} \le R_g R_p A_{sc} F_u$

    II.) $A_{sc} = (\pi/4)(3/4)^2 = 0.441786 in^2$

    III.) $E_c = w_c^{1.5} \sqrt{f_c'}$

    $= (150)^{1.5}(6)^{1/2}$

    $E_c = 4500$ ksi

    IV.) $Q_n = 0.5\, A_{sc} \sqrt{f_c' E_c}$

    $= 0.5(.441786)(6(4500))^{1/2}$

    $Q_n = 36.2965$ kips

    V.) $R_g = 0.85$ (two studs)

    $R_p = 1.0$

    $\Rightarrow R_g R_p A_{sc} F_u$

    $= (0.85)(1.0)(.442)(65)$

    $= 24.4087$ kips $\le 0.5 A_{sc} \sqrt{f_c' E_c}$

    $\therefore Q_n = 24.4087$ kips

    VI.) For two studs:

    $\boxed{\Rightarrow Q_n = 48.82 \text{ kips}}$

    Studs in girder

  - For bottom:

    I.) $E_c = w_c^{1.5} \sqrt{f_c'}$

    $= (150)^{1.5}(3)^{1/2}$

    $E_c = 3181.98$ ksi

    II.) $Q_n = 0.5\, A_{sc} \sqrt{f_c' E_c}$

    $= 0.5(0.441786)(3 \times 3181.98)^{1/2}$

    $Q_n = 21.582$ kips $\le R_g R_p A_{sc} F_u$

    $\therefore Q_n = 21.582$ kips

    III.) For two studs:

    $\boxed{\Rightarrow Q_n = 43.164 \text{ kips}}$

    Studs in bottom concrete

  - Studs in bottom concrete Controls:

    $\boxed{\therefore Q_n = 43.164 \text{ kips/bearing}}$

8 bearings at Diamond Head Abutment Only

- Bearing Lateral Capacity: cont.

  - Weld Strength:
    Weld: $6/16''$
    E70
    $18''$ length total

    I.) $\phi R_n = \phi\,(0.707\,w\,(0.6\,F_{exx}))$
    $= 0.75\,(0.707(6/16)(0.6(70)))$
    $\phi R_n = 8.35144$ kips/in

    II.) For entire length:
    $\phi R_n = (8.35144)(18) = 150.326$ kips

    $\boxed{\therefore\ R_n = 200.435 \text{ kips}}$
    Weld Strength

- Base Metal Shear Strength: (along weld)

  I.) $\phi R_n = \min[1.0(0.6 F_y t)\ ;\ 0.75(0.6 F_u t)]$

  *Note: Both plates have the same thickness

  II.) $1.0(0.6 F_y t) = 1.0(0.6)(50)(3/4)$
  $= 22.5$ kips/in

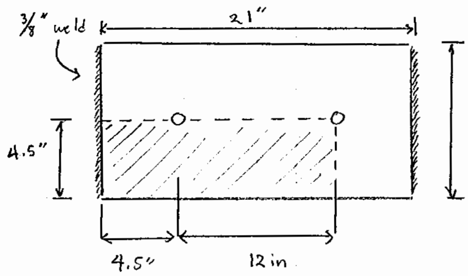
  III.) $0.75\,(0.6\,F_u\,t) = 0.75\,(0.6)(65)(3/4)$
  $= 21.94$ kips/in $\therefore$ controls

  IV.) $\phi R_n = (21.9)(18) = 394.2$ kips (for entire length)

  $\boxed{\therefore\ R_n = 394 \text{ kips}}$
  Base Metal Shear Strength (Along Welds)

Bearing Lateral Capacity :

o Bolt Block Shear :



Studs : $3/4'' \phi$
Steel : $F_y = 50$ ksi
$F_u = 65$ ksi
$t = 3/4''$

I.) $R_n = 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \le 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$

II.) $A_{gv} = (3/4)(4.5+12)(1.0) = 12.375 \text{ in}^2$

III.) $A_{nv} = (3/4)[(16.5) - (1.5)(3/4)](1.0) = 11.5313 \text{ in}^2$

IV.) $A_{nt} = (3/4)[(4.5) - (1/2)(3/4)](1.0) = 3.09375 \text{ in}^2$

V.) $R_n = 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \le 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$
$= 0.6(65)(11.5313) + (1.0)(65)(3.09375) \le 0.6(50)(12.375) + 1.0(65)(3.093)$
$R_n = 650.813$ kips ≤ 572.344 kips (controls)

$$\boxed{\therefore \ R_n = 572.34 \text{ kips}}$$

Bolt Block Shear Capacity

o Weld Block Shear :

I.) $A_{gv} = A_{nv} = 0$ (No longitudinal welds)

II.) $A_{nt} = t_{member}(18)$
$= (3/4)(18) = 13.5 \text{ in}^2$

III.) $R_n = U_{bs} F_u A_{nt}$
$= (1.0)(65)(13.5)$
$R_n = 877.5$ kips

$$\boxed{\therefore \ R_n = 877.5 \text{ kips}}$$

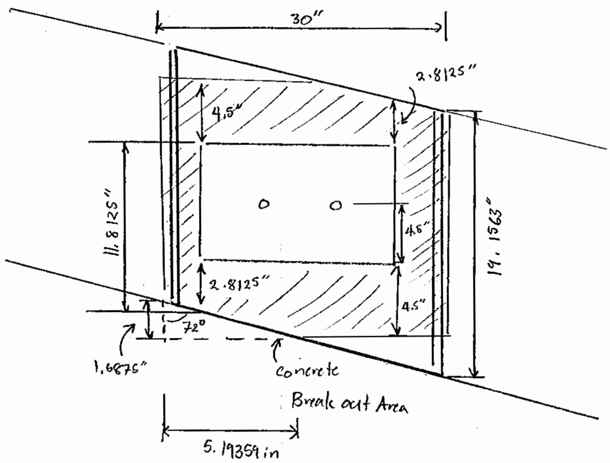
Weld Block Shear

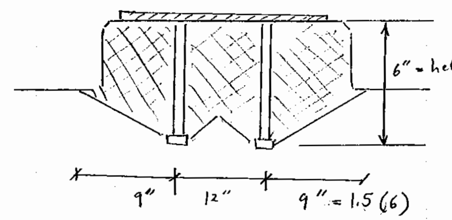
■ Vertical Resistance:

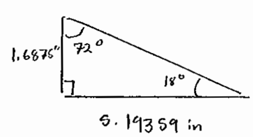
o Concrete Break Out Strength of Anchor in Tension: (ACI Section D.5.2)

- Bottom Concrete:

$f_c' = 3000$ psi







$$\frac{\sin(18)}{1.6875} = \frac{\sin(72)}{x}$$

$x = 5.19359$ in

I.) $A_{nc} = (30)(2.8125 + 9 + 4.5) - 2[(\tfrac{1}{2})(1.69)(5.2)] = 480.61 \text{ in}^2$

II.) $A_{nco} = 9\, h_{ef}^2 = 9(6)^2 = 324 \text{ in}^2$

III.) $\psi_{ec,N} = 1.0$ (Not loaded eccentrically)

IV.) $C_{a2\,min} = (4.5 + 2.8125) = 7.3125$ in

$\therefore\ C_{a2\,min} \leq h_{ef}(1.5)$

$$\psi_{ed,N} = 0.7 + 0.3\,\frac{C_{a2\,min}}{1.5\,h_{ef}}$$

$$= 0.7 + 0.3\,(7.3125/9)$$

$\psi_{ed,N} = 0.94375$

V.) $\psi_{c,N} = 1.25$ (Cast in anchors)

VI.) $\psi_{cp,N} = 1.0$

VII.) $N_b = K_c\,\lambda\,\sqrt{f_c'}\,h_{ef}^{1.5}$

$= (24)(1.0)(3000)^{1/2}(6)^{1.5}$

$N_b = 19319.61\text{bs} = 19.3196$ kips

⇒ For two anchors:

$N_b = 38.6393$ kips

VIII.) $N_{cb} = \frac{A_{nc}}{A_{nco}}\,\psi_{ec},\,\psi_{ed},\,\psi_c,\,\psi_{cp},\,N_b$

$= (480.6/324)(1.0)(.94375)(1.25)(1.0) \times (38.6393)$

$N_{cb} = 67.6152$ kips

$\boxed{\therefore\ N_{cb} = 67.62 \text{ kips}}$

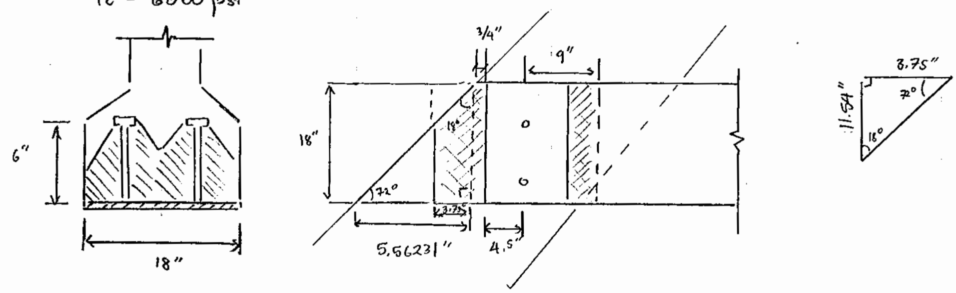
Concrete Break Out Strength
Bottom Concrete

■ Vertical Resistance: Cont.

o Concrete Break Out Strength of Anchor in Tension: (ACI Section D.5.2)

- Girder:

$f_c' = 6000$ psi



I.) $A_{NC} = (18)(18) - \frac{1}{2}(11.5413)(3.75) = 302.36\,in^2$

II.) $A_{NCO} = 9\,h_{ef}^2 = 9(6)^2 = 324\,in^2$

III.) $\psi_{ec,N} = 1.0$

IV.) $C_{a2\,min} = 4.5$ in $\leq h_{ef} = 1.5(6.0) = 9$ in

$$\psi_{ed} = 0.7 + 0.3(C_{a2\,min}/h_{ef}(1.5))$$
$$= 0.7 + 0.3(4.5/9)$$
$$\psi_{ed} = .85$$

V.) $\psi_{c,N} = 1.25$ (cast in anchors)

VI.) $\psi_{cp,N} = 1.0$

VII.) $N_b = K_c \lambda \sqrt{f_c'}\, h_{ef}^{1.5}$

$= (24)(1.0)(6000)^{1/2}(6)^{1.5}$

$N_b = 27322.1$ lbs

$N_b = 27.3221$ kips ⇒ For two bolts $= 54.6442$ kips

VIII.) $N_{cb} = (A_{NC}/A_{NCO})\,\psi_{ec}, \psi_{ed}, \psi_c, \psi_{cp}, N_b$

$= (302.36/324)(1.0)(.85)(1.25)(1.0)(54.6442)$

$N_{cb} = 54.1816$ kips

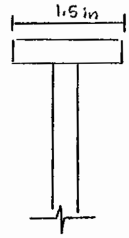
$\boxed{\therefore\ N_{cb} = 54.1816 \text{ kips}}$

Concrete Break Out Strength
Girder

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

■ <u>Vertical Resistance</u>: cont.

○ <u>Pull Out Strength of Anchor in Tension</u>: (ACI Section D.5.3)



≈ estimated

I.) $N_{pn} = \psi_{c,p} N_p$

II.) $A_{brg} = (\pi/4)(1.5)^2 = 1.767 \text{ in}^2$

⇒ For two studs $= 3.534 \text{ in}^2$

III.) $N_p = 8 A_{brg} f_c'$

- <u>Girder</u>:

  $N_p = 8(3.534)(6000) = 169646 \text{ lbs} = 169.65 \text{ kips}$

- <u>Bot</u>:

  $N_p = 8(3.534)(3) = 84.823 \text{ kips}$

IV.) Assume cracked:

$\psi_{c,p} = 1.0$

◎ <u>Girder</u>:

$N_{pn} = 169.65 \text{ kips}$

◎ <u>Bottom</u>:

$N_p = 84.82 \text{ kips}$

- Vertical Resistance: cont.

  - Steel Strength of anchor in tension: (ACI Section D.5.1)

    diameter = 3/4"

    $f_y = 40$ ksi

    $f_u = 65$ ksi

    I.) $N_{sa} = n\,A_{se,N}\,f_{uta}$

    II.) $A_{se,N} = (\pi/4)(3/4)^2 = .441786\ in^2$

    III.) $N_{sa} = n\,A_{se,N}\,f_{uta}$

    $= (2)(.441786)(65)$

    $N_{SA} = 57.4322$ kips

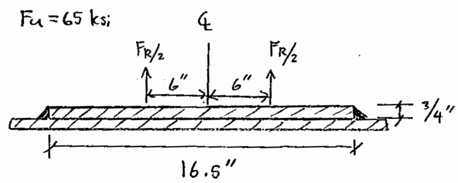
    $\boxed{\therefore\ N_{SA} = 57.43\ \text{kips}}$

    Tensile Capacity of studs

  - Steel Plate Bending/Flextural Capacity:

    $F_y = 50$ ksi

    $F_u = 65$ ksi



* Note: The top steel plate is analyzed as a thin simply supported beam.

I.) $M_{max} = F_R/2\,(16.5/2) - F_R/2\,(6)$

$M_{max} = 1.125\,F_R$

II.) From beam theory:

$$M = \frac{\sigma I}{y}$$

III.) $I = 1/12\,bh^3 = 1/12\,(16.5)(3/4)^3$

$I = 0.580078\ in^4$

IV.) $y = h/2 = (3/8) = .375$ in

V.) $M = \frac{\sigma I}{y} = \frac{F_u I}{y} = \frac{(65)(.58)}{.375}$

$M = 100.547$ k-in

VI.) $F_R = M/1.125 = 100.547/1.125$

$F_R = 89.3751$ kips

VII.) For both plates:

$F_R = (2)(89.3751) = 178.75$ kips

$\boxed{\therefore\ F_R = 178.75\ \text{kips}}$

Plate Flextural Capacity (Both plates)

- **Summary Lateral Capacity:**
  - For one bearing: (8 total at Diamond Head Abutment only)
    - Stud Shear Strength = 43.164 kips ∴ controls
    - Weld Strength = 200.435 kips
    - Base Metal Strength (at welds) = 394 kips
    - Bolt Block Shear = 572.34 kips
    - Weld Block Shear = 877.5 kips

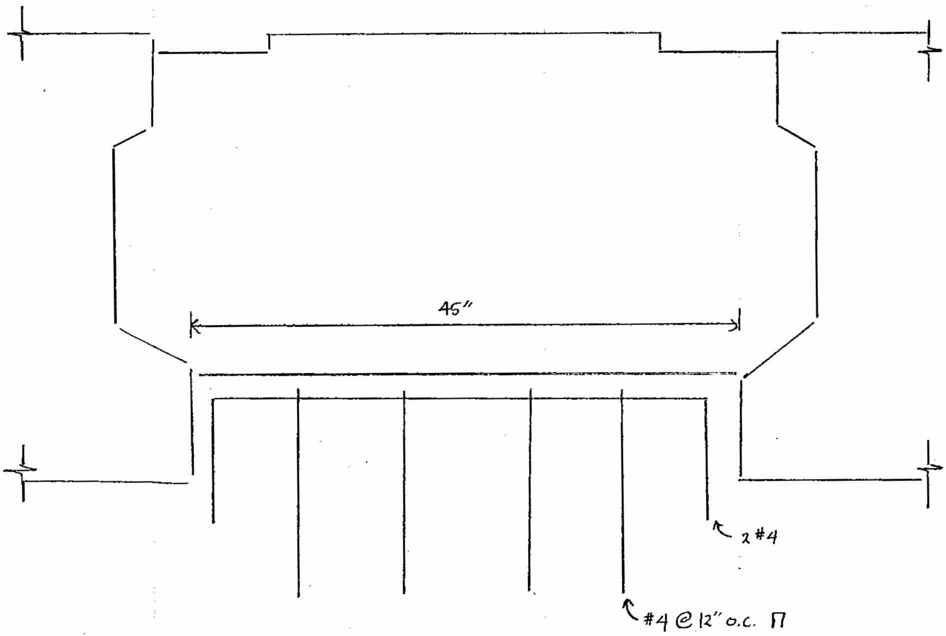
* Note: Concrete pryout or punch out was not considered because of proper reinforcement in the area of the studs, will provide adequate strength to reduce those types of failures.

> ⇒ Total Lateral Capacity Provided:
>
> $H_R = 8(43.164) = 345.312$ kips
>
> (From Bearing Pads)

- **Summary Vertical Capacity:**
  - For one bearing:
    - Concrete Break Out Tension (Bot Conc.) = 67.62 kips
    - Concrete Break Out Tension (Girder) = 54.1816 kips ∴ controls
    - Pull Out Strength of Achor:
      - Girder = 169.65 kips
      - Bottom = 84.82 kips
    - Steel Strength of Achor In Tension = 57.4322 kips
    - Steel plates flexture capacity = 178.75 kips

> ⇒ Total Vertical Capacity Provided:
>
> $V_R = 8(54.1816) = 433.453$ kips
>
> (From Bearing Pads)

■ Creep Block Capacity:



$f_y = 40{,}000$ psi
$f_c' = 3000$ psi

o Shear Friction: (ACI Section 11.6.4)

I) $V_n = A_{vf} f_y \mu$

II) $A_{vf} = 2(\#4 \sqcap) + 4(\#4 \sqcap)$
$= 2(2 \times 0.20) + 4(2 \times 0.20)$
$A_{vf} = 2.4 \text{ in}^2$

III) $\mu = 1.4\lambda = 1.4$ (creep block placed monolithically)

IV) $V_n = A_{vf} f_y \mu$
$= (2.4)(40)(1.4)$
$V_n = 134.4$ kips

$\boxed{\therefore V_n = 134.4 \text{ kips}}$
Creep Block Capacity
(2 at Koko Head abutment only)

V) Check:

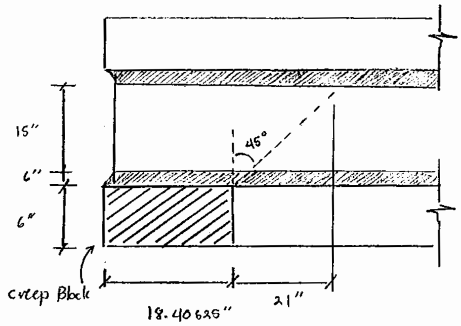
① $0.2 f_c' A_c = 0.2(3)(45)(12 + 6 + 13/32) = 496.97$ kips $> V_n$ (o.k.)

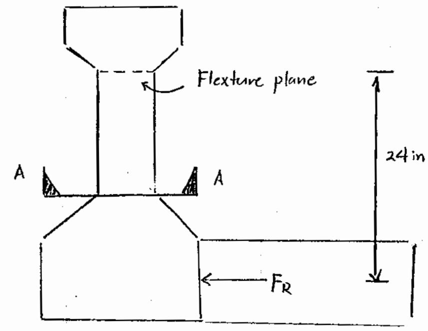
② $(480 + 0.08 f_c') A_c = (480 + (0.08)(3000))(45)(18.40625) = 596363$ lbs $> V_n$ (o.k.)

③ $1600 A_c = 1600(45)(18.40625) = 1.3253 \times 10^6$ lbs $> V_n$ (ok.)

Beam Capacity:

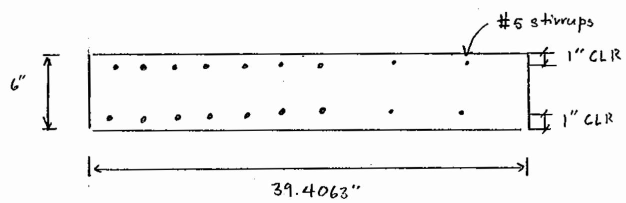
* Note: From experience from the Maipalaoa Stream Bridge the girder web capacity is controled by flexture.





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

- Web Cross Section: (A - A)



$F_y = 40{,}000\,psi$
$f_c' = 6000\,psi$

*Note: only top reinforcement will provide flexture resistance

I) $A_s = (9)(0.31) = 2.79\ in^2$

II) $a = \dfrac{A_s f_y}{0.85 f_c' b} = \dfrac{(2.79)(40)}{0.85(6)(39.4063)}$

$a = 0.5553\ in$

III) $d = 6 - 1 - (\tfrac{1}{2})(0.625) = 4.6875\,in$

IV) $\beta_1 = 0.85 - 0.05\left\{\dfrac{f_c' - 4000}{1000}\right\}$

$= 0.85 - 0.05\left\{\dfrac{(6000) - 4000}{1000}\right\}$

$\beta_1 = 0.75$

V) $\varepsilon_s = 0.003\left\{\dfrac{d - a/\beta_1}{a/\beta_1}\right\}$

$\varepsilon_s = 0.016 \geq \varepsilon_y = .0014$ (o.k.)

VI) $M_n = A_s f_y (d - a/2)$

$= (2.79)(40)(4.6875 - .5553/2)$

$M_n = 492.139\ k\text{-}in$

VII) $M_{max} = 24\,F_R$

$F_R = \dfrac{M_n}{24} = \dfrac{492.139}{24}$

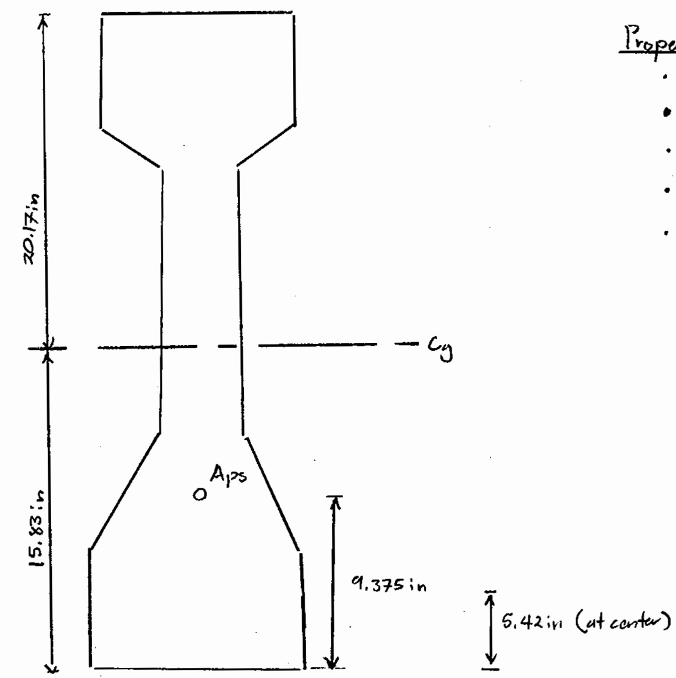
$F_R = 20.506$ kips ∴ controls

Beam Web Flextural Capacity:
$F_R = 20.506$ kips

2 Creep Blocks at Diamond Head Abutment only

⇒ For Both Creep Blocks:
$F_R = 41.01$ kips

Negative Bending Capacity:



Properties:

- $f_c' = 6000 psi$
- $P_e = (18.9\times10^3)(0.7)(24) = 317520$ kips
- $A_{ps} = 24 \times .153 = 3.672\ in^2$
- $f_t = 929.516$ psi
- $f_c = -5100$ psi

○ Compute Geometric Properties: (see Structural analysis pg.1 for areas)

- Centroid:
  - $A_1 = 72 in^2$, $y_1 = 33 in$
  - $A_2 = 9 in^2$, $y_2 = 29 in$
  - $A_3 = 144 in^2$, $y_3 = 18 in$
  - $A_4 = 36 in^2$, $y_4 = 8 in$
  - $A_5 = 108 in^2$, $y_5 = 3 in$

$$\bar{y} = \frac{\Sigma A_i y_i}{\Sigma A_i} = 15.829 \text{ in}$$

$y_b = 15.83$ in
$y_t = 20.17$ in

- Moment of inertia:
  - $I_1 = 1/12\ bh^3 = 1/12 (12)(6)^3$
    $I_1 = 216 in^4$
    $\Rightarrow I + Ad^2 = 216 + (72)(17.17)^2 = 21442.2 in^4$
  - $I_2 = bh^3/36 = (3)^4/36 = 2.25 in^4$
    $\Rightarrow I + Ad^2 = 2.25 + 4.5(13.17)^2 = 782.77 in^4$
  - $I_3 = 1/12\ bh^3 = 1/12 (6)(24)^3$
    $I_3 = 6912 in^4$
    $\Rightarrow I + Ad^2 = 6912 + (144)(2.17)^2 = 7590.08 in^4$
  - $I_4 = bh^3/36 = 6^4/36 = 36 in^4$
    $\Rightarrow I + Ad^2 = 36 + 36(7.83)^2 = 2243.12 in^4$
  - $I_5 = 1/12\ bh^3 = 1/12 (18)(6)^3 = 324 in^4$
    $\Rightarrow I + Ad^2 = 324 + (108)(12.83)^2 = 18101.8 in^4$

$I_c = 21442.2 + (2)(782.77) + 7590.08 + (2)(2243.12) + 18101.8$

$I_c = 53185.9 in^4$

- **Negative Bending Capacity:**
  - Geometric Properties

$A_c = 369 \text{ in}^2$
$I_c = 53185.9 \text{ in}^4$
$C_t = 20.17 \text{ in}$ $\quad S_t = 2636.88 \text{ in}^3$
$C_b = 15.83 \text{ in}$ $\quad S_b = 3359.82 \text{ in}^3$
$r^2 = 144.14 \text{ in}^2$
$e_c = 10.41 \text{ in}$
$L = 580.68 \text{ in}$

I) w_D:

Self weight $= (2.5625)(150) = 384.375 \text{ lb/ft}$
Topping $= (8/12)(5 + 3/12)(150) = 525 \text{ lb/ft}$

$\Rightarrow w_D = 384.375 + 525 = 909.375 \text{ lb/ft}$
$w_D = 75.781 \text{ lb/in}$

II) Tensile Limit: (unsubmerged)

$$W_{wt} = \frac{8}{L^2}\left(M_D + S_t f_t - \left\{\frac{S_t P_e}{A_c}\right\}\left(\frac{e C_t}{r^2} - 1\right)\right)$$

$$M_D = \frac{w_D L^2}{8} = \frac{(75.781)(580.68)^2}{8}$$
$$M_D = 3.19408 \times 10^6 \text{ lb-in}$$

$$\Rightarrow W_{wt} = \frac{8}{(580.68)^2}\left(3.19408\times10^6 + (2636.88)(929.516) - \frac{(2636.88)(317520)}{369}\left\{\frac{10.41 \cdot 20.17}{144.14} - 1\right\}\right)$$

$W_{wt} = 109.347 \text{ lb/in}$

$F_w = 63.49$ kips (Tensile limit)

III) Compression Limit: (unsubmerged)

$$W_{wc} = \frac{8}{L^2}\left(M_D - S_b f_c - \frac{S_b P_e}{A_c}\left(1 + \frac{e C_b}{r^2}\right)\right)$$
$$= \frac{8}{(580.68)^2}\left(3.19408\times10^6 - (3359.82)(-5100) - \frac{(3359.82)(317520)}{369}\left\{1 + \frac{(10.41)(15.83)}{144.14}\right\}\right)$$

$W_{wc} = 335.308 \text{ lb/ft}$

$F_w = 194.71$ kips (compression limit)

■ Negative Bending Capacity:

IV.) Submerged:

* Assume 50% air pocket

$w_D = 909.375 - 6.0625(64) - 5.9(64) = 143.775$ lb/ft

$w_D = 11.9813$ lb/in

$\Rightarrow M_D = 504{,}994$ lb-in

V.) Tensile Limit:

$w_{wt} = \frac{8}{L^2}[M_D + 1.41475\times10^6]$

$w_{wt} = 45.5471$ lb/in

$F_w = 26.448$ kips (tension)

VI.) Compression:

$w_{wc} = \frac{8}{L^2}[M_D + 1.09387\times10^7]$

$w_{wc} = 271.508$ lb/in

$F_w = 157.66$ kips (compression)

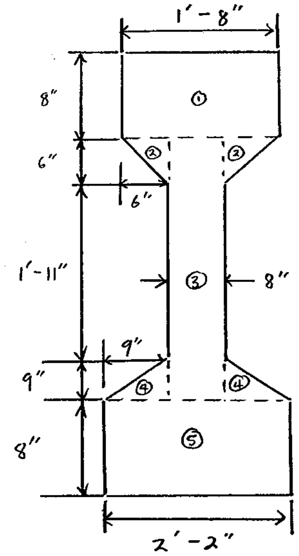
■ Summary of Results:

- Unsubmerged:
  - Tensile $F_w = 63.49$ kips
  - Compression $F_w = 194.71$ kips
- Submerged:
  - Tensile $F_w = 26.448$ kips
  - Compression $F_w = 157.66$ kips
- Loss:
  - T = 58.34%
  - C = 19.03%

- **Buoyancy Calculations:**

Calculations By: Daniel Lum

- Typical Girder Section: (Sheet 17)



$A_1 = (1 + 8/12)(8/12) = 1.111\ ft^2$

$A_2 = 2[\tfrac{1}{2}(6/12)(6/12)] = 0.25\ ft^2$

$A_3 = (8/12)(6/12 + 1 + 11/12 + 9/12) = 2.111\ ft^2$

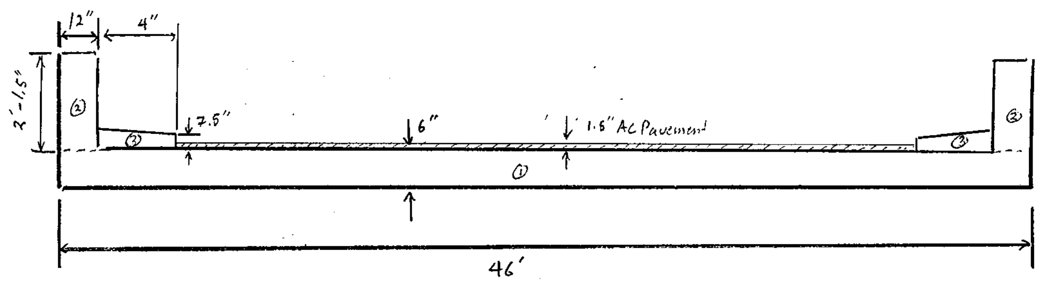
$A_4 = 2[\tfrac{1}{2}(9/12)(9/12)] = 0.5625\ ft^2$

$A_5 = (2 + 2/12)(8/12) = 1.444\ ft^2$

- $A_T = \sum_{i=1}^{5} A_i = 5.47917\ ft^2$

$\boxed{A_T = 5.48\ ft^2}$ Girder Section

- Deck Section: (sheet 6)



- Concrete:

$A_1 = (46)(6/12) = 23\ ft^2$

$A_2 = 2(12/12)(2 + 1.5/12) = 4.25\ ft^2$

$A_3 = 2[(4)(7.5/12)] = 5\ ft^2$

- $A_T = \sum_{i=1}^{3} A_i = 32.25\ ft^2$

$\boxed{A_T = 32.25\ ft^2}$ Concrete Deck

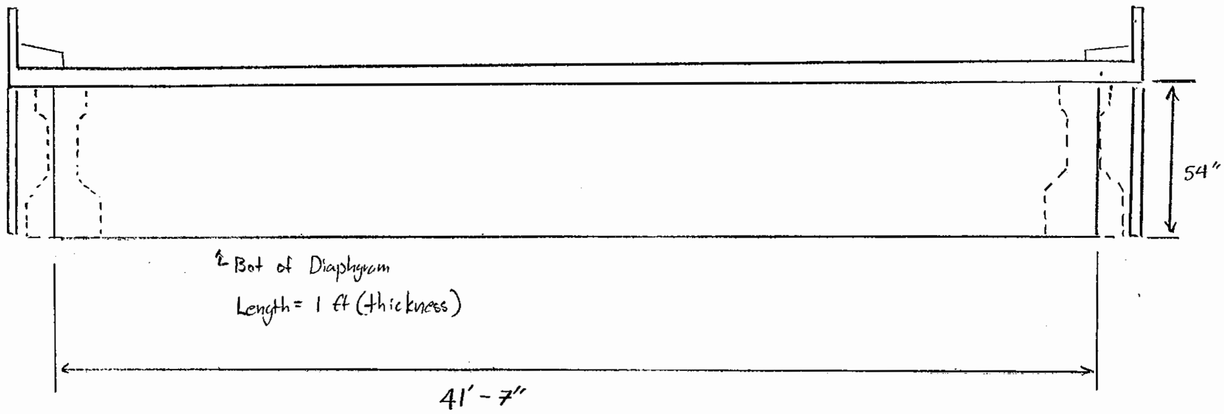
- AC Pavement:

$A_T = (36)(1.5/12) = 4.5\ ft^2$

$\boxed{A_T = 4.5\ ft^2}$ AC Pavement

▲ Concrete Diaphgram:

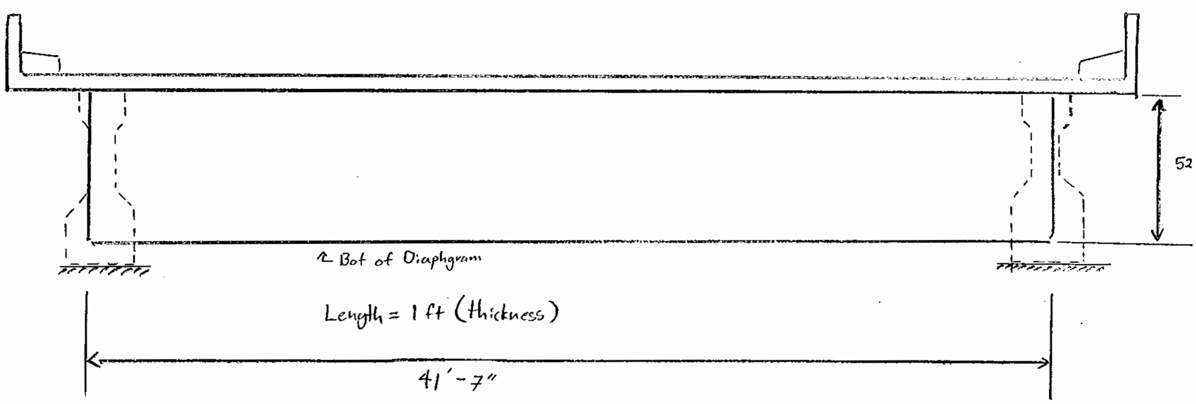
○ Section: Thru Abutment Diaphgram (sheet 14)



$$Area = (41 + 7/12)(54/12) = 187.125 \ ft^2$$

$\boxed{A_T = 187.13 \ ft^2}$ Thru Abutment Diaphgram

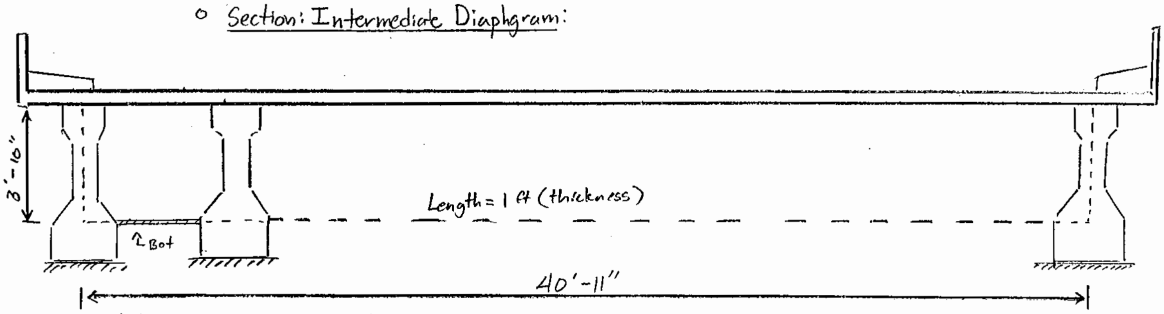
○ Section: Thru Pier Cap Diaphgram:



$$Area = (41 + 7/12)(52/12) = 180.194 \ ft^2$$

$\boxed{A_T = 180.19 \ ft^2}$ Thru Pier Cap Diaphgram

○ Section: Intermediate Diaphgram:



* Diaphgram occupies area between girders

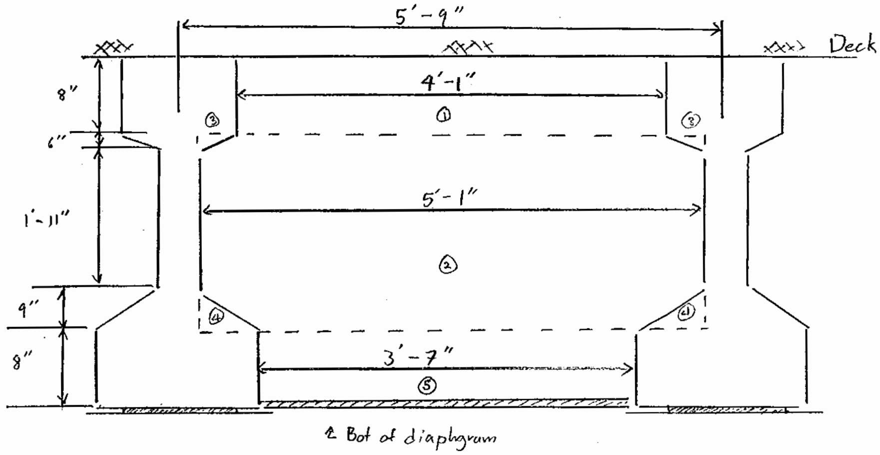
$$\text{Area of girders} = 7(1.111 + 0.25 + 2.111 + 0.5625) = 28.243 \ ft^2$$

$$Area = (40 + 11/12)(3 + 10/12) - 28.243 = 128.604 \ ft^2$$

$\boxed{A_T = 128.60 \ ft^2}$ Intermediate Diaphgram

▲ Air Pocket Calculations:

* Note: Volume of trapped air is dependent on Thru Abutment Diaphgram, as it has the deepest depth.



$$A_1 = (4 + 1/12)(8/12) = 2.722 \text{ ft}^2$$

$$A_2 = (5 + 1/12)(3 + 2/12) = 16.0972 \text{ ft}^2$$

$$A_3 = 2\left[\tfrac{1}{2}(6/12)(6/12)\right] = 0.25 \text{ ft}^2$$

$$A_4 = 2\left[\tfrac{1}{2}(9/12)(9/12)\right] = 0.5625 \text{ ft}^2$$

$$A_5 = (3 + 7/12)(8/12) = 2.3889 \text{ ft}^2$$

- $A_T = A_1 + A_2 - A_3 - A_4 + A_5$
  $= (2.72) + (16.09) - (0.25) - (0.5625) + (2.389)$
  $A_T = 20.3958 \text{ ft}^2$

▲ Reduction of Air Pocket:

* Assume the bridge is submerged to top of deck

- $h = \underbrace{(1.5/12)}_{\text{AC Pave}} + \underbrace{(6/12)}_{\text{Deck}} + \underbrace{(54/12)}_{\text{to bot of diaphgram}} = 5.125 \text{ ft}$

- $P_2 = 14.7 + h(64/144)$
  $= 14.7 + (5.125)(64/144)$
  $P_2 = 16.9778 \text{ psi}$

- $A_2 = \dfrac{P_1 A_1}{P_2} = \dfrac{(14.7)(20.3958)}{16.9778}$
  $A_2 = 17.6595 \text{ ft}^2$

$\boxed{A_T = 17.66 \text{ ft}^2}$ Compressed Air Pocket

▲ Collection of Data:

■ Concrete: ($\gamma_{RC} = 150\ ^{lb}/_{ft^3}$)

○ Typical Girder:
Area = $5.48\ ft^2$
Amount = 8
Length = 318 ft

○ Thru Abutment Diaphgram:
Area = $187.13\ ft^2$
Amount = 2
Length = 1 ft

○ Deck:
Area = $32.25\ ft^2$
Amount = 1
Length = 318 ft

○ Thru Pier Cap Diaphgram:
Area = $180.19\ ft^2$
Amount = 2
Length = 1 ft

○ Intermediate Diaphgram:
Area = $128.60\ ft^2$
Amount = 6
Length = 1 ft

■ AC Pavement: ($\gamma_{AC} = 152\ ^{lb}/_{ft^3}$)
Area = $4.5\ ft^2$
Amount = 1
Length = 318 ft

■ Air Pocket: ($\gamma_{sea\ water} = 64\ ^{lb}/_{ft^3}$)
Area = $17.66\ ft^2$
Amount = 7
Length = 310 ft

▲ Total Self weight:

○ Self Weight = $[(7)(\text{Girder})(L) + (1)(\text{Deck})(L) + (2)(\text{Abut Dia})(L) + (2)(\text{Pier Dia})(L) + (6)(\text{Int. Dia.})(L)]\ \gamma_{RC} + (1)(AC)(L)\ \gamma_{AC}$

$= [(7)(5.48)(318) + (32.25)(318) + 2(187.13) + 2(180.19) + (6)(128.60)]\ 150 + (1)(4.5)(318)(152)$

$\boxed{\text{Self Weight} = 3.81155 \times 10^6\ lbs}$

▲ Buoyant Force:

○ Submerged Volume = Air Pocket + Submerged concrete (*Assume submerged to top of deck*)
$= 7(17.66)(310) + (23960.2 - \underbrace{9.25(318)}_{\text{Railing}} + 1431)$

$S\forall = 60770.8\ ft^3$

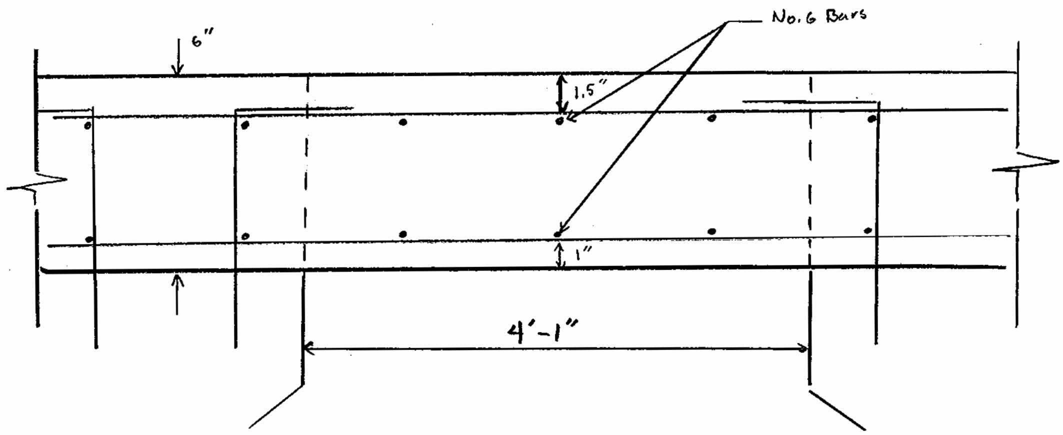
○ Buoyant Force = $S\forall \times \gamma_{sw}$
$= (60770.8)(64)$
$\boxed{BF = 3.88933 \times 10^6\ lbs}$

▲ Summary:

$\boxed{\begin{array}{l} \text{Self Weight} = 3812\ \text{kips} \\ \text{Buoyant Force} = 3889\ \text{kips} \\ \text{Residual Weight} = -77.3\ \text{kips} \end{array}}$ (-2.0% of original self weight)

∴ Bridge is Buoyant

- Deck Moment Capacity:



$f_c' = 4000$ psi
$f_y = 60{,}000$ psi
$b = 49$ in

No. 6 Bar:
$A_s = 0.44$ in²
$d_b = 0.750$ in

- Positive Bending Capacity:
  - $d = 6 - 1 - 0.750/2$
    $= 4.625$ in
  - $A_s = 3(0.44)$
    $= 1.32$ in²
  - $T = A_s f_y$ — assuming $\varepsilon_s \geq \varepsilon_y$
    $= (1.32)(60)$
    $T = 79.2$ kips
  - $a = \dfrac{A_s f_y}{0.85 f_c' b} = \dfrac{(79.2)}{0.85(4)(49)}$
    $a = 0.4754$ in
  - $f_c' \leq 4000$ psi
    $\Rightarrow \beta_1 = 0.85$

    $c = a/\beta_1 = 0.5593$ in
  - $\varepsilon_s = \left(\dfrac{d - c}{c}\right)\varepsilon_{cu}$
    $= \left(\dfrac{4.625 - 0.56}{0.56}\right)0.003$
    $\varepsilon_s = .0218$
  - $\varepsilon_y = \dfrac{f_y}{E_s} = \dfrac{60}{29{,}000} = .00207$

    $\therefore \varepsilon_s \geq \varepsilon_y$ o.k.
  - $M_n = A_s f_y (d - a/2)$
    $= (79.2)(4.625 - 0.4754/2)$
    $M_n = 347.474$ k-in

    $M_n = 28.96$ k-ft
  - Flexure Reduction Factor = 0.90

    $\Rightarrow \phi M_n = 0.90(28.96)$

    $\boxed{\phi M_n = 26.06 \text{ k-ft}}$ Typical Span Positive Bending Capacity

- Negative Bending Capacity:
  - $d = 6 - 1.5 - 0.75/2 = 4.125$ in
  - $\varepsilon_s = \left(\dfrac{d - c}{c}\right)\varepsilon_{cu} = \dfrac{(4.125 - 0.56)}{0.56}(0.003)$

    $\varepsilon_s = .0191 \geq \varepsilon_y$ o.k.
  - $M_n = A_s f_y (d - a/2)$
    $= (79.2)(4.125 - 0.4754/2)$
    $M_n = 307.874$ k-in $= 25.65$ k-ft
  - $\phi M_n = 0.90(25.65)$

    $\boxed{\phi M_n = 23.09 \text{ k-ft}}$ Typical Span Negative Bending Capacity

■ <u>Deck Shear Capacity</u>:

$d = 4.625 in$

$b = 49 in$

$V_c = 2\lambda\sqrt{f_c'}\, b_w d$

$= 2(1.0)(4000)^{1/2}(49)(4.625)$

$V_c = 28666 \text{ lbs}$

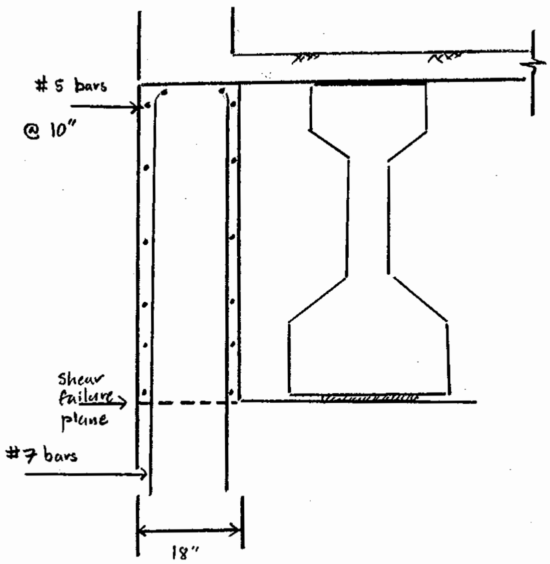
$\phi V_c = 0.75(28666)$

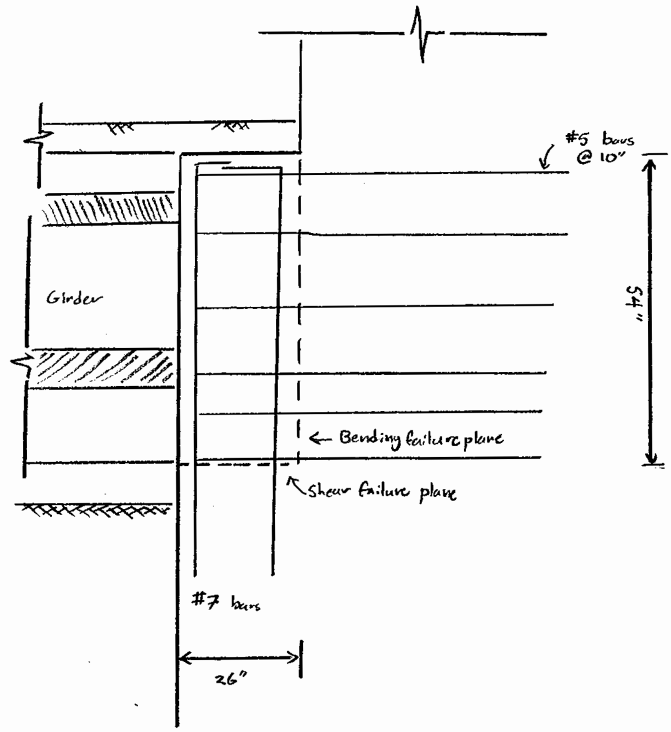
$= 21499.5 \text{ lbs}$

$\phi V_c = 21.4995 \text{ kips}$

$\boxed{\phi V_c = 21.5 \text{ kips}}$

- Wing Wall Capacity:





$f_c' = 4000\text{psi} \quad f_y = 60{,}000 \text{ psi}$

o Shear Failure: (Bottom Plane)

$$V_c = 2\lambda\sqrt{f_c'}\, b_w d$$
$$= 2(1.0)\sqrt{4000}\,(18)(26)$$
$$V_c = 59197.8 \text{ lbs} = 59.1978 \text{ kips}$$

$$\phi V_c = (0.75)(59197.8) = 44398.4 \text{ lbs}$$

$$\boxed{\Rightarrow \phi V_c = 44.398 \text{ kips}}$$

o Shear Failure: (Vertical Plane)

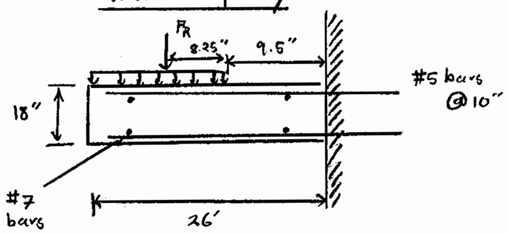
$$V_c = 2\lambda\sqrt{f_c'}\, b_w d$$
$$= 2(1)\sqrt{4000}\,(18)(54)$$
$$V_c = 122949 \text{ kips} = 122.949 \text{ kips}$$

$$\phi V_c = 0.75\,(122949) = 92212 \text{ kips}$$

$$\boxed{\Rightarrow \phi V_c = 92.212 \text{ kips}}$$

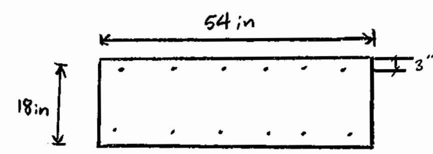
Wing Wall Capacity:

- Flexture Capacity:



Depth into page = 54 in

Bottom of Failure section



$f_c' = 4000 \text{ psi}$ $f_y = 60{,}000 \text{ psi}$

I) Moment at fixed end:

$$M = F_R(8.25 + 9.5)$$
$$M = 17.75\ F_R$$

① Method ①:

I) Moment Capacity $= (f_y \times n \times A_s)$ (depth Rebars) ← No. 5 bars

$$= (60{,}000 \times (6) \times (0.31))(18 - 3 - .625/2)$$
$$M_n = 1.63913 \times 10^6 \text{ lb-in}$$

II) $M_n = F_R(17.75)$

$$F_R = \frac{1.63913 \times 10^6 \text{ lb-in}}{17.75}$$
$$F_R = 92345.1 \text{ lbs}$$

$$\boxed{\Rightarrow F_R = 92.3451 \text{ kips}}$$ Flexture Capacity

② Method ②:

I) $M_n = \omega f_c'(1 - 0.59\omega) b d^2$

II) $\rho = \dfrac{A_s}{A_c} = \dfrac{(6 \times 0.31)}{(18 \times 54)}$

$$\rho = .001914$$

III) $\omega = \rho \dfrac{f_y}{f_c'} = \dfrac{(.001914)(60)}{4}$

$$\omega = .02871$$

IV) $M_n = \omega f_c'(1 - 0.59\omega) b d^2$

$$= (.02871)(4)(1 - 0.59(.02871)) \times (54)(18^2)$$
$$M_n = 1975.21 \text{ k-in}$$

V) $F_R = \dfrac{M_n}{17.75} = \dfrac{(1975.21)}{17.75}$

$$\boxed{F_R = 111.279 \text{ kips}}$$

o Flexture Capacity:

③ Method ③:

I) $a = \frac{A_s f_y}{0.85 f_c' b} = \frac{(6 \times 0.31)(60)}{0.85 (4)(54)}$

$a = .607843 \text{ in}$

II.) $d = 18 - 3 - 0.625/2$

$d = 14.6875 \text{ in}$

III.) $M_n = A_s f_y (d - a/2)$

$= 111.6 (14.6875 - .60784/2)$

$M_n = 1605.21 \text{ k-in}$

IV.) $F_R = \frac{M_n}{17.75} = \frac{(1605.21)}{17.75}$

$\boxed{F_R = 90.43 \text{ kips}}$ ∴ Most Conservative

▲ Summary of Results: (Wing Wall)

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

∴ Bottom will shear, then will fail in flexture:

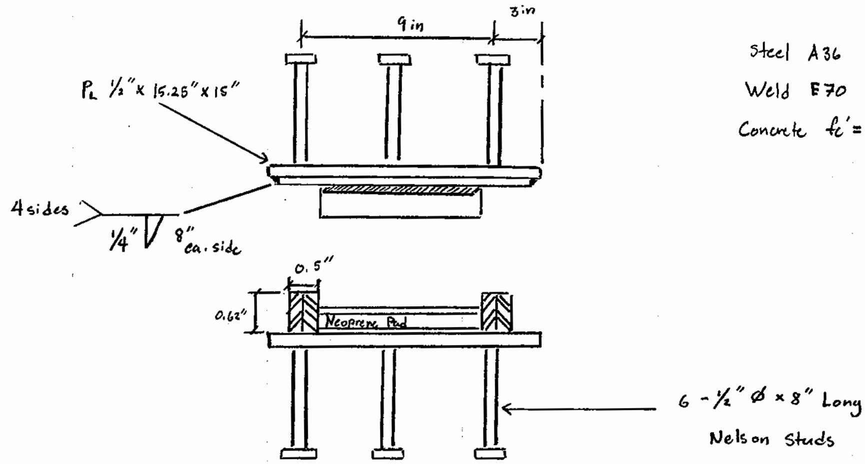
$\boxed{\Rightarrow \text{Capacity} = 149.63 \text{ kips}}$ per abutment

■ <u>Bearing Capacity</u> : (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 A36
Weld E70
Concrete $f_c' = 4000\,psi$



▲ <u>Steel Capacities</u>:

o <u>Side Steel Restrainers</u>: (.62" x 0.5")

• <u>Metal Shear Strength</u>

$F_y = 36$ ksi   Length = 9 in (ea. side)
$F_u = 58$ ksi

I) $V_n = 0.6\, F_y\, A\, C_v$

II) $2.24\left(\frac{E}{F_y}\right)^{1/2} = 2.24\left(29{,}000/36\right)^{1/2}$

$= 63.5764$

III) $h/t = .62/.5 = 1.24$

∴ $h/t < 2.24\,(E/F_y)^{1/2}$

⇒ Strength is governed by shear yielding
$C_v = 1.0$

IV) $V_n = 0.6\, F_y\, A C_v$
$= 0.6\,(36)(0.5 \times 9)(1.0)$
$V_n = 97.2$ kips

$\boxed{\Rightarrow V_n = 97.2 \text{ kips/Bearing Pad}}$

▲ Steel Capacities:

o Nelson Stud Shear Capacity:

* Assumptions:

① Studs are stainless steel (as formed)

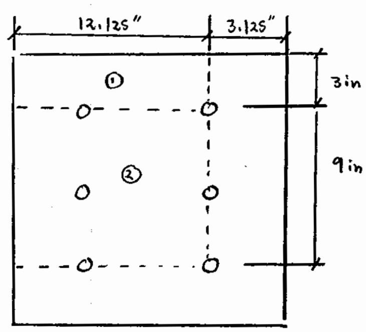
② Thread diameter ½ - 13 UNC ⇒ $A_b = .1416 \text{ in}^2$

• From Nelson Charts:

Shear Strength = 7982 lbs = 7.982 kips

$$\boxed{V_n = 7.982 \text{ kips/stud}}$$

o Block Shear:



t = ½ in

A36 steel

$F_y = 36$ ksi

$F_u = 58$ ksi

Block Shear ① controls strength capacity

However ② is more likely

② Block Shear:

I) $A_{gv} = t_{member}(\text{Length Shear} \times \# \text{Bolt lines})$

$= (½)(12.125 \times 3)$

$A_{gv} = 18.1875 \text{ in}^2$

II) $A_{nv} = t_{member}(\text{length} - \# \text{hole dia} (d + 1/8))(\# \text{bolt lines})$

$= (½)(12.125 - (1.5)(½ + 1/8))(3)$

$A_{nv} = 16.7813 \text{ in}^2$

III) $A_{nt} = (½)[(9) - (2)(½ + 1/8)](3)$

$A_{nt} = 11.625 \text{ in}^2$

IV) $R_n = 0.6 F_u A_{nv} + U_{bs} F_u A_{nt}$

$= 0.6(58)(16.7813) + 1.0(58)(11.625)$

$R_n = 1258.24$ kips

or

$R_n = 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$

$= 0.6(36)(18.1875) + (1.0)(58)(11.625)$

$R_n = 1067.1$ kips ∴ controls

$$\boxed{\Rightarrow R_n = 1067.1 \text{ kips}}$$

Block Shear

▲ Weld Capacity:

o E70 Fillet Weld: ($1/4'' = 4/16''$)

I.) $\phi R_n = 1.392 D$
$= 1.392(4)$
$\phi R_n = 5.568$ kips/in

II.) Weld is 8" long on 4 sides:

$\phi R_n = (5.568)(8 \times 4) = 178.176$ kips

$\boxed{\phi R_n = 178.2 \text{ kips}}$

o Base Metal Capacity:

$t = 1/2''$
$F_y = 36$ ksi
$F_u = 58$ ksi

I.) Yield Strength:
$\phi R_n = 0.6 F_y t = 0.6(36)(1/2)$
$\phi R_n = 10.8$ k/in

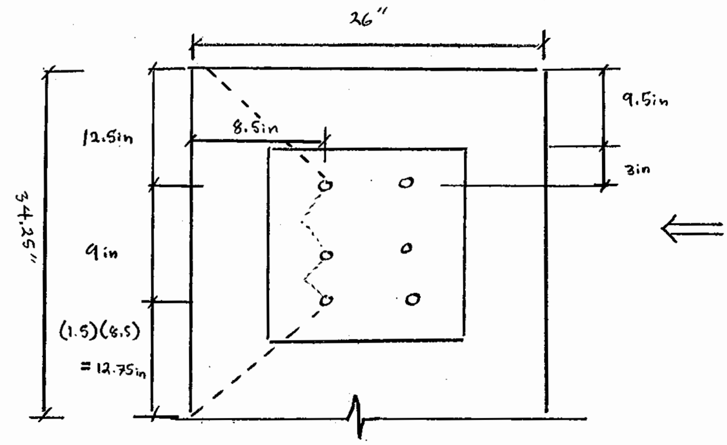
II.) Rupture Strength:
$\phi R_n = 0.45 F_u t = 0.45(58)(1/2)$
$\phi R_n = 13.05$ k/in

III.) ∴ Yield Controls: (For 32")
$\phi R_n = 10.8(32) = 345.6$ kips

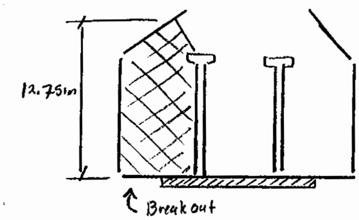
$\boxed{\phi R_n = 345.6 \text{ kips}}$

▲ Concrete Capacity:

○ Bolts in Girder: $f_c' = 6000$ psi



* Note: The back row will cause concrete break out. Front row will fail in shear



• Using ACI 318-08 Appendix D: (Section D.6.2)

I.) $V_{cbg} = \left(A_{vc}/A_{vco}\right)\psi_{ec,v}\,\psi_{ed,v}\,\psi_{c,v}\,\psi_{h,v}\,V_b$

II.) $c_{a1} = 8.5\text{in}$

III.) $A_{vc} = (34.25)(12.75) = 436.688\ \text{in}^2$

IV.) $A_{vco} = 4.5 c_{a1}^2$

$= 4.5(8.5)^2$

$A_{vco} = 325.125\ \text{in}^2$

V.) Section D.6.2.2:

$l_e = h_{ef} = 8\text{in}$

$d_a = 1/2\ \text{in}$

$V_b = \left(7\left(l_e/d_a\right)^{0.2}\sqrt{d_a}\right)\lambda\sqrt{f_c'}\,(c_{a1})^{1.5}$

$= \left(7(16)^{0.2}\sqrt{1/2}\right)(1.0)(6000)^{1/2}(8.5)^{1.5}$

$V_b = 16542.9$ lbs

VI.) Anchors are not loaded eccentrically:

$\psi_{ec} = 1.0$

VII.) $c_{a2} = 12.5 < 1.5c_{a1} = 12.75$

$\psi_{ed} = 0.7 + \dfrac{0.3\,c_{a2}}{(1.5c_{a1})}$

$= 0.7 + \dfrac{0.3(12.5)}{12.75}$

$\psi_{ed} = .994$

VIII.) $\psi_{c,v} = 1.4$

IX.) $h_a = 8\text{in} < 1.5c_{a1}$:

$\psi_{h,v} = \left(\dfrac{1.5c_{a1}}{h_a}\right)^{1/2}$

$= \left(12.75/8\right)^{1/2}$

$\psi_{h,v} = 1.26244$

X.) $V_{cbg} = \left(436.7/325.1\right)(1.0)(.994)(1.4)\times(1.2624)(16542.9)$

$V_{cbg} = 39039.9$ lbs

∴ Total Capacity:

$V_n = V_{cbg} + 3(\text{Stud Shear})$

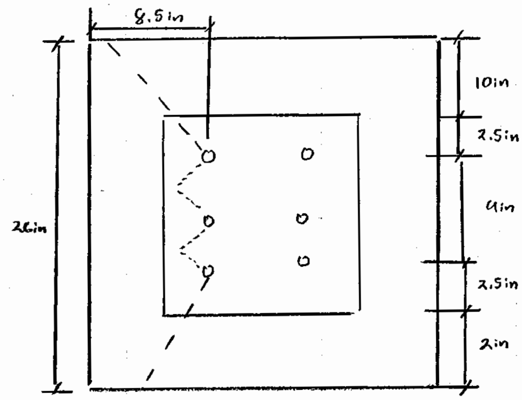
$= 39.040 + 3(7.982)$

$V_n = 62.9859$ kips

$\boxed{V_n = 62.990\ \text{kips}}$

▲ **Concrete Capacity:**

o **Bolts in Abutment:** $f_c' = 4000\,psi$



I.) $C_{a_1} = 8.5$ in

$h = 1.5(8.5) = 12.75$ in

II.) $A_{vc} = (26)(12.75) = 331.5\,in^2$

III.) $A_{vco} = 4.5\,C_{a_1}^2 = 325.125\,in^2$

IV.) $l_e = h_{ef} = 8\,in$

$d_a = 1/2$ in

$$V_b = \left(7\left(\frac{l_e}{d_a}\right)^{0.2}\sqrt{d_a}\right)\lambda\sqrt{f_c'}\,(C_{a_1})^{1.5}$$

$$= (7(16)^{0.2}(1/2)^{1/2})(1.0)(4000)^{1/2}(8.5)^{1.5}$$

$V_b = 13507.2$ lbs

V.) $C_{a_2} = 12.5 \le 1.5C_{a_1} = 12.75$

$\psi_{ed} = .994$

VI.) $\psi_{cv} = 1.4$

VII.) $h_a = 8\,in \le 1.5C_{a_1}$

$\psi_{hv} = 1.26244$

X.) $V_{cbg} = (A_{vc}/A_{vco})\,\psi_{ec},\,\psi_{cd},\,\psi_c,\,\psi_h,\,V_b$

$= (331.5/325.125)(1.0)(.994)(1.4)(1.26244)(13507.2)$

$V_{cbg} = 24194.9$ lbs

XI.) Total Capacity:

$V_n = V_{cbg} + 3$ (stud Shear)

$= 24.194 + 3(7.982)$

$V_n = 48.1409$ kips

$$\boxed{V_n = 48.14\ kips}$$

National® Brand 42-182 100 SHEETS

■ 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 = 62.99 kips
- Concrete Abutment Capacity = 48.14 kips (likely failure)

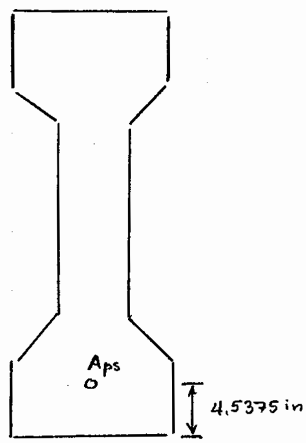
* Note: Two bearing pads at far end will fail due to concrete abutment. The remaining 14 will fail due to Nelson Studs.

$\therefore$ Total Resistance = 2(48.14) + (14)(47.892)

$$\boxed{TR = 766.77 \text{ kips}}$$

16 Bearing pads (Lateral Resistance)

Negative Bending Capacity:



Properties:

$f_c' = 6000$ psi

$A_{ps} = 40 \times .153 = 6.12\ in^2$

$f_{pu} = 270,000$ psi

$P_e = (270000)(0.7)(6.12) = 1.15668 \times 10^6$ lbs

- Centroid:
  - $A_1 = 1.111\ ft^2$
    $y_1 = 50\ in$
  - $A_2 = 0.25\ ft^2$
    $y_2 = 44\ in$
  - $A_3 = 2.111\ ft^2$
    $y_3 = 27\ in$
  - $A_4 = 0.5625\ ft^2$
    $y_4 = 11\ in$
  - $A_5 = 1.444\ ft^2$
    $y_5 = 4\ in$

$$\bar{y} = \frac{\Sigma A_i y_i}{\Sigma A_i}$$

$\bar{y} = 24.735$ in

$C_b = 24.735$ in

$C_t = 29.265$ in

$e = 20.1975$ in

- Moment of Inertia:
  - $I_1 = 1/12\ bh^3 = 1/12(20)(8)^3 = 853.3\ in^4$
    $I_1 + Ad^2 = (853.3) + (160)(25.265)^2 = 102974\ in^4$
  - $I_2 = 1/36\ bh^3 = 1/36(6)^4 = 36\ in^4$
    $I_2 + Ad^2 = 36 + 18(19.265)^2 = 6716.52\ in^4$
  - $I_3 = 1/12\ bh^3 = 1/12(8)(38)^3 = 36581.3\ in^4$
    $I_3 + Ad^2 = 36581.3 + (304)(2.265)^2 = 38140.9\ in^4$
  - $I_4 = 1/36\ bh^3 = 1/36(9)^4 = 182.25\ in^4$
    $I_4 + Ad^2 = (182.25) + (40.5)(13.735)^2 = 7822.58\ in^4$
  - $I_5 = 1/12\ bh^3 = 1/12(26)(8)^3 = 1109.33\ in^4$
    $I_5 + Ad^2 = (1109.33) + (208)(20.735)^2 = 90536.9\ in^4$

$I_c = (102974) + (2)(6716.52) + 38140.9 + (2)(7822.58) + 90536.9$

$I_c = 260730\ in^4$

- Unsubmerged Case:
  - Geometric Properties:

$A_c = 782.12\ in^2$ $\quad S_t = 8909.28\ in^3$

$I_c = 260730\ in^4$ $\quad S_b = 10540.9\ in^4$

$C_t = 29.265\ in$ $\quad L = 1272\ in$

$C_b = 24.735\ in$ $\quad N = 24$

$r^2 = 330.406\ in^2$ $\quad f_t = 429.516\ psi$

$e = 20.1975\ in$ $\quad f_c = -5100\ psi$

$P_e = 1.15668 \times 10^6\ lbs$

I.) w_D = self weight + topping

$= (5.48)(150) + (5+9/12)(6/12)(150) + (5+9/12)(1.5/12)(152)$

$= 1362.5\ lb/ft$

$w_D = 113.54\ lb/in$

$\Rightarrow M_D = w_D L^2/8 = 2.29632 \times 10^7\ lb\text{-}in$

II.) Tensile Limit:

$$w_{wt} = \frac{8}{L^2}\left[M_D + S_t f_t - \frac{S_t P_e}{A_c}\left\{\frac{e C_t}{r^2} - 1\right\}\right]$$

$= 8/L^2\ [M_D - 2.11386 \times 10^6]$

$= 8/(1272)^2\ [(2.29632 \times 10^7) - 2.11386 \times 10^6]$

$w_{wt} = 103.088\ lb/in$

$F_w = 131.13\ kips$ (tension)

III.) Compression Limit:

$$w_{wc} = \frac{8}{L^2}\left[M_D - S_b f_c - \frac{S_b P_e}{A_c}\left\{1 + \frac{e c_b}{r^2}\right\}\right]$$

$= 8/L^2\ [M_D + 1.45986 \times 10^7]$

$= 8/1272^2\ [2.2963 \times 10^7 + 1.45986 \times 10^7]$

$w_{wc} = 185.721\ lb/in$

$F_w = 236.238\ kips$ (compression)

- Submerged Case:

* Assume 50% air pocket

I.) $w_D = 1362.5 - (9.07375)(64) - 8.83(64) = 216.66\ lb/ft$

$w_D = 18.055\ lb/in$

$\Rightarrow M_D = 3.65159 \times 10^6\ lb\text{-}in$

II.) Tensile Limit:

$w_{wt} = 8/1272^2\ [3.6 \times 10^6 - 2.11386 \times 10^6]$

$= +7.603\ lb/in$

$F_w = 9.671\ kips$ (tension)

III.) Compression Limit:

$w_{wc} = 8/1272^2 [3.6 \times 10^6 + 1.45986 \times 10^7]$

$w_{wc} = 90.2367\ lb/in$

$F_w = 114.78\ kips$ (compression)

- **Summary of Results:**

  - Unsubmerged:
    - Tensile: F_w = 131.13 kips
    - Compression: F_w = 236.238 kips

  - Submerged:
    - Tensile: F_w = 9.6710 kips
    - Compression: F_w = 114.78 kips

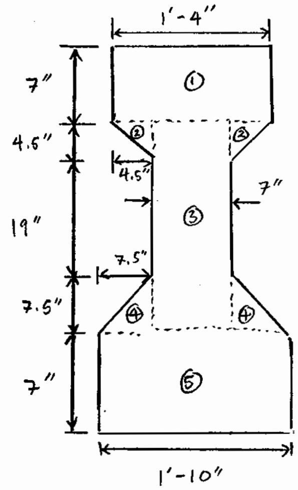
  - Loss:
    - Tensile = 92.62%
    - Compression = 51.41%

Calculations By: Daniel Lum

Buoyancy Calculations:

Typical Girder Section: AASHTO Type III Girder (Spans 2, 3, & 4)

- Sheet T3



$A_1 = (1+4/12)(7/12) = 0.778\ ft^2$

$A_2 = 2[\frac{1}{2}(4.5/12)(4.5/12)] = 0.1406\ ft^2$

$A_3 = (7/12)(4.5/12 + 19/12 + 7.5/12) = 1.507\ ft^2$

$A_4 = 2[1/2(7.5/12)(7.5/12)] = 0.3906\ ft^2$

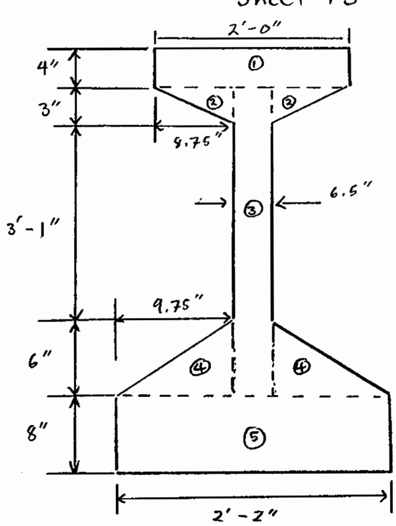
$A_5 = (1+10/12)(7/12) = 1.0694\ ft^2$

- $A_T = \sum_{i=1}^{5} A_i = 559.5\ in^2$

$\boxed{A_T = 3.885\ ft^2}$ Type III Girder

Typical Girder Section: Keehi Type IV Girder (span 1)

- Sheet T3



$A_1 = (2)(4/12) = 0.667\ ft^2$

$A_2 = 2[1/2(3/12)(8.75/12)] = 0.1823\ ft^2$

$A_3 = (6.5/12)(3/12+3+1/12+6/12) = 2.0764\ ft^2$

$A_4 = 2[1/2(9.75/12)(6/12)] = 0.40625\ ft^2$

$A_5 = (2+2/12)(8/12) = 1.444\ ft^2$

- $A_T = \sum_{i=1}^{5} A_i = 4.77604\ ft^2$

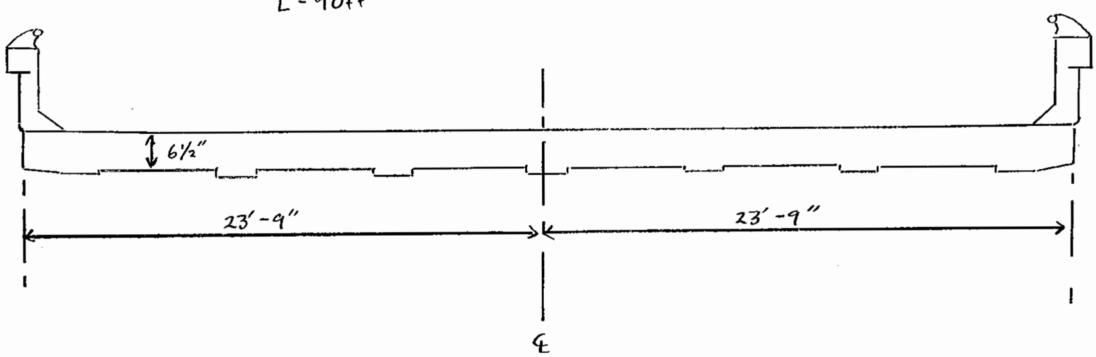
$\boxed{A_T = 4.776 ft^2}$ Type IV Girder

▲ 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
$L = 90ft$

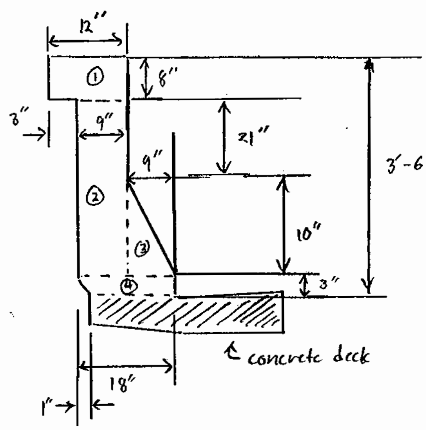


• Deck:

$$A_T = \left(23 + \tfrac{9}{12} + 23 + \tfrac{9}{12}\right)\left(\tfrac{6.5}{12}\right) = 25.729\ ft^2$$

$\boxed{A_T = 25.73\ ft^2}$ Deck Span 1
$L = 90ft$

• Railing:



$$A_1 = \left(\tfrac{13}{12}\right)\left(\tfrac{8}{12}\right) = 0.67\ ft^2$$

$$A_2 = \left(\tfrac{9}{12}\right)\left(\tfrac{21}{12} + \tfrac{10}{12}\right) = 1.9375\ ft^2$$

$$A_3 = \tfrac{1}{2}\left(\tfrac{9}{12}\right)\left(\tfrac{10}{12}\right) = 0.3125\ ft^2$$

$$A_4 = \left(\tfrac{18}{12}\right)\left(\tfrac{3}{12}\right) = 0.375\ ft^2$$

• $A_T = \sum_{i=1}^{4} A_i = 3.29167\ ft^2$

$\boxed{A_T = 3.292\ ft^2}$ Railing

▲ Deck Section: (spans 2, 3, & 4)

☆ Note: The width of the bridge varies from abutment 1 to abutment 2
Slope of width = .005247 ft/ft

☆ An average width will be used for spans 2, 3, & 4

I.) Span 2:

- Width = $(23 + 9/12) + (.005247)(90 + 30)$
  Width = 24.3796 ft
- thickness (t) = 6.75 in (sheet S2)
- Area = $(23 + 9/12 + 24.3796)(6.75/12) = 27.0729$ ft

$\boxed{A_T = 27.07\ ft^2}$ Deck Span 2
L = 60 ft

II.) Span 3:

- Width = $(23 + 9/12) + (.005247)(90 + 60 + 30)$
  Width = 24.6944 ft
- $t = 6.75$ in (sheet S2)
- Area = $(23. + 9/12 + 24.6944)(6.75/12) = 27.25\ ft^2$

$\boxed{A_T = 27.25\ ft^2}$ Deck Span 3
L = 60 ft

III.) Span 4:

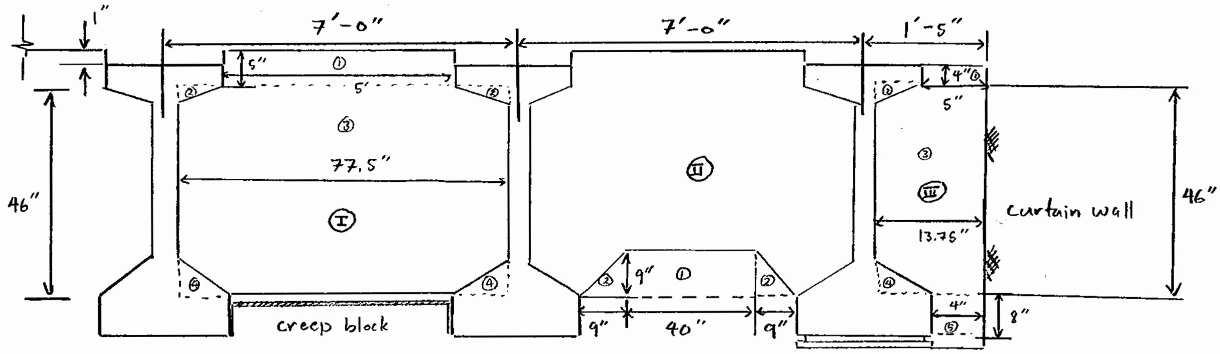
- Width = $(23 + 9/12) + (.005247)(90 + 60 + 60 + 30)$
  Width = 25.0093 ft
- $t = 6.75$ in (sheet S2)
- Area = $(23 + 9/12 + 25.0093)(6.75/12) = 27.4271\ ft^2$

$\boxed{A_T = 27.43\ ft^2}$ Deck Span 4
L = 60 ft

- Concrete Diaphragms:
  - Beam D : (Kechi Type IV Girder)

    Span 1

    - Sheet 526



I.) Area I:

$A_1 = (5)(5/12) = 2.083\ ft^2$

$A_2 = 2\{ \tfrac{1}{2}(8.75/12)(3/12)\} = 0.1823\ ft^2\ (-)$

$A_3 = (77.5/12)(46/12) = 24.7569\ ft^2$

$A_4 = 2\{ \tfrac{1}{2}(9.75/12)(6/12)\} = 0.40625\ ft^2\ (-)$

- $A_T = A_1 - A_2 + A_3 - A_4$

  $= (2.083) - (.1823) + 24.7569 - 0.40625$

  $A_T = 26.2517\ ft^2$

$\boxed{A_T = 26.25\ ft^2}$ Area I Concrete Diaphragm

II.) Area II:

$A_1 = (40/12)(9/12) = 2.5\ ft^2\quad (-)$

$A_2 = 2\{ \tfrac{1}{2}(9/12)(9/12)\} = 0.5625\ ft^2\ (-)$

- $A_T = A_{T\,area\,I} - A_1 - A_2$

  $= (26.2517) - 2.5 - 0.5625$

  $A_T = 23.1892\ ft^2$

$\boxed{A_T = 23.19\ ft^2}$ Area II Concrete Diaphragm

III.) Area III:

$A_1 = (5/12)(4/12) = 0.1389\ ft^2$

$A_2 = \tfrac{1}{2}(8.75/12)(3/12) = .091\ ft^2\ (-)$

$A_3 = (13.75/12)(46/12) = 4.39\ ft^2$

$A_4 = \tfrac{1}{2}(9.75/12)(6/12) = 0.203\ ft^2\ (-)$

$A_5 = (4/12)(8/12) = 0.22\ ft^2$

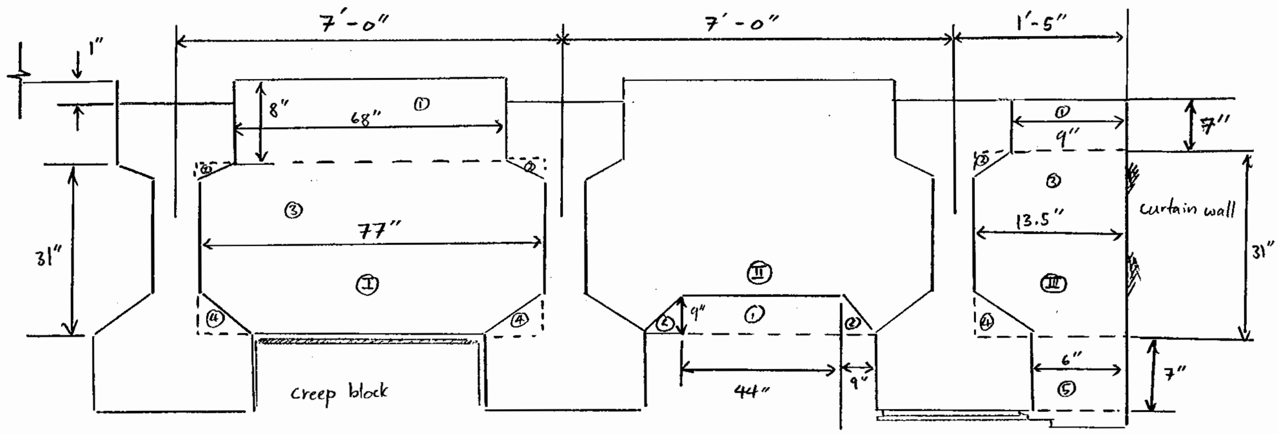
- $A_T = A_1 - A_2 + A_3 - A_4 + A_5$

  $= (0.139) - (.091) + (4.39) - (.203) + (.22)$

  $A_T = 4.4592\ ft^2$

$\boxed{A_T = 4.46\ ft^2}$ Area III Air Pocket

- **Concrete Diaphragms: Cont.**
  - Beam A : (AASHTO Type III Girder)
    - Sheet S24



I.) Area I:

$A_1 = (68/12)(8/12) = 3.78 \text{ ft}^2$

$A_2 = 2\{1/2(4.5/12)(4.5/12)\} = 0.1406 \text{ ft}^2$ (-)

$A_3 = (77/12)(31/12) = 16.576 \text{ ft}^2$

$A_4 = 2\{1/2(7.5/12)(7.5/12)\} = 0.3906 \text{ ft}^2$ (-)

- $A_T = A_1 - A_2 + A_3 - A_4 = 19.8229 \text{ ft}^2$

$\boxed{A_T = 19.82 \text{ ft}^2}$ Area I Concrete Diaphragm

II.) Area II:

$A_1 = (44/12)(9/12) = 2.75 \text{ ft}^2$

$A_2 = 2\{(1/2)(9/12)(9/12)\} = 0.5625 \text{ ft}^2$

- $A_T = A_{area\,I} - A_1 - A_2$
  $= 19.8229 - 2.75 - 0.5625$
  $A_T = 16.5104 \text{ ft}^2$

$\boxed{A_T = 16.51 \text{ ft}^2}$ Area II Concrete Diaphragm

III.) Area III:

$A_1 = (9/12)(7/12) = 0.4375 \text{ ft}^2$

$A_2 = 1/2(4.5/12)(4.5/12) = 0.0703 \text{ ft}^2$ (-)

$A_3 = (13.5/12)(31/12) = 2.90625 \text{ ft}^2$

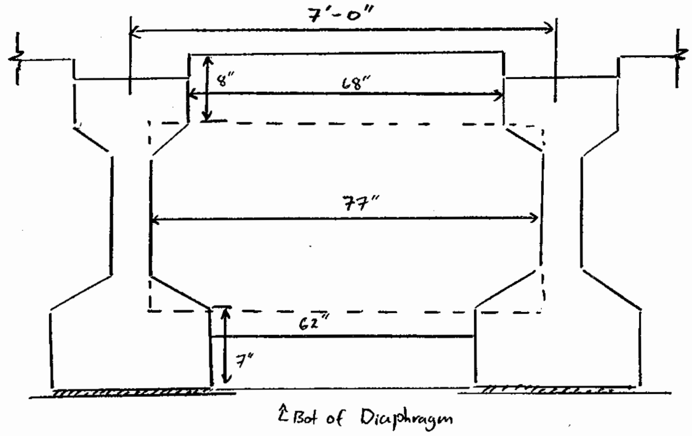
$A_4 = 1/2(7.5/12)(7.5/12) = 0.1953 \text{ ft}^2$ (-)

$A_5 = (6/12)(7/12) = 0.29167 \text{ ft}^2$

- $A_T = A_1 - A_2 + A_3 - A_4 + A_5 = 3.36979 \text{ ft}^2$

$\boxed{A_T = 3.37 \text{ ft}^2}$ Area III Air Pocket

- Concrete Diaphragms: Cont.
  - Beam B:

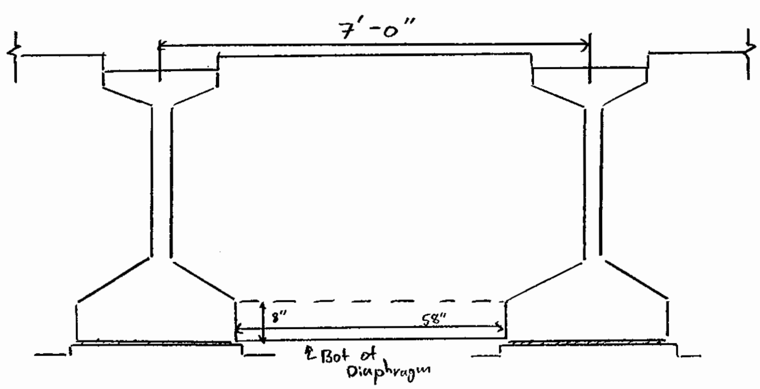


* Diaphragm goes to the bottom of the girders

$$A_T = 19.8229 + (63/12)(7/12) = 22.8368\ ft^2$$

$$\boxed{A_T = 22.84\ ft^2}$$

- Beam G:



$$A_T = 26.2517 + (58/12)(8/12) = 29.4739\ ft^2$$

$$\boxed{A_T = 29.47\ ft^2}$$

- Concrete Diaphragms: cont.
  - Beam D: cont.

$$\text{Total Area} = 4(\text{Area II}) + 2(\text{Area I})$$
$$= 4(23.19) + 2(26.25)$$
$$\text{Total Area} = 145.26\ ft^2$$

$\boxed{A_T = 145.26\ ft^2}$ Beam D Diaphragm
Thickness = 1ft

- Beam A: cont.

$$\text{Total Area} = 4(\text{Area II}) + 2(\text{Area I})$$
$$= 4(16.51) + 2(19.82)$$
$$\text{Total Area} = 105.68\ ft^2$$

$\boxed{A_T = 105.68\ ft^2}$ Beam A Diaphragm
Thickness = 1 ft

- Beam B: Cont.

$$\text{Total Area} = 6(\text{Area})$$
$$= 6(22.84)$$
$$\text{Total Area} = 137.04\ ft^2$$

$\boxed{A_T = 137.04\ ft^2}$ Beam B Diaphragm
Thickness = 1.1667 ft

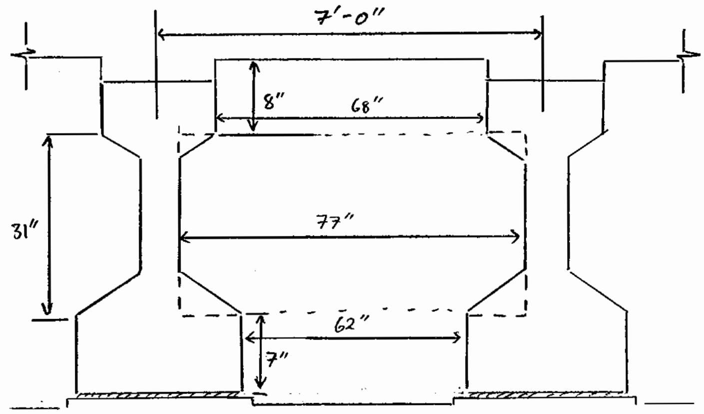
- Beam G: Cont.

$$\text{Total Area} = 6(\text{Area})$$
$$= 6(29.47)$$
$$\text{Total Area} = 176.82\ ft^2$$

$\boxed{A_T = 176.82\ ft^2}$ Beam G Diaphragm
Thickness = 2ft

■ Air Pocket Calculation:

▲ AASHTO Type III Girders: (No Creep Block Section)

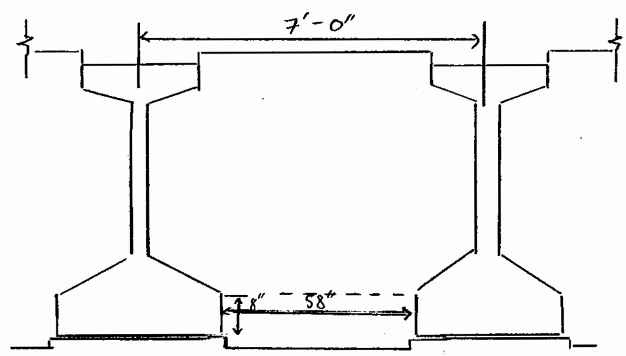


* Note: Air will fill entire area between girders

$$A_T = \underbrace{19.8229}_{\text{upper area}} + \left(\tfrac{62}{12}\right)\left(\tfrac{7}{12}\right) = 22.8368\ ft^2$$

$\boxed{A_T = 22.84\ ft^2}$ AASHTO Type III Girder (No Creep Block) Air Pocket

▲ Keehi Type IV Girder: (No Creep Block Section)



$$A_T = \underbrace{26.2517}_{\text{upper area}} + \left(\tfrac{58}{12}\right)\left(\tfrac{8}{12}\right) = 29.4739\ ft^2$$

$\boxed{A_T = 29.47\ ft^2}$ Keehi Type IV Girder (No Creep Block) Air Pocket

▲ AASHTO Type III Girder: (Creep Block Section)

$\boxed{A_T = 19.82\ ft^2}$ AASHTO Type III Girder Air Pocket

▲ Keehi Type IV Girder: (Creep block Section)

$\boxed{A_T = 26.25\ ft^2}$ Keehi Type IV Girder Air Pocket

▲ Reduction of air pocket:

o AASHTO Type III Girder:

◆ No Creep Block:

- $h = (6.5/12) + (8/12 + 31/12 + 7/12) = 4.375\ ft$
- $P_2 = 14.7 + h(64/144)$
  $= 14.7 + (4.375)(64/144)$
  $P_2 = 16.644\ psi$
- $A_2 = \dfrac{P_1 A_1}{P_2} = \dfrac{(14.7)(22.84)}{16.644}$
  $A_2 = 20.1718\ ft^2$

$\boxed{A_T = 20.17\ ft^2}$ Compressed Air

◆ Creep Block:

- $h = (6.5/12) + (8/12 + 31/12) = 3.79167\ ft$
- $P_2 = 14.7 + h(64/144)$
  $= 14.7 + (3.79)(64/144)$
  $P_2 = 16.3852\ psi$
- $A_2 = \dfrac{P_1 A_1}{P_2} = \dfrac{(14.7)(19.82)}{16.3852}$
  $A_2 = 17.7816\ ft^2$

$\boxed{A_T = 17.78\ ft^2}$ Compressed Air

◆ Side Area: (From Pg. 5)

- $h = 4.375\ ft$
- $P_2 = 16.644\ psi$
- $A_2 = \dfrac{P_1 A_1}{P_2} = \dfrac{(14.7)(3.37)}{16.644}$
  $A_2 = 2.976\ ft^2$

$\boxed{A_T = 2.98\ ft^2}$ Compressed Air

o Keehi Type IV Girder:

◆ No Creep Block:

- $h = (6.5/12) + (5/12 + 54/12) = 5.4583\ ft$
- $P_2 = 14.7 + h(64/144)$
  $= 14.7 + (5.46)(64/144)$
  $P_2 = 17.1259\ psi$
- $A_2 = \dfrac{P_1 A_1}{P_2} = \dfrac{(14.7)(29.47)}{17.1259}$
  $A_2 = 25.2955\ ft^2$

$\boxed{A_T = 25.30\ ft^2}$ Compressed Air

◆ Creep Block:

- $h = (6.5/12) + (5/12 + 46/12) = 4.79167\ ft$
- $P_2 = 14.7 + (4.792)(64/144) = 16.8296\ psi$
- $A_2 = \dfrac{P_1 A_1}{P_2} = \dfrac{(14.7)(26.25)}{16.8296}$
  $A_2 = 22.9283\ ft^2$

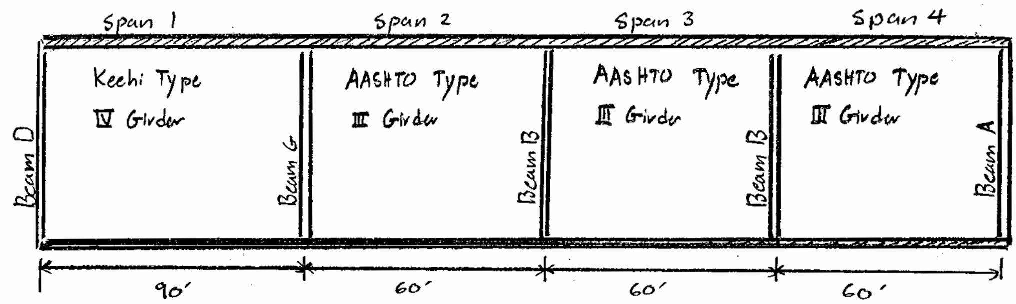
$\boxed{A_T = 22.93\ ft^2}$ Compressed Air

◆ Side Area: (From Pg. 4)

- $h = 5.4583\ ft$
- $P_2 = 17.1259\ psi$
- $A_2 = \dfrac{P_1 A_1}{P_2} = \dfrac{(14.7)(4.46)}{17.1259}$
  $A_2 = 3.82824\ ft^2$

$\boxed{A_T = 3.83\ ft^2}$ Compressed Air

Buoyancy Calculations:



- Span 1:
  - Concrete:

$$\forall_{S1} = \{7(\text{Keehi Girder}) + (\text{Deck}) + 2(\text{Railing})\}(90) + (\text{Beam D Diaphragm})(1)$$

$$= \{7(4.776) + 25.73 + 2(3.292)\}(90) + (147.26)(1)$$

$$\boxed{\forall_{S1} = 6064.4\ ft^3} \text{ Concrete}$$

  - Air Pocket:

$$\forall_{S1} = \{6(\text{No Creep Block Area})\}(90)$$

$$= \{6(25.30)\}(90)$$

$$\boxed{\forall_{S1} = 13662.0\ ft^3} \text{ Air}$$

- Span 2:
  - Concrete:

$$\forall_{S2} = \{7(\text{AASHTO}) + \text{Deck} + 2(\text{Railing})\}(60) + (\text{Beam G Diaphragm})(1)$$

$$= \{7(3.885) + 27.07 + 2(3.292)\}(60) + (176.82)(2)$$

$$\boxed{\forall_{S2} = 4004.6\ ft^3} \text{ Concrete}$$

  - Air Pocket:

$$\forall_{S2} = \{6(\text{No Creep Block Area})\}(60)$$

$$= \{6(20.17)\}(60)$$

$$\boxed{\forall_{S2} = 7261.2\ ft^3} \text{ Air}$$

■ Buoyancy Calculations: Cont.

- Span 3:
  - Concrete:

$$\forall_{S3} = (60)\{7(\text{AASHTO}) + \text{Deck} + 2(\text{Railing})\} + \text{Beam B Diaphragm}(1.1667)$$
$$= (60)\{7(3.885) + (27.25) + 2(3.292)\} + (137.04)(1.1667)$$
$$\forall_{S3} = 3821.62\ ft^3$$

$\boxed{\forall_{S3} = 3821.6\ ft^3}$ Concrete

  - Air Pocket:

$\boxed{\forall_{S3} = 7261.2\ ft^3}$ Air

- Span 4:
  - Concrete:

$$\forall_{S4} = \{7(\text{AASHTO}) + \text{Deck} + 2(\text{Railing})\}(60) + (\text{Beam B})(1.1667) + (\text{Beam A})(1)$$
$$= \{7(3.885) + 27.43 + 2(3.292)\}(60) + (137)(1.1667) + 105.68(1)$$

$\boxed{\forall_{S4} = 3938.1\ ft^3}$ Concrete

  - Air Pocket:

$$\forall_{S4} = \{6(\text{No Creep})\}(60)$$
$$= \{6(20.17)\}(60)$$

$\boxed{\forall_{S4} = 7261.2\ ft^3}$ Air

▲ Self Weight: ($\gamma_{RC} = 150\ lb/ft^3$)

- Total Volume $= 6064.4 + 4004.6 + 3821.6 + 3938.1$
  $T\forall = 17828.7\ ft^3$

- Self Weight $= T\forall \times \gamma_{RC}$
  $= (17828.7)(150)$
  $SW = 2.67431 \times 10^6\ lbs$

▲ Buoyant Force: ($\gamma_{seawater} = 64\ lb/ft^3$)

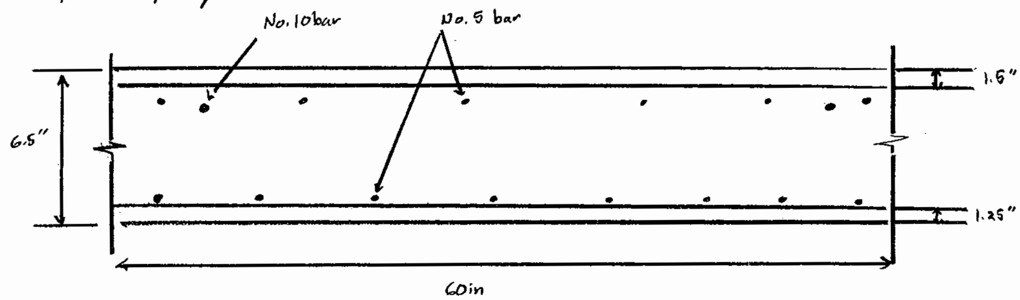
- Total Submerged Volume = (Volume Concrete − Railing) + Air Pockets
  $= (17828.7 - (2)(2.98)(270)) + 13662 + 3(7261.2)$
  $TS\forall = 51665.1\ ft^3$

- Buoyant Force $= TS\forall \times \gamma_{SW}$
  $= (51665.1)(64)$
  $BF = 3.30657 \times 10^6\ lbs$

- Summary of Buoyancy Calculations:
  - Self Weight = 2674.3 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

No. 5:
$A_s = 0.31\,in^2$
$d_b = 0.625$ in

No. 10:
$A_s = 1.27\,in^2$
$d_b = 1.27\,in^2$

* Note: Treated as singly reinforced slab

I) $d = 6.5 - (1.25) - 0.625/2 = 4.9375$ in

II) $A_s = 8$ No. 5
$= 8(0.31)$
$A_s = 2.48\,in^2$

III) $a = \dfrac{A_s f_y}{0.85 f_c' b} = \dfrac{(2.48)(60)}{(0.85 \times 3.75 \times 60)}$
$a = 0.778$ in

IV) Check steel has yielded:
• $c = a/\beta_1 = 0.778/0.85$
$c = 0.915$

• $\varepsilon_s = \left(\frac{d-c}{c}\right)\varepsilon_{cu} = .0132 \geq \varepsilon_y$ ok

V.) $\phi = 0.90$

$\phi M_n = \phi A_s f_y (d - a/2)$
$= 0.90(2.48)(60)(4.9375 - 0.778/2)$
$\phi M_n = 609.132$ k-in

$\boxed{\phi M_n = 50.76 \text{ k-ft}}$ Positive Bending

○ Negative Bending:

I) $d = 6.5 - (1.5) - 0.625/2 = 4.6875$ in

II) $A_s = 6$ No. 5 $+ 2$ No. 10
$= 6(0.31) + 2(1.27)$
$A_s = 4.4\,in^2$

III) $a = \dfrac{A_s f_y}{0.85 f_c' b} = \dfrac{(4.4)(60)}{0.85(3.75)(60)}$
$a = 1.38$ in

IV.) $\varepsilon_s \geq \varepsilon_y$ ok

V) $\phi = 0.90$

$\phi M_n = \phi A_s f_y (d - a/2)$
$= 0.90(4.4)(60)(4.6875 - 1.38/2)$
$\phi M_n = 949.759$ k-in

$\boxed{\phi M_n = 79.15 \text{ k-ft}}$ Negative Bending

- Span 2, 3 & 4 Capacity: (Moment)

Same reinforcement as span 1
$\Rightarrow t = 6.75 \text{ in}$

- Positive Bending:

$f_c' = 3,750 \text{ psi}$
$f_y = 60,000 \text{ psi}$
$b = 60 \text{ in}$

I.) $d = 6.75 - 1.25 - 0.625/2 = 5.1875 \text{ in}$

II.) $A_s = 2.48 \text{ in}^2$
$a = 0.778 \text{ in}$

III.) $\phi = 0.90$

$\phi M_n = \phi A_s f_y (d - a/2)$
$= 0.90(2.48)(60)(5.1875 - 0.778/2)$
$\phi M_n = 642.612 \text{ k-in}$

$\boxed{\phi M_n = 53.55 \text{ k-ft}}$ Positive Bending

- Negative Bending:

I.) $d = 6.75 - 1.5 - 0.625/2 = 4.9375 \text{ in}$

II.) $A_s = 4.4 \text{ in}^2$
$a = 1.38 \text{ in}$

III.) $\phi = 0.90$

$\phi M_n = \phi A_s f_y (d - a/2)$
$= 0.90(4.4)(60)(4.9375 - 1.38/2)$
$\phi M_n = 1009.16 \text{ k-in}$

$\boxed{\phi M_n = 84.10 \text{ k-ft}}$ Negative Bending

- Shear Capacity: (Span 1)

$V_c = 2\lambda\sqrt{f_c'}\, b_w d$

$= 2(1.0)(3750)^{1/2}(60)(4.94)$

$V_c = 36301.4$ lbs

$\phi V_c = (0.75)(36301.4)$

$= 27226.1$ lbs

$\boxed{\phi V_c = 27.26 \text{ kips}}$

- Shear Capacity: (Spans 2, 3 & 4)

$V_c = 2\lambda\sqrt{f_c'}\, b_w d$

$= 2(1.0)(3750)^{1/2}(60)(5.1875)$

$V_c = 38120.2$ lbs

$\phi V_c = (0.75)(38120.2)$

$= 28590.1$ lbs

$\boxed{\phi V_c = 28.59 \text{ kips}}$

- **Vertical Hinge Restrainer:**
  - (Sheet 18)

3/4" continous looped galv. cable

(4) at each abutment secures bridge to creep block

→ 8 total

6x19 wire strand or wire rope

Breaking strength = 23 tons

Total Strength = (8)(23) = 184 tons = 368 kips

$\boxed{\text{Vertical Resistance} = 368 \text{ kips}}$

(Vertical Hinge Restrainers)

- **Buoyancy:**

  - Self Weight + Vertical Hinge Restrainers = 2674.3 + 368
    = 3042.3 kips

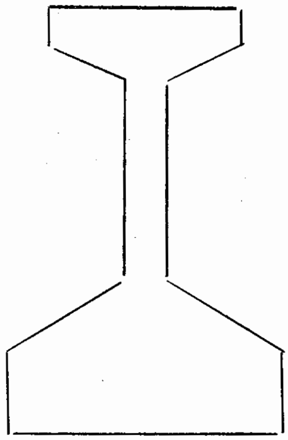
  - Buoyant Force = 3306.6 kips

  - (SW + VHR) − BF = 3042.3 − 3306.6
    = −264.3 kips (N.G.)

∴ Bridge Will Be Buoyant

⇒ Buoyant Force > Vertical Resistance

- Keehi Type IV Girder: (span 1)



Properties:

$P_e = 793.4$ kips

$f_c' = 4500$ psi

$e = 20.46$ in

- Centroid:
  - $A_1 = 0.667\ ft^2$, $y_1 = 56in$
  - $A_2 = .1823\ ft^2$, $y_2 = 53\ in$
  - $A_3 = 2.0764\ ft^2$, $y_3 = 31\ in$
  - $A_4 = .40625\ ft^2$, $y_4 = 10\ in$
  - $A_5 = 1.444\ ft^2$, $y_5 = 4\ in$

$$\bar{y} = \frac{\Sigma A_i y_i}{\Sigma A_i}$$

$\bar{y} = 25.3815$ in

$y_t = 32.62$ in

$y_b = 25.38$ in

- Moment of inertia:
  - $I_1 = \frac{1}{12} bh^3 = \frac{1}{12}(24)(4)^3$

    $I_1 = 128\ in^4$

    $I_1 + Ad^2 = 128 + (96.048)(30.62)^2$

    $= 90181.1\ in^4$

  - $I_2 = \frac{1}{36} bh^3 = \frac{1}{36}(8.75)(3)^3$

    $I_2 = 6.5625\ in^4$

    $I_2 + Ad^2 = (6.5625) + (13.1256)(27.62)^2$

    $= 10,019.6\ in^4$

  - $I_3 = \frac{1}{12} bh^3 = \frac{1}{12}(6.5)(46)^3$

    $I_3 = 52723.7\ in^4$

    $I_3 + Ad^2 = (52723.7) + (299)(5.62)^2$

    $= 62167.2\ in^4$

  - $I_4 = \frac{1}{36} bh^3 = \frac{1}{36}(9.75)(6)^3$

    $I_4 = 58.5\ in^4$

    $I_4 + Ad^2 = 58.5 + (29.25)(15.38)^2$

    $= 6977.42\ in^4$

  - $I_5 = \frac{1}{12} bh^3 = \frac{1}{12}(26)(8)^3$

    $I_5 = 1109.33\ in^4$

    $I_5 + Ad^2 = 1109.33 + (207.99)(21.38)^2$

    $= 96,184.1\ in^4$

$I_c = 90181.1 + (2)(10019.6) + 62167.2 + (2)(6977.42) + 96184.1$

$I_c = 282526\ in^4$

- Keehi Type IV Girder: (span 1)
  - Geometric Properties:

$A_c = 687.744\ in^2$ | $f_t = 804.984$ psi
$I_c = 282526\ in^4$ | $f_c = -3825$ psi
$C_t = 32.62$ in | $P_e = 793.4$ kips
$C_b = 25.38$ in | $S_t = 8661.13\ in^3$
$r^2 = 410.802\ in^2$ | $S_b = 11131.8\ in^3$
$e_c = 20.46$ in | $N = 7$ girders
$L = 1080$ in

I.) w_D = Self Weight + topping + future wearing surface

$= (4.776)(160) + (7)(6.5/12)(150) + 24(7)$

$w_D = 1453.15$ lb/ft

$w_D = 121.096$ lb/in

$\Rightarrow M_D = w_D L^2/8 = 1.76558 \times 10^7$ lb-in

II.) Tensile Limit: (unsubmerged)

$$w_{w_t} = \frac{8}{L^2}\left(M_D + S_t f_t - \left\{\frac{S_t P_e}{A_c}\right\}\left(\frac{e C_t}{r^2} - 1\right)\right)$$

$$= \frac{8}{(1080)^2}\left(M_D + (8661.13)(804.984) - \frac{(8661.13)(793.4\times10^3)}{687.744}\left(\frac{20.46\,(32.62)}{410.802} - 1\right)\right)$$

$$= (6.85871\times10^{-6})(M_D + 730851)$$

$w_{w_t} = 126.109$ lb/in

$F_w = 136.197$ kips (tension)

III.) Compression Limit: (unsubmerged)

$$w_{w_c} = \frac{8}{L^2}\left(M_D - (S_b f_c) - \frac{S_b P_e}{A_c}\left(1 + \frac{e C_b}{r^2}\right)\right)$$

$$= 8/L^2\,(M_D + 1.35043\times10^7)$$

$w_{w_c} = 213.718$ lb/in

$F_w = 230.82$ kips (compression)

IV) When submerged:

* Assume 50% air pocket
  ⇒ each girder will take half of the air pocket from each side

$w_D = 1453.15$ lb/ft $- 8.56767(64) - \underbrace{(13.125)(64)}_{\text{air pocket}}$

$w_D = 64.8191$ lb/ft

$w_D = 5.4016$ lb/in

$\Rightarrow M_D = 787552$ lb-in

V.) Tensile Limit: (submerged)

$w_{w_t} = (8/1080^2)(787552.00 + 730851)$

$w_w = 10.414$ lb/in

$F_w = 11.247$ kips (tension)

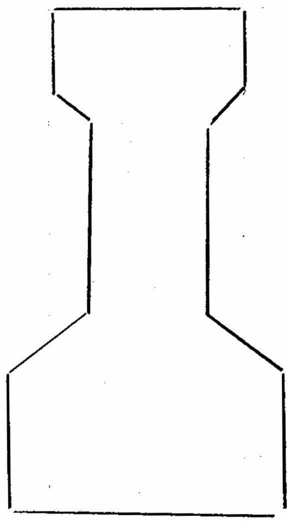
VI.) Compression Limit: (submerged)

$w_{w_c} = (8/1080^2)(787552.00 + 1.35\times10^7)$

$w_{w_c} = 98.0237$ lb/in

$F_w = 105.866$ kips (compression)

■ AASHTO Type III Girder : (span 2, 3, & 4)



Properties:

$f_c' = 4000 psi$

$P_e = 636.2$ kips

$e_c = 16.20$ in

○ Centroid:

- $A_1 = 0.778\ ft^2$
  $y_1 = 41.5 in$
- $A_2 = .1406\ ft^2$
  $y_2 = 36.5 in$
- $A_3 = 1.507\ ft^2$
  $y_3 = 22.5 in$
- $A_4 = .3906\ ft^2$
  $y_4 = 9.5 in$
- $A_5 = 1.0694\ ft^2$
  $y_5 = 3.5 in$

$$\bar{y} = \frac{\Sigma y_i A_i}{\Sigma A_i}$$

$\bar{y} = 20.2749 in$

$y_b = 20.275 in$

$y_t = 24.725 in$

○ Moment of Inertia:

- $I_1 = \frac{1}{12} bh^3 = \frac{1}{12}(16)(7)^3$
  $I_1 = 457.3\ in^4$
  $I_1 + Ad^2 = 457.3 + (112)(21.225) = 50927.8\ in^4$

- $I_2 = \frac{1}{36} bh^3 = \frac{1}{36}(4.5)^4 = 11.39\ in^4$
  $I_2 + Ad^2 = 11.39 + (10.123)(16.225)^2 = 2676.33\ in^4$

- $I_3 = \frac{1}{12} bh^3 = \frac{1}{12}(7)(21)^3 = 17378.1\ in^4$
  $I_3 + Ad^2 = (17378.1) + (217.008)(2.225)^2 = 18452.4\ in^4$

- $I_4 = \frac{1}{36} bh^3 = \frac{1}{36}(7.5)^4 = 87.89\ in^4$
  $I_4 + Ad^2 = 87.89 + 28.1232(10.775) = 390.917\ in^4$

- $I_5 = \frac{1}{12} bh^3 = \frac{1}{12}(22)(7)^3 = 628.833\ in^4$
  $I_5 + Ad^2 = 628.833 + (153.994)(16.775)^2 = 43962.7\ in^4$

$$I_c = 50927.8 + (2)(2676.33) + 18452.4 + (2)(390.917) + 43962.7$$

$$I_c = 119477\ in^4$$

- AASHTO Type III Girder: cont.
  - Geometric Properties:

$A_c = 559.44 \text{ in}^2$ — $f_t = 758.947$ psi
$I_c = 119477 \text{ in}^4$ — $f_c = -3400$ psi
$C_t = 24.725$ in — $P_e = 636.2$ kips
$C_b = 20.275$ in — $S_t = 4832.25 \text{ in}^3$
$r^2 = 213.566 \text{ in}^2$ — $S_b = 5892.84 \text{ in}^3$
$e_c = 16.20$ in
$L = 720$ in

I.) w_D = Self Weight + topping + wearing surface
$= (3.885)(150) + (7)(6.5/12)(150) + 24(7)$
$w_D = 1319.5 \text{ lb/ft}$
$w_D = 109.958 \text{ lb/in}$

$\Rightarrow M_D = w_D L^2/8 = 7.1253 \times 10^6$ lb-in

II.) Tensile Limit: (unsubmerged)

$$w_{wt} = \frac{8}{L^2}\left[M_D + S_t f_t - \left\{\frac{S_t P_e}{A_c}\right\}\left(\frac{e c_b}{r^2} - 1\right)\right]$$
$$= 8/L^2 \left[M_D + (-1.14374 \times 10^6)\right]$$

$w_{wt} = 8/720^2 \left[7.1253 \times 10^6 - 1.14374 \times 10^6\right]$
$w_{wt} = 92.308 \text{ lb/in}$
$F_w = 66.4617$ kips (tension)

III.) Compression Limit: (unsubmerged)

$$w_{wc} = \frac{8}{L^2}\left[M_D - (S_b f_c) - \left\{\frac{S_b P_e}{A_c}\right\}\left(1 + \frac{e c_b}{r^2}\right)\right]$$
$$= 8/L^2 \left[M_D + 3.02783 \times 10^6\right]$$
$$= 8/720^2 \left[7.1253 \times 10^6 + 3.02783 \times 10^6\right]$$

$w_{wc} = 156.684 \text{ lb/in}$
$F_w = 112.813$ kips (compression)

IV) When submerged:

$w_D = 1319.5 - (7.67667)(64) - 9.91(64)$ *50% air pocket
$w_D = 193.953 \text{ lb/ft}$
$w_D = 16.1628 \text{ lb/in}$
$\Rightarrow M_D = 1.04735 \times 10^6$ lb-in

IV.) Tensile Limit: (submerged)

$w_{wt} = 8/720^2 \left[1.04735 \times 10^6 - 1.14374 \times 10^6\right]$
$w_{wt} = -1.48755 \text{ lb/in}$
$F_w = -1.071$ kips (failure)

V.) Compression Limit: (submerged)

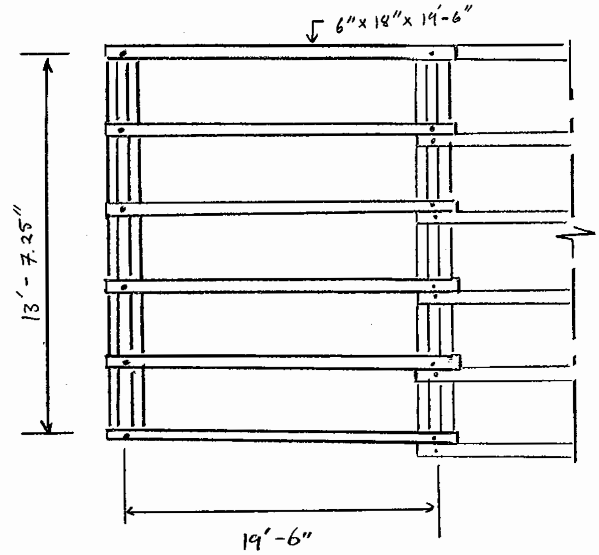
$w_{wc} = 8/720^2 \left[1.04735 \times 10^6 + 3.02783 \times 10^6\right]$
$w_{wc} = 62.8886 \text{ lb/in}$
$F_w = 45.2798$ kips (compression)

Summary of Results:

- **Keehi Type IV**:
  - Unsubmerged:
    - Tension: F_{ur} = 136.197 kips
    - Compression: F_{ur} = 230.82 kips
  - Submerged:
    - Tension: F_{ur} = 11.247 kips
    - Compression: F_{ur} = 105.866 kips
  - Loss once submerged:
    - Tension: 91.74%
    - Compression: 54.13%

- **AASHTO Type III**: (50% air pocket)
  - Unsubmerged:
    - Tension: F_{ur} = 66.4617 kips
    - Compression: F_{ur} = 112.813 kips
  - Submerged:
    - Tension: F_{ur} = -1.071 kips
    - Compression: F_{ur} = 45.2798 kips
  - Loss:
    - Tension: 101.61%
    - Compression: 54.13%

- **Buoyancy Calculations:**
  - Air Pocket Calculations:
    - Plan View: (Sheet 2)



Top View

- $A_1 = (19 + 6/12)(13 + 7.25/12) = 265.281\ ft^2$
- $A_{beam} = (19 + 6/12)(6/12) = 9.75\ ft^2$
- Air Void Area $= A_1 - 6(A_{beam})$
  $= (265.281) - 6(9.75)$
  $A_{AV} = 206.781\ ft^2$

* Note: There are 8 total 19'-6" x 13'-7.25" sections under the Makaha 3A Bridge

- Air Pocket Volume:
  - Total Area $= 8(A_{AV})$
    $= 8(206.781)$
    Total Area $= 1654.25\ ft^2$
  - Depth of pocket = 18"
  - Air Pocket Volume $= (18/12)(1654.25)$
    $= 2481.38\ ft^3$

$$\boxed{\text{Air Pocket} = 2481.38\ ft^3}$$

- Reduction of Air Pocket:
  * Note: Slope of bridge is ignored
  - $h = \underbrace{(6/12)}_{\text{Deck}} + \underbrace{(18/12)}_{\text{Girder Depth}} + \underbrace{(1/12)}_{\text{Asphalt}} = 2.083\ ft$
  - $P_2 = 14.7 + h(64/144)$
    $= 14.7 + (2.083)(64/144)$
    $P_2 = 15.6259$ psi
  - $V_2 = \frac{P_1 V_1}{P_2} = \frac{(14.7)(2481.38)}{(15.6259)}$
    $V_2 = 2334.34\ ft^3$

$\boxed{V_T = 2334.34\ ft^3}$ Compressed Air Pocket

Old Makaha No. 3A Bridge

Self Weight Calculations

γ = 31.2 lbs/cubic ft (G = 0.5 of water - Value for Hawaii Douglas Fir [12% moisture])

Timber:

Member:	No. Req.	Width (in)	Height (in)	Length (ft)	Volume (cubic ft)	Total Volume Per Item (cubic ft)
Sill	3	12	12	34	34.00	102.00
Post	5	12	12	18	18.00	90.00
Cap	3	12	12	34	34.00	102.00
Solid Bridging	5	4	18	16	8.00	40.00
Solid Bridging	5	4	18	10	5.00	25.00
Stringers	48	6	18	20	15.00	720.00
Decking (Floor)	566	8	6	26	8.67	4905.33
Decking (Floor)	1	1	6	26	1.08	1.08
Wheel Guard	4	8	8	20	8.89	35.56
Deck Starter	2	6	6	26	6.50	13.00
Wheel Guard	4	10	12	20	16.67	66.67
Railing	8	4	8	20	4.44	35.56
Railing	8	3	8	20	3.33	26.67
Railing Post	13	8	8	12	5.33	69.33
Sidewalk Joist Block	1	8	14	8	6.22	6.22
Sidewalk Joist Block	1	12	14	12	14.00	14.00
Guard Rail	1	3	8	18	3.00	3.00
Guard Posts	2	12	12	10	10.00	20.00
Guard Rail	1	10	12	18	15.00	15.00
Guard Rail	1	4	8	18	4.00	4.00
Cross Bridging	14	2	4	16	0.89	12.44
Sidewalk Planking	30	2	8	16	1.78	53.33
Bracing	12	3	10	20	4.17	50.00
Corner Blocking	1	2	6	10	0.83	0.83
Splice Block	6	6	18	3	2.25	13.50
Splice Block	3	8	18	3	3.00	9.00
Sidewalk End Timber	1	8	8	8	3.56	3.56

Railing Post Blocks	10	8	8	1.46	0.65	6.48	
Wall Plate	4	3	8	16	2.67	10.67	
					Total Volume of Timber	6454.23	cubic ft
					Total Weight of Timber	201372.02	lbs

Steel Repair Beams:

Member:	No. Req.	Cross Sectional Area (sq ft)	Length (ft)	Volume (cubic ft)	Total Volume (cubic ft)
W10x22	20	0.045	20	0.901	18.03

Member:	No. Req.	Nominal Weight (lb/ft)	Length (ft)	Weight (lbs)	Total Weight (lbs)
W10x22	20	22	20	440	8800

γ = 152 lbs/cubic ft (specific weight of asphalt)

Asphalt:

Member:	No. Req	Width (ft)	Thickness (in.)	Length (ft)	Volume (cubic ft)	Total Weight (lbs)
Road Topping	1	32.875	1	78.83	215.97	32827.51

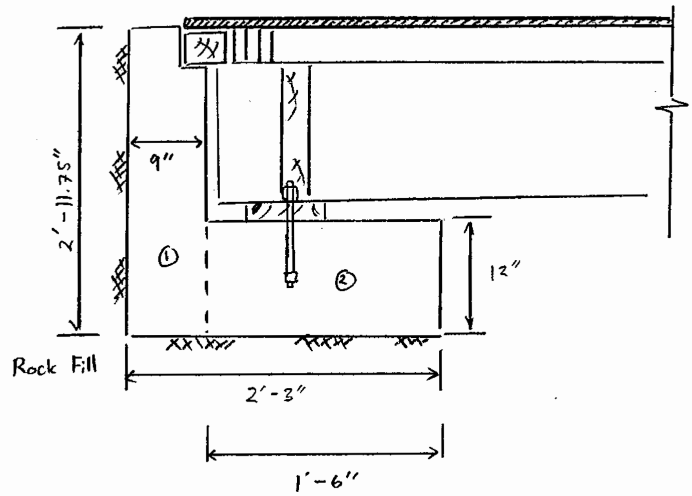
Bridge Total:

Member:	Volume (ft^3)	Weight (lbs)
Timber	6454.23	201372.02
Steel Repair Beams	18.03	8800
Asphalt	215.97	32827.51
Total	6688.23	242999.54

Pg. 3

▲ Self Weight Calculations: Cont.

○ Abutment Concrete: ($\gamma_{\text{Reinforced concrete}} = 150\ \text{lb}/\text{ft}^3$)



* Note: There are 10 - 3/4"ϕ × 18" bolts on each abutment. ∴ It is estimated that the friction keeping the concrete in place is less than the bolt capacities

$A_1 = (9/12)(2 + 11.75/12) = 2.23438\ \text{ft}^2$

$A_2 = (1 + 6/12)(12/12) = 1.5\ \text{ft}^2$

• $A_T = A_1 + A_2 = 3.73438\ \text{ft}^2$

○ Total Volume:

• Length = 32.875 ft

• Volume = $2(A_T)(L)$
= $2(3.73438)(32.875)$
Volume = $245.535\ \text{ft}^3$

○ Concrete Weight = $\forall \times \gamma_{RC}$
= $(245.535)(150)$
CW = 36830.3 lbs

▲ Total Self Weight:

○ From Pg. 2:

Self Weight = Timber + Steel + Asphalt + Concrete
= 242999.54 + 36830.3
Self Weight = 279830 lbs

- **Total Self Weight**:
  - From pg. 2, 3 & 4

$$\boxed{\text{Self Weight} = 279.83 \text{ kips}}$$

- **Buoyant Force**: ($\gamma_{seawater} = 64 \text{ lb/ft}^3$)
  - * Note: Assume bridge is submerged to the top of the deck

  - Submerged Volume = Bridge Volume + Air Pocket + Asphalt + Concrete Abutment
    = (Bridge − Railing − Sidewalk) + Air Pocket + Asphalt + Concrete Abutment
    = (6454.23 − 247.11) + 2334.34 + 215.97 + 245.535

    $S\forall = 9002.97 \text{ ft}^3$

  - Buoyant Force = $S\forall \times \gamma_{sw}$
    $= (9002.97)(64)$

    $$\boxed{BF = 576190 \text{ lbs}}$$

- **Residual Weight**:

  Residual Weight = SW − BF
  = 279830 − 576190

  $$\boxed{RW = -296360 \text{ lbs}}$$

Summary of Results:

- Self Weight = 279.8 kips
- Buoyant Force = 576.2 kips
- Residual Weight = −296.4 kips
- % Retained Weight = −106.0 %

∴ Bridge is Buoyant

Deck Capacity:

* Bridge deck is made of 2" x 6" x 26'-0" (S6S)
Douglas Fir wood, with 12% moisture content

⇒ From Wood Design Engineering Handbook

Compression ($\perp$) to grain = 760.0 lb/in² — where ($\perp$) = perpendicular
Shear/Tension ($\perp$) to grain = 350.0 lb/in²

o Moment Capacity:

I.) $f_b = M/Z$

II.) $Z = bh^2/6 = (26 \times 12)(6)^2/6$
$Z = 1872 \text{ in}^3$

III.) $M = f_b Z$
$= (760)(1872)$
$M = 1.42272 \times 10^6$ lb-in
$M = 118.56$ k-ft

IV.) $\varnothing M_n = 0.90(118.56 \text{ k-ft})$

$\varnothing M_n = 106.704$ k-ft

Moment Capacity:
$M_n = 118.56$ k-ft
$\varnothing M_n = 106.70$ k-ft

o Shear Capacity:

I.) $f_s = k(V/A)$

II.) $A = (26 \times 12)(6) = 1872 \text{ in}^2$

III.) For rectangular cross section:
⇒ $k = 3/2$

IV.) $V = f_s(A/k)$
$= (350)(1872)(2/3)$
$V = 436800$ lbs $= 436.8$ kips

IV.) $\varnothing V_n = 0.75(436.8) = 327.6$ kips

Shear Capacity:
$V_n = 436.8$ kips
$\varnothing V_n = 327.6$ kips

- **Vertical Resistance:** (Bolts in bridge bents)

Specific Gravity Douglas Fir $= 0.5\,\gamma_w = 31.2\ \text{lb/ft}^3$
$\Rightarrow G = 0.5$

o Ultimate Withdrawal Load: (Drift Bolt)
$d = 7/8$ in
$L = 6$ in

I.) $P = 6{,}600G^2DL$
$= 6{,}600(0.5)^2(7/8)(6)$
$P = 8662.5$ lbs/bolt

II.) 4 Bolts Per Bent (8 bolts total)

$P = 8(8662.5) = 69300$ lbs

$\boxed{\Rightarrow P = 69.3 \text{ kips}}$ Vertical Resistance

- **Lateral Resistance:** (bolts in bridge bents)
* It is assumed that the drift bolts behave similar to wood nails

I.) $P = KD^{3/2}$

$\Rightarrow$ For softwood $G = 0.48 - 0.52$
$K = 2{,}200$

II.) $P = KD^{3/2}$
$= (2{,}200)(7/8)^{3/2}$
$P = 1800.67$ lbs/Bolt

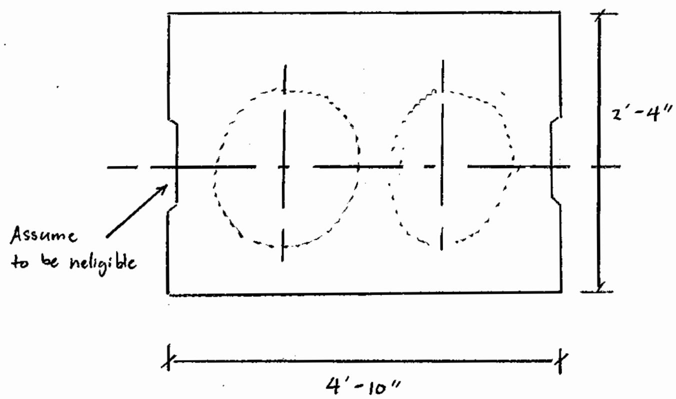
III.) For Entire Bridge:

$P = 8(1800.67) = 14405.4$ lbs

$\boxed{P = 14.4 \text{ kips}}$ Lateral Resistance

Buoyancy Calculations:

Hollow Core Planks:



2 - 16" dia void holes

$$A_1 = (4 + 10/12)(2 + 4/12) = 11.2778\ ft^2$$

$$A_2 = 2\left[\pi/4\,(16/12)^2\right] = 2.7925\ ft^2$$

- $A_T = A_1 - A_2 = 8.48525\ ft^2$

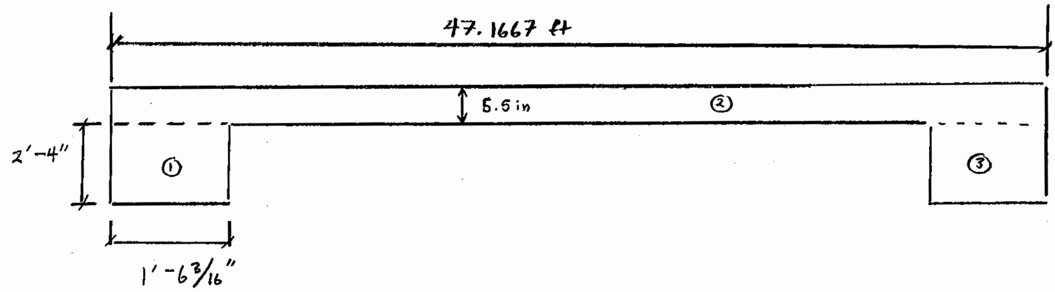
- 9 Hollow core plank
  $\therefore\ A_T = 9(8.48525) = 76.3673\ ft^2$

$$\boxed{A_T = 76.367\ ft^2}$$

Hollow Core Planks
L = 70 ft

▲ Self Weight: cont.

o Bridge Deck:



$A_1 = (1.515625)(2.333) = 3.53646\ ft^2$

$A_2 = (47.1667)(5.5/12) = 21.6181\ ft^2$

$A_3 = A_1 = 3.53646\ ft^2$

$\cdot\ A_T = A_1 + A_2 + A_3 = 28.691\ ft^2$

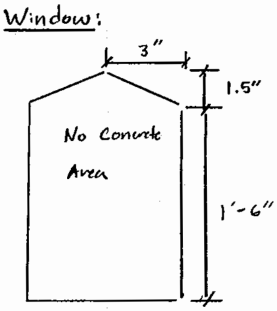
$\boxed{\Rightarrow A_T = 28.69\ ft^2}$

Deck Area

Length = 70 ft

o Railing:

- Window:



$A = [\frac{1}{2}(3)(1.5)](2) + [(6)(18)]$

$A = 112.5\ in^2 = 0.78125\ ft^2$

- Railing:

$A = (22.5)(32) = 720\ in^2 = 5\ ft^2$

- Posts:

$A = (2 + 9/12)(40/12) = 9.1667\ ft^2$

- One set of Railing:

Amount of Windows = $(7 \times 3) = 21$ (-)

Amount of Railing = $7 \times 3 = 21$

Amount of Posts = 1

$A_T = 21(5.00) - (21)(.78125) + 9.1667$

$A_T = 97.7604\ ft^2$

- Volume of all Railing:

$V_T = (97.7604)(4)(1.0)$

$V_T = 391.042\ ft^3$

$\boxed{V_T = 391.042\ ft^3}$

Total Volume of Railing

- **Buoyancy Calculations: cont.**

  - Self Weight:

    - Concrete Volume = [Planks + deck] (L) + Railing
      $= [76.367 + 28.691](70) + 391.042$
      $C\forall = 7745.12\ ft^3$

    - Self Weight = $C\forall \times \gamma_{RC}$
      $= (7745.12)(150)$
      $SW = 1.16177 \times 10^6$ lbs = 1161.77 kips

  - Buoyant Force:

    - $BF = S\forall \times \gamma_{sw}$
      = (Submerged Concrete + Air pocket) γ_w
      $= (7354.08 + 0)(64)$
      $BF = 470661$ lbs = 470.66 kips

    - Residual Weight = 691.11 kips

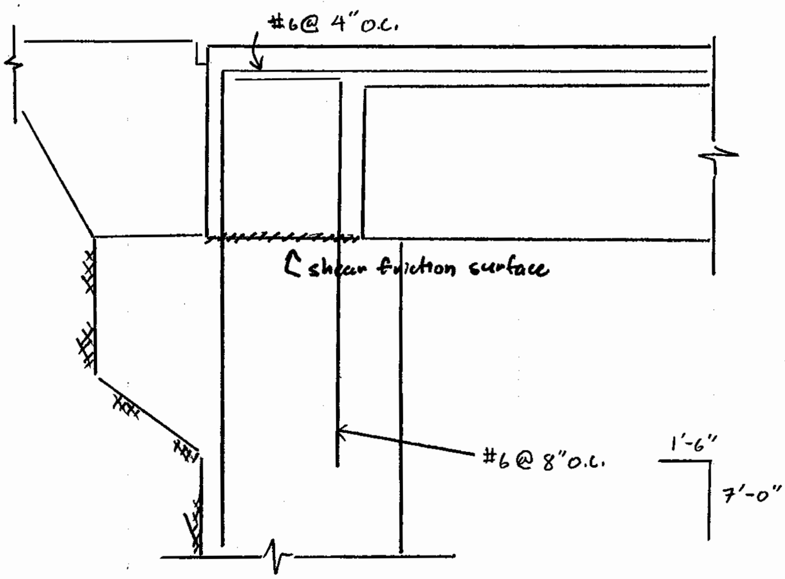
- **Summary of Results:**

  - Self Weight = 1161.8 kips
  - Buoyant Force = 470.66 kips
  - Residual Weight = 691.11 kips
  - % Retained Weight = 59.49%

∴ Bridge is not Bouyant

■ Lateral Resistance:

* Lateral resistance will be provided by the abutments, as the deck is tied into the abutment by #6 Hook bars.



$f_y = 60{,}000$ psi

$\mu = 0.8$

I.) Bridge Width = 46'-10" = 562"

II.) $562/8 = 70.25$ #6 (Hook Bar)

$562/4 = 140.5$ #6 (Bent Bar)

Total = 210 #6

III.) $A_{vf} = 210(.44) = 92.4\ in^2$

IV.) $V_n = A_{vf}\, f_y\, \mu$

$= (92.4)(60)(0.8)$

$V_n = 4435.2$ kips

⇒ For Both abutments

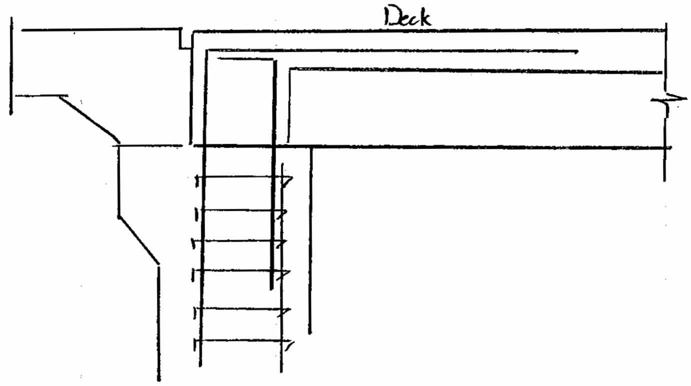
$V_n = (2)(4435.2)$

$$\boxed{V_n = 8870.4 \text{ kips}}$$

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 tieing the deck to the abutment will control the capacity of the bridge.



$f_c' = 4000$ psi
$f_y = 60{,}000$
#6 Bar:
$A_s = 0.44$ in²
Amount = 210/abutment

I) Reinforcement Conforms to ASTM A 615
⇒ Grade 60
∴ $f_y = 60{,}000$ psi
$f_b = 90{,}000$ psi

From ASTM 615:
#6 Bars:
Yield Strength = 26,400 lbs
Tensile Strength = 39,600 lbs

II) Vertical Capacity = 210 (Tensile)
= 210 (39.6)
Vertical Cap. = 8316 kips

⇒ For both abutments
$P_n = (8316)(2) = 16632$ kips

$\boxed{P_n = 16632 \text{ kips}}$

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 prestressing may cause the hollow cores to fail.

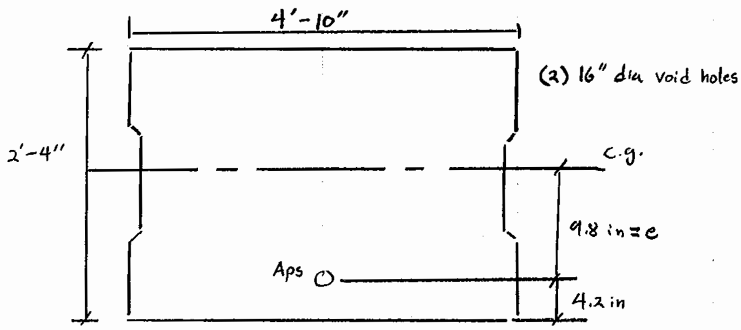
* Note: If the stresses in the concrete exceed:

$f_t = 12\sqrt{f_c'}$ (From PCI Handbook Section 1.3.1.2)

$= (12.0)(8000)^{1/2}$

$f_t = 1073.31\text{ psi}$

The bridge deck will fail.



- $f_c' = 8000$ psi
- $f_{pu} = 270{,}000$ psi
- (53) ½"Ø Prestress Strands:
  - $A_{ps} = 0.153\ in^2/\text{strand}$
  - $P_e = 1325$ kips/53 strands
  - Strands are not harped
  - Uncoated - low relaxation strands
- $E_c = 57{,}000\sqrt{f_c'} = 5.098\times10^6$ psi

○ Compute Geometric Properties:

* Assume rectangular section

I.) $I_g = \frac{1}{12}bh^3 = \frac{1}{12}(58)(28)^3$

$I_g = 106101\ in^4$

II.) $I_{circles} = \frac{\pi}{4}r^4$

$= \frac{\pi}{4}(8)^4$

$I_{circle} = 3216.99\ in^4$

III.) $I_c = I_g - 2I_{circle}$

$= 106101 - 2(3216.99)$

$I_c = 99667.4\ in^4$

IV.) $A_c = (58)(28) = 1624\ in^2$

$c = \frac{1}{2}h = 14\text{ in}$

$S = I_c/c = 7119.1\ in^3$

$r^2 = I_c/A_c = 61.3715\ in^2$

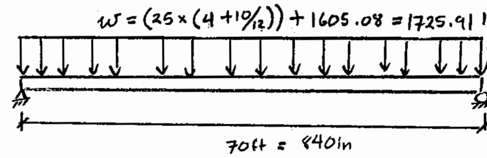
○ Compute Moments:

* During storm event assume no live loads

- From Plans:
  - Future wearing surface = 25 psf
- For one plank: (self weight)

$w_D = [(8.485)+(4+10/12)(5.5/12)](150) = 1605.\ \text{lb/ft}$

$w = (25\times(4+10/12)) + 1605.08 = 1725.91\ \text{lb/ft} = 143.8\ \text{lb/in}$



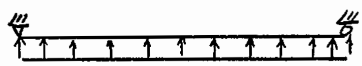
$M_{max} = \frac{wL^2}{8} = \frac{(143.8)(840)^2}{8}$

$M_{max} = 1.26855\times10^7\ \text{lb-in}$

■ 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.



w_w

* Note: The force is analyzed for one plank only

$$M_{wave} = \frac{w_w L^2}{8}$$

○ Compute stresses:

I) In negative bending: Top will be in tension $\therefore f_t = 12\sqrt{f_c'} = +1073.31$ psi (T)
Bot will be in compression $\therefore f_c = 0.85 f_c' = -6800$ psi (c)

II) $$f^t = \frac{-P_e}{A_c}\left(1 - \frac{ec}{r^2}\right) - \frac{M}{S} = f_t$$

$$-\frac{(1325\times10^3)}{(1624)}\left(1 - \frac{(9.8)(14)}{61.3715}\right) - \frac{M}{7119.1} = 1073.31$$

$$1008.08 - \frac{M}{7119.1} = 1073.31$$

$$M = -464379$$

III) $$M = M_{max} - \frac{w_w L^2}{8} = -464379 \text{ lb-in}$$

$$-\frac{w_w (840)^2}{8} = -464379 - 1.26855\times10^7$$

$$w_w = 149.092 \text{ lb/in}$$

∴ A distributed load of 149.09 lb/in will not fail a single plank

⇒ Each plank can resist:
$F_w = (149.092)(840) = 125237$ lbs

For 9 planks:
$F_w = 9(125237) = 1.12713\times10^6$ lbs

$$\boxed{F_w = 1127.13 \text{ kips}}$$

Capacity in Tension

■ Deck Capacity: Cont.

O Compute Stresses: cont.

IV.) $f_b = -\frac{P_e}{A_c}\left(1 + \frac{ec}{r^2}\right) + \frac{M_b}{S} = f_c$

$$-\frac{(1325\times10^3)}{(1624)}\left(1 + \frac{(48)(14)}{61.3715}\right) + \frac{M_b}{7119.1} = -6800$$

$$-2639.85 + M_b/7119.1 = -6800$$

$$M_b = -2.96165\times10^7 \text{ lb-in}$$

V.) For M_b:

Wave moment will be additive to prestressing

$\therefore\ M_b = M_{max} - \frac{w_w L^2}{8}$

$$\Rightarrow M_b = (1.26855\times10^7) - \frac{w_w L^2}{8} = -2.96165\times10^7$$

$$-\frac{w_w L^2}{8} = -4.2302\times10^7$$

$$w_w = \frac{(+4.2302\times10^7)(8)}{(840)^2}$$

$$w_w = 479.615 \text{ lb/in}$$

$\therefore$ A distributed load of 479.615 lb/in will not fail a single plank

$\Rightarrow$ Each plank can resist:

$$F_w = (479.615)(840) = 402876 \text{ lbs}$$

For 9 planks:

$$F_w = 9\,(402876) = 3.62589\times10^6 \text{ lbs}$$

$$\boxed{F_w = 3625.89 \text{ kips}}$$

Capacity in Compression

- **Deck capacity: cont.**

  - Buoyant Force Addition:

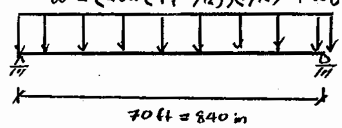
    * Note: If the bridge becomes submerged the self weight will be reduced
    ⇒ Assume bridge is submerged to top of the deck.

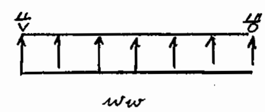
I.) $w_D = [(8.485) + (4 + 10/12)(5.5/12)](150 - 64)$

$w_D = 920 \text{ lb/ft} = 76.6853 \text{ lb/in}$ (self weight for one plank)

II.)

$w = (25 \times (4 + 10/12))(1/12) + w_D = 86.7361 \text{ lb/in}$





$M_w = \dfrac{w_w L^2}{8}$

$$M_{max} = \frac{wL^2}{8} = \frac{(86.7361)(840)^2}{8}$$

$$M_{max} = 7.65012 \times 10^6 \text{ lb-in}$$

III.) $f^t = \dfrac{-P_c}{A_c}\left(1 - \dfrac{ec}{r^2}\right) - \dfrac{M_t}{S} = f_t$

$$\frac{-(1325 \times 10^3)}{(1624)}\left(1 - \frac{(9.8)(14)}{61.3715}\right) - \frac{M_t}{7119.1} = 1073.31$$

$$1008.08 - \frac{M_t}{7119.1} = 1073.31$$

$$M_t = -464379 \text{ lb-in}$$

$$\Rightarrow M_{max} - \frac{w_w L^2}{8} = -464379$$

$$\frac{w_w L^2}{8} = 7.16063 \times 10^6 + 464379$$

$$w_w = 86.4513 \text{ lb/in}$$

IV.) Force resisted by 9 planks:

$$F_w = (9)(86.4513)(840) = 653572 \text{ lbs}$$

$$\boxed{F_w = 653.572 \text{ kips}}$$

Capacity in Tension

- Deck Capacity: cont.

  - Buoyant Force Addition: cont.

V.) $f_b = \frac{-P_e}{A_c}\left(1+\frac{ec}{r^2}\right) + \frac{M_b}{S} = f_c$

$M_b = -2.96165\times10^7$ lb-in (same as before)

$\Rightarrow M_b = 7.65012\times10^6 - \frac{w_w L^2}{8} = -2.96165\times10^7$

$-\frac{w_w L^2}{8} = -3.3815\times10^7$

$w_w = 383.391$ lb/in

$\Rightarrow$ Force Resisted by 9 planks:

$F_w = (9)(383.391\times840) = 2.89844\times10^6$ lbs

$\boxed{F_w = 2898.44 \text{ kips}}$

Capacity in compression

■ Deck Capacity:

○ Deflection: (caused by estimated wave loads)

* From plans sht S11.1

$$\text{Camber (upward)} = \text{camber at erection} - \Delta d \text{ (deck slab)}$$
$$= 2.79 - 0.64$$
$$\text{Camber (upward)} = 2.15 \text{ in}$$

- To calculate upward deflection due to wave forces use:

$$\delta_w = \frac{5 w_w \ell^4}{384 E_c I_e}$$

* Note: The section is cracked as $f_r = 7.5\sqrt{f_c'}$ has been exceeded

∴ $I_{effective}$ must be used

I.) $d_p = \underbrace{(5.5)}_{deck} + (28 - 4.2) = 29.3$ in

II.) $\rho_p = \dfrac{A_{ps}}{b\, d_p} = \dfrac{(0.153 \times 53)}{(58)(29.3)} = .004772$

III.) $E_c = 57{,}000\sqrt{f_c'} = 57{,}000(8000)^{1/2}$

$E_c = 5.09823 \times 10^6$ psi

$E_{ps} = 28.5 \times 10^6$ psi

$$n_p = \frac{E_{ps}}{E_c} = \frac{(28.5 \times 10^6)}{5.09823 \times 10^6} = 5.59017$$

IV.) $I_{cr} = n_p A_{ps} d_p^2 (1 - 1.6\sqrt{n_p \rho_p})$

$= (5.59017)(8.109)(29.3)^2(1 - 1.6(5.59017 \times .004772)^{1/2})$

$I_{cr} = 49085.4$ in^4

V.) No live load:

∴ $M_a = 0$

$$I_e = (M_{cr}/M_a)^3 I_g + [1 - (M_{cr}/M_a)^3] I_{cr}$$
$$I_e = I_{cr} = 49085.4 \text{ in}^4 < I_g \ \checkmark$$

- Deflection:

	Deflection:	ℓ (in)	δ_w (in)	Existing Camber (in)	Total (in)	
Buoyant Force Not Included	① $w_w = 149.092$ lb/in	840	3.86223	2.15	6.012	> $\ell/800$* (N.G.)
	② $w_w = 479.615$ lb/in	840	12.4244	2.15	14.57	> $\ell/800$ (N.G.)
Buoyant Force Included	③ $w_w = 86.4513$ lb/in	840	2.23952	2.29	4.530	> $\ell/800$ (N.G.)
	④ $w_w = 383.391$ lb/in	840	9.93176	2.29	12.22	> $\ell/800$ (N.G.)

* AASHTO Max Permissible Deflection

- **Summary of Results:**

  - Unsubmerged Case: (i.e. no buoyant force)

    $F_w = 1127.13$ kips (total capacity) * Weak in tension

    $w_w = 149.09$ lb/in (Plank distributed load capacity)

  - Submerged Case: (i.e. buoyant force included)

    $F_w = 653.57$ kips (total capacity) * Weak in tension

    $w_w = 86.45$ lb/in (Plank distributed load capacity)

  ⇒ Reduction in capacity once submerged:

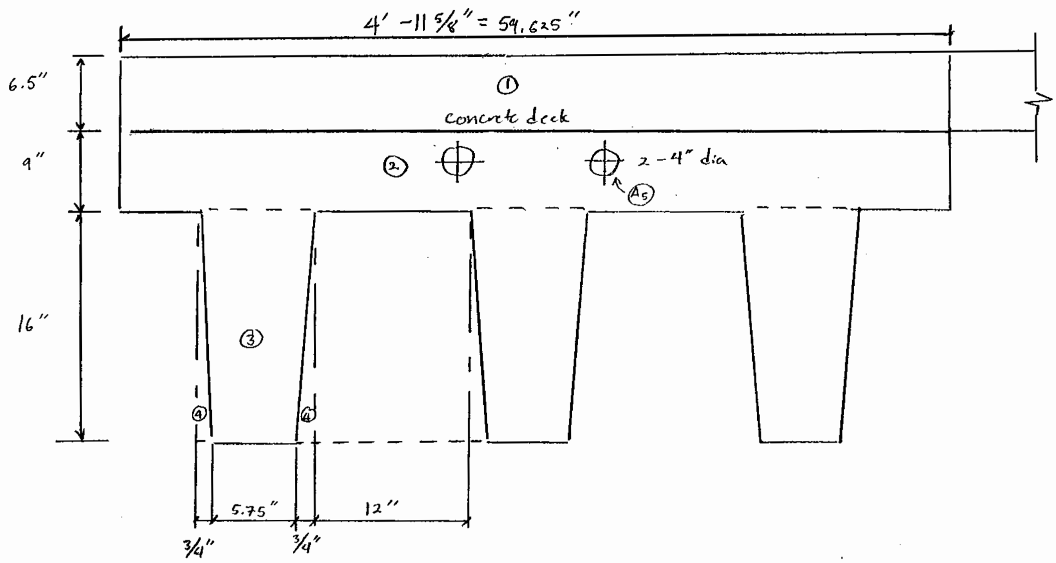
  Tension = 42.01%

  Compession = 20.06%

■ Buoyancy Calculations:

▲ Self Weight:

○ Trideck A:



Concrete:

$A_1 = (59.625)(6.5)(1/144) = 2.69141 \text{ ft}^2$

$A_5 = 2\left[(\pi/4)(4/12)^2\right] = .17453 \text{ ft}^2$ (-)

$A_2 = (59.625/12)(9/12) = 3.72656 \text{ ft}^2$

$A_3 = (7.25/12)(16/12) = 0.80556 \text{ ft}^2$

$A_4 = 2\left[\tfrac{1}{2}(3/4)(16)(1/144)\right] = .0833 \text{ ft}^2$ (-)

- $A_T = A_1 + A_2 + 3A_3 - 3A_4 - A_5$
  $= 2.69141 + 3.72656 + 3(.80556) - 3(.0833) - .17453$
  $A_T = 8.75917 \text{ ft}^2$

$\boxed{A_T = 8.759 \text{ ft}^2}$ Trideck A Concrete

Air Pocket:

$A_a = (12/12)(16/12) + .08333$

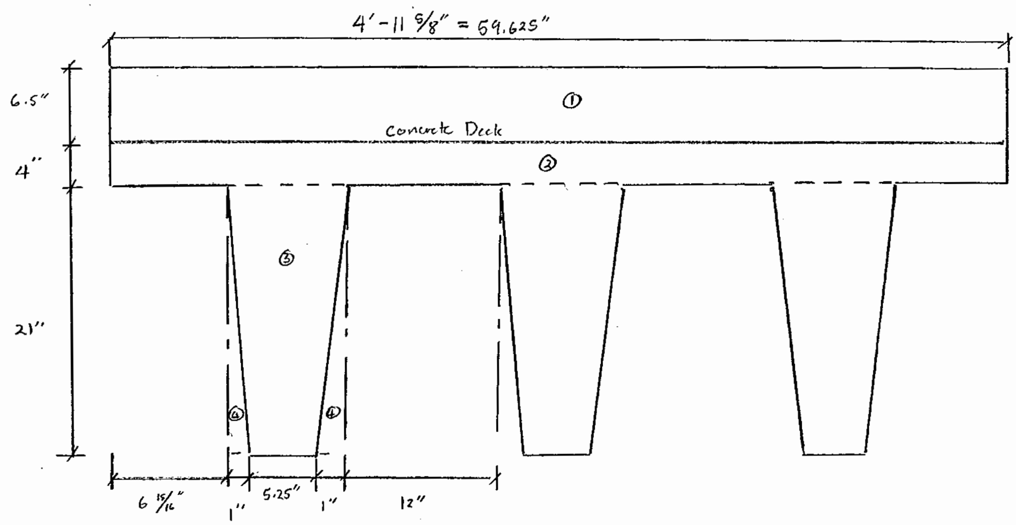
$A_a = 1.41667 \text{ ft}^2$

$A_T = 2A_a = 2.833 \text{ ft}^2$

$\boxed{A_T = 2.833 \text{ ft}^2}$ Trideck A Air Pocket

▲ Self Weight: cont.

○ Trideck B, C, & D:



Concrete:

$A_1 = (59.625/12)(6.5/12) = 2.69141\ ft^2$

$A_2 = (59.625/12)(4/12) = 1.65625\ ft^2$

$A_3 = (7.25/12)(21/12) = 1.05729\ ft^2$

$A_4 = 2[(1/2)(1/12)(21/12)] = 0.14583\ ft^2\ (-)$

• $A_T = A_1 + A_2 + 3A_3 - 3A_4$
$= (2.69) + (1.656) + 3(1.057) - 3(.146)$
$A_T = 7.08204\ ft^2$

$\boxed{A_T = 7.082\ ft^2}$ Trideck B, C, D Concrete

Air Pocket:

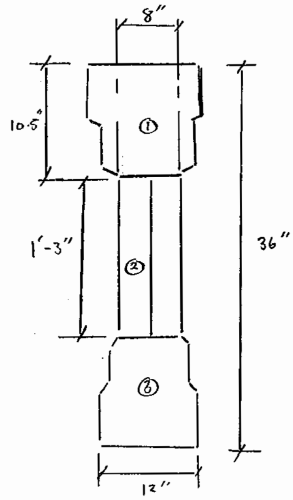
$A_a = (13/12)(21/12) + 2[(1/2)(1/12)(21/12)]$
$A_a = 1.89583\ ft^2$

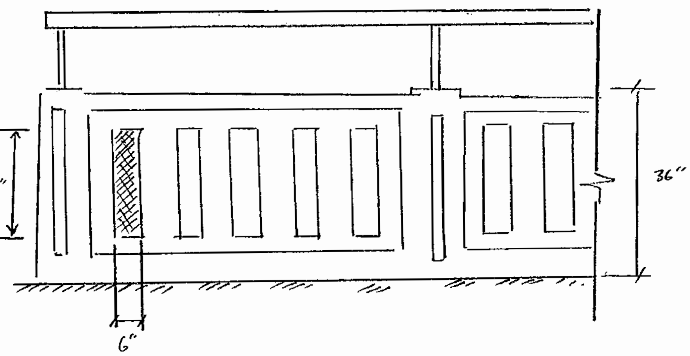
$A_T = 2A_a = 3.79166\ ft^2$

$\boxed{A_T = 3.792\ ft^2}$ Trideck B, C, D Air Pocket

▲ <u>Self Weight</u>: cont.

o <u>Railing</u>:





* The Area of the railing is estimated from sheet S5.3

$$A_1 \cong (12)(10.5)(1/144) = 0.875\ ft^2$$

$$A_2 \cong (15)(8.0)(1/144) = 0.833\ ft^2$$

$$A_3 \cong (12)(10.5)(1/144) = 0.875\ ft^2$$

$$\cdot\ A_T = A_1 + A_2 + A_3 = 2.583\ ft^2$$

◆ <u>Missing Volume</u>: (shaded area)

$$\forall_m = (6)(12+3)(8) = 720 in^3 = 0.4167\ ft^3$$

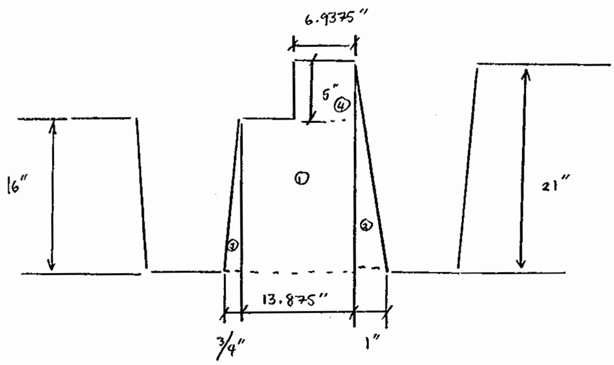
◆ <u>Railing Volume</u>:

$$\forall_R = (2.583)(170) - (0.4167)(90)$$

$$\boxed{\forall_R = 401.667\ ft^3}$$

▲ Air Pocket:

o Between Trideck A & B:



$A_1 = (13.875/12)(16/12) = 1.54167 \ ft^2$

$A_2 = 1/2\,(1/12)(21/12) = .072917 \ ft^2$

$A_3 = 1/2\,(3/4 \times 1/12)(16/12) = .006944 \ ft^2$

$A_4 = (6.9375/12)(5/12) = .240885 \ ft^2$

• $A_T = \sum_{i=1}^{4} A_i = 1.86241 \ ft^2$

$\boxed{A_T = 1.862 \ ft^2}$

o Between Tridecks B, C, & D:

$A = (13.875)(21)(1/144) + 2\left[(1/2)(1/12)(21/12)\right] = 2.16927 \ ft^2$

$\boxed{A_T = 2.169 \ ft^2}$

▲ Compression of Air Pocket:

o For all Pockets: (Assume submerged to top of deck)

• $h = 25 + 6.5 + \underbrace{2}_{\text{AC Pavement}} = 33.5 \text{ in}$

$h = 2.79 \ ft$

• $P_2 = P_1 + h\,(64/144)$

$= (14.7) + 2.79\,(64/144)$

$P_2 = 15.9407 \ psi$

• $P_1/P_2 = 14.7/15.94 = 0.92216$

o Between B, C, & D:

$A = (2.169)(.92216)$

$\boxed{\Rightarrow A = 2.00043 \ ft^2}$

o Trideck A:

$A = (2.833)(0.92216)$

$\boxed{\Rightarrow A = 2.6128 \ ft^2}$

o Trideck B, C, D:

$A = (3.792)(0.92216)$

$\boxed{\Rightarrow A = 3.49654 \ ft^2}$

o Between A & B:

$A = (1.862)(.92216)$

$\boxed{\Rightarrow A = 1.71745 \ ft^2}$

▲ Self Weight: (γ_{RC} = 160 pcf)

$$\text{Concrete Volume} = (\text{Trideck A} + 9\,\text{Trideck B, C, D})(170) + 3\,\text{Railing}$$
$$= (8.759 + (9)(7.082))(170) + (3)(401.667)$$
$$C\forall = 13529.5 \text{ ft}^3$$

$$\text{AC Volume} = (3/12)(40)(170) = 1133.33 \text{ ft}^3$$

$$\text{Self Weight} = C\forall \times \gamma_{RC} + AC \times \gamma_{AC}$$
$$= (13529.5)(160) + (1133.33)(152)$$
$$\boxed{\text{Self Weight} = 2.33699 \times 10^6 \text{ lbs}}$$

▲ Buoyant Force:

✱ Assume submerged to top of deck

$$\text{Submerged Volume} = \text{Concrete} - \text{Railing} + \text{AC Pavement} + \text{Air Pocket}$$
$$= (13529.5 - (3)(401.667)) + (1133.33) + [(1)(2.6128) + 9(3.495654) + (1)(1.71745) + 8(2.0)](170)$$
$$S\forall = 22264.3 \text{ ft}^3$$

$$\text{Buoyant Force} = S\forall \times \gamma_{sw} = (22264.3)(64)$$

$$\boxed{BF = 1.42491 \times 10^6 \text{ lbs}}$$

▲ Residual Weight:

$$RW = SW - BF$$
$$= (2.33699 \times 10^6 - 1.42491 \times 10^6)$$
$$\boxed{RW = 912{,}076 \text{ lbs}}$$

% 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

Trideck A Deck Capacity:

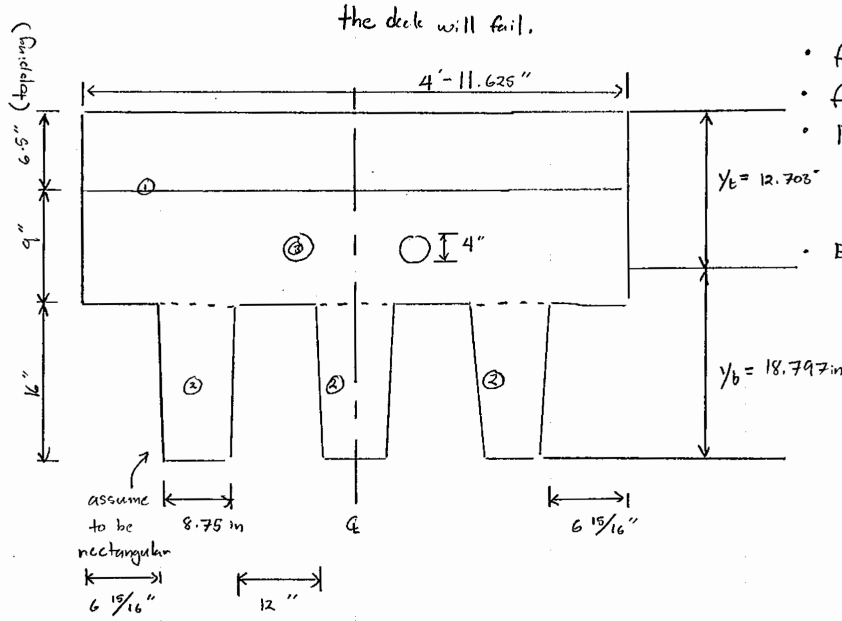
* Note: If the stresses in the concrete exceed:

$$f_t = 12\sqrt{f_c'}$$
$$= 12(9000)^{1/2}$$
$$f_t = 1138.42 \text{ psi (Tension)}$$

or

$$f_c = 0.85 f_c'$$
$$= 0.85(9000)$$
$$f_c = 7650 \text{ psi (compression)}$$

the deck will fail.



- $f_c' = 9000$ psi
- $f_{pu} = 270{,}000$ psi
- $P_e = 738$ kips
  ⇒ Span 2
  Strands are harped low relaxation strands
- $E_c = 57{,}000\sqrt{f_c'} = 5.40749 \times 10^6$ psi

Compute Geometric Properties:

▲ Moment of inertia:

I) Compute Center of gravity:

- $A_1 = (59.625)(15.5) = 924.188 \text{ in}^2$
  $y_1 = 23.75$ in
- $A_2 = (8.75)(16)(3) = 420 \text{ in}^2$
  $y_2 = 8$ in
- $A_3 = 2(\pi/4)(4)^2 = 25.1327 \text{ in}^2$
  $y_3 = 20.5$ in

- $$y_c = \frac{(924.188)(23.75) + (420)(8) - (25.14)(20.5)}{924.188 + 420 - 25.1327}$$
  $y_c = 18.797$ in (from bottom)

II) $I_1 = \frac{1}{12} bh^3 = \frac{1}{12}(59.625)(15.5)^3 = 18503 \text{ in}^4$

III) $I_2 = \frac{1}{12} bh^3 = \frac{1}{12}(8.75)(16)^3 = 2986.67 \text{ in}^4$

IV) $I_3 = \frac{\pi}{4} r^4 = (\frac{\pi}{4})(2)^4 = 12.5664 \text{ in}^4$

V) for ①: $I_1 + Ad^2$
$= (18503 + (924.188)(12.703 - 7.75)^2)$
$= 41175.4 \text{ in}^4$

for ②: $I_2 + Ad^2$
$= (2986.67) + (140)(18.797 - 8)^2$
$= 19307.2 \text{ in}^4$

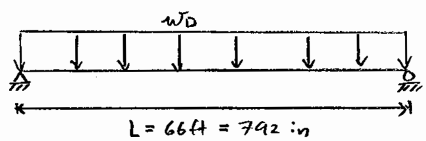
for ③: $I_3 + Ad^2$
$= (12.5664) + (12.5664)(20.5 - 18.797)^2$
$= 49.0116 \text{ in}^4$

VI) $I_c = ① + (3)② - (2)③$
$= 41175.4 + (3)(19307.2) - (2)(49.0116)$

$I_c = 98999 \text{ in}^4$
$A_c = 1319.06 \text{ in}^2$
$y_t = 12.703$ in $S^t = 7793.36 \text{ in}^3$
$y_b = 18.797$ in $S_b = 5266.74 \text{ in}^3$
$r^2 = 75.0529 \text{ in}^2$

- **Trideck A Capacity:**
  - Compute Moments:
    - * During storm event assume no live loads
    - From plans: Future wearing surface = 25 psf = 124.19 lb/ft (for one deck)
    - Dead load = (8.759)(160) = 1401.44 lb/ft

$$\text{Total } w_D = 1525.66 \text{ lb/ft} = 127.138 \text{ lb/in}$$



$$M_{max} = \frac{wL^2}{8} = \frac{(127.138)(792)^2}{8}$$

$$M_{max} = 9.96865 \times 10^6 \text{ lb-in}$$

- Wave induced moment:
  - * Assume that wave will produce a distributed load on entire deck of bridge

$$M_{wave} = w_w L^2/8 = 78408\, w_w$$

- Compute Stresses:
  - I.) In negative bending: Top will be in tension $\le f_t = +1138.42$ psi
    Bot will be in compression $\le f_c = -7650$ psi
  - II.) At center:
    $e_c = 18.65 - 5.86 = 12.797$ in
  - III.) $f_t = -\frac{Pe}{A_c}\left(1 - \frac{ec^t}{r^2}\right) - \frac{M_T}{S_t} \le f_t$

$$= \frac{-738{,}000}{(1319.06)}\left(1 - \frac{(12.797)(12.703)}{75.0529}\right) - \frac{M_T}{7793.36} = 1138.42$$

$$+5.08386 \times 10^6 - M_T = 8.87212 \times 10^6$$

$$M_T = -3.78825 \times 10^6$$

$$M_{max} - 78408\, w_w = -3.78825 \times 10^6$$

$$w_w = \frac{M_{max} + 3.78825 \times 10^6}{78408}$$

$$w_w = 175.453 \text{ lb/in}$$

$\therefore$ Trideck A can resist: (in tension)

$$F_w = 175.453\,(792)\,(1/1000)$$

$$F_w = 138.959 \text{ kips}$$

■ Trideck A Deck Capacity: cont.

o Compute Stresses: cont.

V.) $f_b = \dfrac{-P_e}{A_c}\left(1 + \dfrac{e c_b}{r^2}\right) + \dfrac{M}{S_b} \le 0.85 f_c'$

$= \dfrac{-(738{,}000)}{(1319.06)}\left(1 + \dfrac{(12.793)(18.797)}{75.0629}\right) + \dfrac{M}{5266.74} = -7650$

$-2352.66 + M/5266.74 = -7650$

$M = -2.78997 \times 10^7$

VI.) $M_{max} - 78408\, W_w = -2.78997 \times 10^7$

$9.96865 \times 10^6 - 78408\, W_w = -2.78997 \times 10^7$

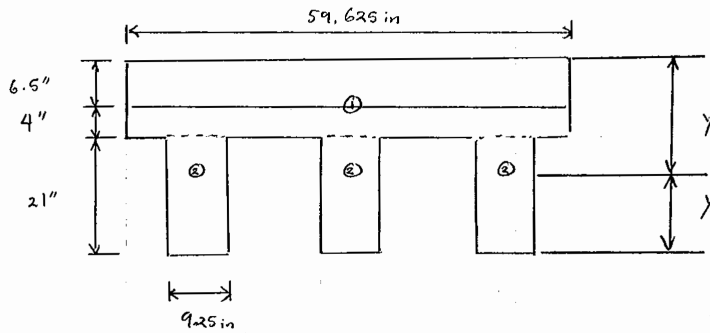
$W_w = 482.965$ lb/in

∴ The trideck can resist: (in compression)

$F_w = (482.965)(792) = 382509$ lbs

$F_w = 382.51$ kips

■ Trideck B, & C Deck Capacity:



- f_c' = 9000 psi
- f_{pu} = 270 ksi
- P_e = 703 kips

o Compute Geometric Properties:

I.) Compute COG:

- $A_1 = (59.625)(10.5) = 626.063\ in^2$

  $y_1 = 26.25\ in$
- $A_2 = (3)[(9.25)(21)] = 582.75\ in^2$

  $y_2 = 10.5\ in$
- $Y_c = \dfrac{(626.063)(26.25) + (582.75)(10.5)}{626.063 + 582.75}$

$Y_c = 18.6572\ in$

II.) $I_1 = \frac{1}{12} bh^3 = \frac{1}{12}(59.625)(10.5)^3$

$I_1 = 5751.95\ in^4$

III.) $I_2 = \frac{1}{12} bh^3 = \frac{1}{12}(9.25)(21)^3$

$I_2 = 7138.69\ in^4$

IV.) for ①: $I_1 + Ad^2$

$= (5751.95) + (626.063)(12.8428 - 5.25)^2$

$= 41844.9\ in^4$

for ②: $I_2 + Ad^2$

$= (7138.69) + (194.25)(18.6572 - 10.5)^2$

$= 20064.1\ in^4$

V.) $I_c = ① + (3)②$

$= 41844.9 + (3)(20064.1)$

$I_c = 102037\ in^4$

$I_c = 102037\ in^4$

$A_c = 1208.81\ in^2$

$y^t = 12.8428$ in　　S^t = 79[illegible]8 in^3

$Y_b = 18.6572$ in　　$S_b = 5469.04\ in^3$

$r^2 = 84.411\ in^2$

- **Trideck B & C Deck Capacity: Cont.**
  - Compute Moment:
    - $w_D = (7.082)(160) + 124.219 = 1257.34 \text{ lb/ft}$

      $w_D = 104.778 \text{ lb/in}$

$$M_D = w_D L^2/8 = (104.778)(792)^2/8$$

$$M_D = 8.21543 \times 10^6 \text{ lb-in}$$

  - Tensile Limit:
    - I) At center:

      $e_c = 18.6572 - 5.13 = 13.5272 \text{ in}$

    - II) Using equations developed during this thesis:

$$W_{wt} = \frac{8}{L^2}\left(M_D + (S_t f_t) - \left(\frac{S_t P_e}{A_c}\right)\left(\frac{e c_t}{r^2} - 1\right)\right)$$

$$= \frac{8}{(792)^2}\left((8.21543\times10^6) + (7945.08 \times 1138.42) - \frac{(7945.08 \times 703000)}{1208.81}\left\{\frac{(13.5272)(12.84)}{84.411} - 1\right\}\right)$$

$$W_{wt} = 157.806 \text{ lb/in}$$

$$F_w = 124.98 \text{ kips (tension)}$$

  - Compression Limit:

$$W_{wc} = \frac{8}{L^2}\left(M_D - (S_b f_c) - \frac{S_b P_e}{A_c}\left(1 + \frac{e c_b}{r^2}\right)\right)$$

$$= \frac{8}{(792)^2}\left(8.21543\times10^6 - (5469.04)(-7650) - \frac{(5469.04 \times 703000)}{1208.81}\left(1 + \frac{(13.5272)(18.6572)}{84.411}\right)\right)$$

$$w_{wc} = 476.525 \text{ lb/in}$$

$$F_w = 377.4 \text{ kips (compression)}$$

- **Trideck D Deck Capacity:**
  - I) Geometric Properties:

    $I_c = 102037 \text{ in}^4$

    $A_c = 1208.81 \text{ in}^2$

    $y_t = 12.8428 \text{ in}$ $\quad S_t = 7945.08 \text{ in}^3$

    $y_b = 18.6572 \text{ in}$ $\quad S_b = 5469.04 \text{ in}^3$

    $r^2 = 84.411 \text{ in}^2$

    $P_e = 714000 \text{ lbs}$

    $e_c = 13.5272 \text{ in}$

  - II) $M_{max} = 8.21545 \times 10^6 \text{ lb-in}$

  - III) Tensile Limit:

$$w_{wt} = \frac{8}{L^2}\left(M_D + (S_t f_t) - \frac{S_t P_e}{A_c}\left(\frac{e c_t}{r^2} - 1\right)\right)$$

$$= \frac{8}{(792)^2}\left[8.21543\times10^6 + (7945.08 \times 1138.42) - \frac{(7945.08)(714000)}{1208.81}\left(\frac{13.5272(12.8428)}{84.411} - 1\right)\right]$$

$$w_{wt} = 156.804 \text{ lb/in} \Rightarrow F_w = 124.189 \text{ kips}$$

■ Trideck D Capacity : cont.

IV.) Compression Limit:

$$w_{wc} = \frac{8}{L^2}\left[M_D - (S_b f_c) - \frac{S_b P_e}{A_c}\left(1 + \frac{e c_b}{r^2}\right)\right]$$

$$= \frac{8}{(792)^2}\left\{(8.21543\times10^6) - (5469.04)(-7650) - \frac{(5469.04)(714000)}{1208.81}\left(1 + \frac{13.5272(18.6572)}{84.411}\right)\right\}$$

$w_{wc} = 8/792^2\,(8.21543\times10^6 + 2.89494\times10^7)$

$w_{wc} = 473.992$ lb/in

$F_w = 375.402$ kips (compression)

■ Submerged Case:

* It is assumed bridge is submerged to top of the deck
  ∴ Buoyant Force must be considered

○ Trideck A:

I.) w_D = Dead loads − Buoyant Force
$= (124.219) + (8.759\times160) - (8.759 + 2.6128)(64)$

$w_D = 797.864$ lb/ft

$w_D = 66.4887$ lb/in

⇒ $M_D = 5.21324\times10^6$ lb-in

II.) $w_{wt} = \frac{8}{L^2}(M_D + 3.78825\times10^6)$

$w_{wt} = 114.803$ lb/in

$F_w = 90.92$ kips (tension)

III.) $w_{wc} = \frac{8}{L^2}(M_D + 2.78997\times10^7)$

$w_{wc} = 422.316$ lb/in

$F_w = 334.47$ kips (compression)

○ Trideck B & C:

I.) $w_D = 48.3594$ lb/in

$M_D = 3.79176\times10^6$ lb-in

II.) $w_{wt} = \frac{8}{L^2}(M_D + 4.15784\times10^6)$

$w_{wt} = 101.388$ lb/in

$F_w = 80.299$ kips (tension)

III.) $w_{wc} = \frac{8}{L^2}(M_D + 2.91479\times10^7)$

$w_{wc} = 420.106$ lb/in

$F_w = 332.724$ kips (compression)

- **Submerged Case: cont.**
  - Trideck D:

I.) $M_D = 3.79176 \times 10^6$ lb-in (Same as trideck B & C)

II.) $w_{wt} = \frac{8}{L^2}(M_D + 4.07927 \times 10^6)$

$= \frac{8}{792^2}(3.79176 \times 10^6 + 4.07927 \times 10^6)$

$w_{wt} = 100.385$ lb/in

$F_w = 79.5$ kips (tension)

III.) $w_{wc} = \frac{8}{L^2}(M_D + 2.89494 \times 10^7)$

$w_{wc} = 417.574$ lb/in

$F_w = 330.718$ kips (Compression)

- **Bridge Deck Capacity: (span #2)**
  - Unsubmerged Case:

Tension: $F_w = (1)A + (1)B + (7)C + (1)D$

$= 138.959 + 124.98 + (7)(124.98) + (124.189)$

$F_w = 1262.99$ kips

Compression: $F_w = 3777.11$ kips

  - Submerged Case:

Tension: $F_w = 812.812$ kips

Compression: $F_w = 3326.98$ kips

- **Summary of Results:**
  - Unsubmerged:

Tension: $F_w = 1262.99$ kips

Compression: $F_w = 3777.11$ kips

  - Submerged

Tension: $F_w = 812.812$ kips

Compression: $F_w = 3326.98$ kips

  - Loss once submerged:

Tension $= 35.64\%$

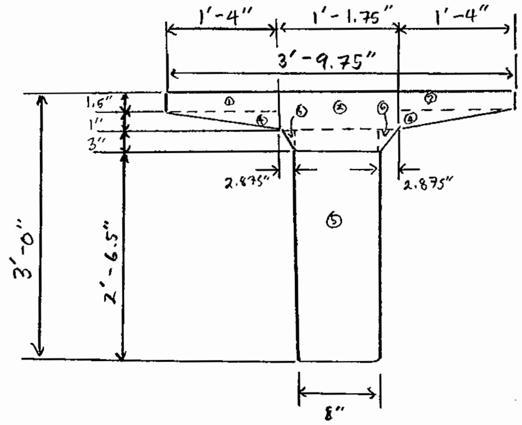
Compression $= 11.92\%$

Calculations By: Daniel Lum

Buoyancy Calculations:

Typical Girder Section: Precast Tee Girder (Widened Section)

- Sheet 5 of widening



$A_1 = (1+4/12)(1.5/12) = 0.1667\ ft^2$

$A_2 = (1+1.75/12)(2.5/12) = 0.2387\ ft^2$

$A_3 = (1+4/12)(1.5/12) = 0.1667\ ft^2$

$A_4 = 2[1/2(16/12)(1/12)] = 0.111\ ft^2$

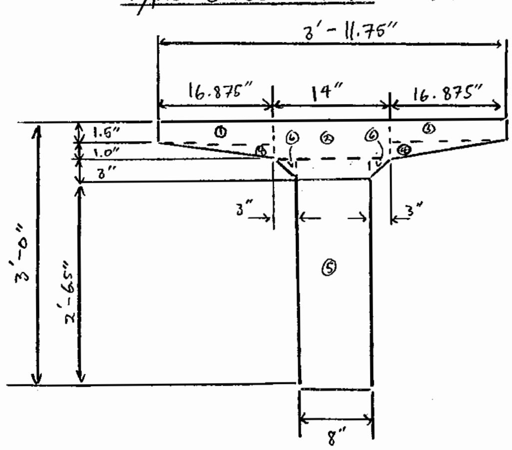
$A_5 = (8/12)(2+6.5/12+3/12) = 1.861\ ft^2$

$A_6 = 2\{1/2(2.875/12)(3/12)\} = 0.0599\ ft^2$

- $A_T = \sum_{i=1}^{6} A_i = 2.60417\ ft^2$

$\boxed{A_T = 2.60\ ft^2}$ Precast Tee Girder (Widened Section)

Typical Girder Section: Precast Tee Girder (Existing Section)



$A_1 = (16.875/12)(1.5/12) = 0.1758\ ft^2$

$A_2 = (14/12)(2.5/12) = 0.2430\ ft^2$

$A_3 = (16.875/12)(1.5/12) = 0.1758\ ft^2$

$A_4 = 2[1/2(16.875/12)(1.0/12)] = 0.1172\ ft^2$

$A_5 = (8/12)(2+6.5/12+3/12) = 1.8611\ ft^2$

$A_6 = 2[1/2(3/12)(3/12)] = 0.0625\ ft^2$

- $A_T = \sum_{i=1}^{6} A_i = 2.63542\ ft^2$

$\boxed{A_T = 2.64\ ft^2}$ Precast Tee Girder (Existing Section)

▲ Deck Section:

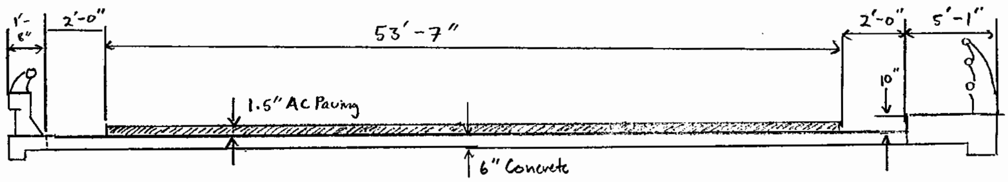
★ Note:

The deck is sloped with the high point at the center

⇒ Slope is ignored for the purposes of these calculations

$\gamma_{AC} = 152\ \text{lb/ft}^3$

$\gamma_{RC} = 150\ \text{lb/ft}^3$



• AC Pavement:

$$A_{AC} = (53 + 7/12)(1.5/12) = 6.6979\ ft^2$$

$\boxed{A_{AC} = 6.70\ ft^2}$ AC Pavement

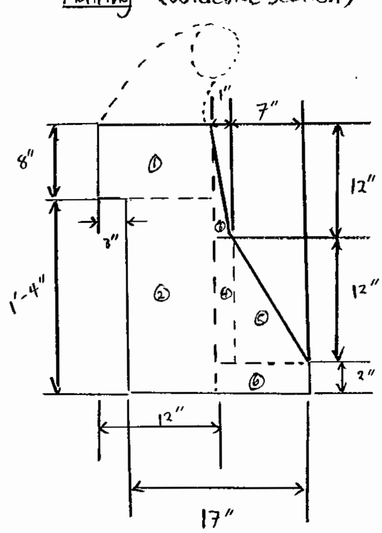
• Concrete:

$$A_T = ([1 + 8/12] + 2 + [53 + 7/12] + 2 + [5 + 1/12])(6/12) + (5 + 1/12)(10/12)$$
$$= 36.4028\ ft^2$$

$\boxed{A_T = 36.40\ ft^2}$ Concrete (Deck)

Left railing not included

▲ Railing: (Widened Section)



$A_1 = (12/12)(8/12) = 0.667\ ft^2$

$A_2 = (9/12)(1 + 4/12) = 1.0\ ft^2$

$A_3 = 1/2\,(1/12)(12/12) = .04167\ ft^2$

$A_4 = (1/12)(13/12) = 0.0833\ ft^2$

$A_5 = 1/2\,(7/12)(12/12) = 0.29167\ ft^2$

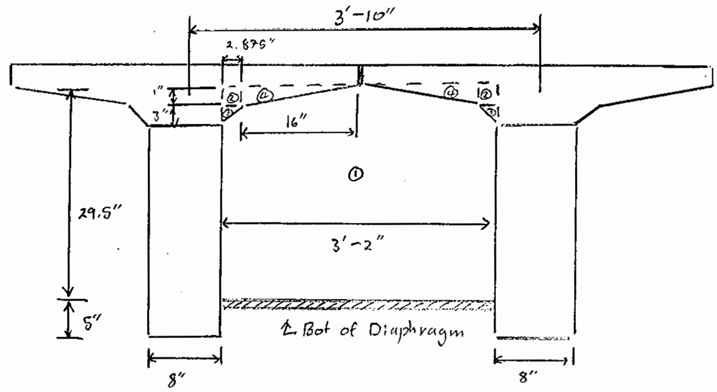
$A_6 = (8/12)(2/12) = 0.111\ ft^2$

• $A_T = \sum_{i=1}^{6} A_i = 2.19444\ ft^2$

$\boxed{A_T = 2.19\ ft^2}$ Railing Widened Section

▲ Concrete Diaphragm:

- Widened Section:



$A_1 = (3 + 3/12)(29.5/12) = 7.78472\ ft^2$

$A_2 = 2[(2.875/12)(1/12)] = 0.039931\ ft^2\ (-)$

$A_3 = 2[1/2\,(2.875/12)(3/12)] = 0.059896\ ft^2\ (-)$

$A_4 = 2[1/2\,(16/12)(1/12)] = 0.1111\ ft^2\ (-)$

- $A_T = A_1 - A_2 - A_3 - A_4$
  $= 7.57378\ ft^2$

$\boxed{A_T = 7.57\ ft^2}$ Concrete Diaphragm (Widened Section)

Thickness ≈ 1 ft

- Existing section:
  * Spacing center to center = 4'-0"

$A_1 = (3 + 4/12)(29.5/12) = 8.1944\ ft^2$

$A_2 = 2[(3/12)(1/12)] = .04167\ ft^2\ (-)$

$A_3 = 2[1/2\,(3/12)(3/12)] = .0625\ ft^2\ (-)$

$A_4 = 2[1/2\,(16.875/12)(1/12)] = 0.1172\ ft^2\ (-)$

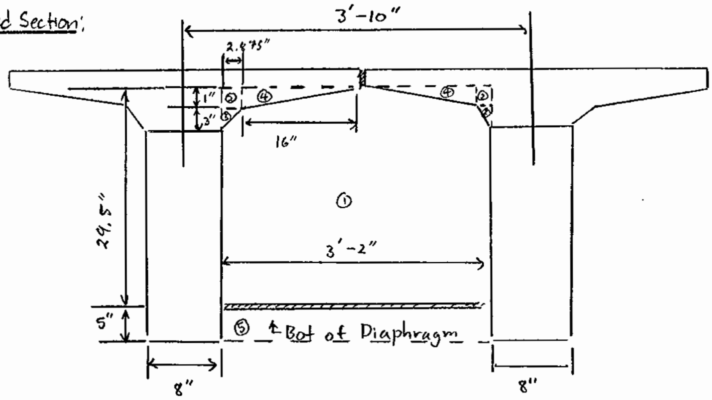
- $A_T = A_1 - A_2 - A_3 - A_4$
  $= 7.97303\ ft^2$

$\boxed{A_T = 7.97\ ft^2}$ Concrete Diaphragm (Existing Section)

Thickness ≈ 1 ft

▲ Air Pocket Calculations:

- Widened Section:



★ Note: The bottom of the diaphragm is 5 inches above the bottom of girders, but air will become trapped under full depth of the girders.

$A_1 = (3 + 2/12)(29.5/12) = 7.78472 \text{ ft}^2$

$A_5 = (3 + 2/12)(5/12) = 1.3194 \text{ ft}^2$

$A_2 = 2[(2.875/12)(1/12)] = 0.039931 \text{ ft}^2 \ (-)$

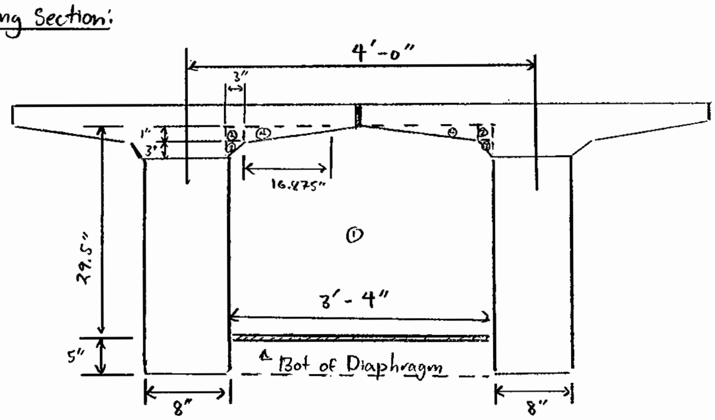
$A_3 = 2[\frac{1}{2}(2.875/12)(3/12)] = .059896 \text{ ft}^2 (-)$

$A_4 = 2[1/2(16/12)(1/12)] = .1111 \text{ ft}^2 \ (-)$

- $A_T = A_1 - A_2 - A_3 - A_4 + A_5 = 8.89323 \text{ ft}^2$

$\boxed{A_T = 8.8932 \text{ ft}^2}$ Air (Widened Section)

- Existing Section:



$A_1 = (3 + 4/12)(29.5/12) = 8.1944 \text{ ft}^2$

$A_5 = (3 + 4/12)(5/12) = 1.3889 \text{ ft}^2$

$A_2 = 2[(3/12)(1/12)] = .04167 \text{ ft}^2 \ (-)$

$A_3 = 2[1/2(3/12)(3/12)] = .0625 \text{ ft}^2 \ (-)$

$A_4 = 2[1/2(16.875/12)(1/12)] = 0.1172 \text{ ft}^2 (-)$

- $A_T = A_1 - A_2 - A_3 - A_4 + A_5 = 9.36196 \text{ ft}^2$

$\boxed{A_T = 9.362 \text{ ft}^2}$ Air (Existing Section)

▲ Reduction in air pocket:

* Note: It is assumed that the bridge is submerged to the top of deck.

○ Widened Section:

- $h = \underbrace{(1.5/12)}_{\text{AC Pave}} + \underbrace{(6/12)}_{\text{Deck}} + \underbrace{(3)}_{\text{to bot of air pocket}} = 3.625\text{ ft}$

- $P_2 = 14.7 + h(64/144)$
  $= 14.7 + (3.625)(64/144)$
  $P_2 = 16.311\text{ psi}$

- $A_2 = \frac{P_1 A_1}{P_2} = \frac{(14.7)(8.8932)}{16.3111}$
  $A_2 = 8.01478\text{ ft}^2$

  $\boxed{A_T = 8.015\text{ ft}^2}$ Compressed Air (Widened Section)

○ Existing Section:

- $h = 3.625\text{ ft}$

- $P_2 = 16.311\text{ psi}$

- $A_2 = \frac{P_1 A_1}{P_2} = \frac{(14.7)(9.362)}{16.3111}$
  $A_2 = 8.43728\text{ ft}^2$

  $\boxed{A_T = 8.437\text{ ft}^2}$ Compressed Air (Existing Section)

▲ Collection of Data:

○ AC Pavement: ($\gamma_{AC} = 152\text{ lb/ft}^3$)
Area = 6.70 ft²
Amount = 1
Length = 100.67 ft

○ Concrete: ($\gamma_{RC} = 150\text{ lb/ft}^3$)

- Tee Girder: (Widened)
  Area = 2.60 ft²
  Amount = 7
  Length = 100.67 ft

- Railing: (Widened)
  Area = 2.19 ft²
  Amount = 1
  Length = 100.67 ft

- Diaphgram: (Existing)
  Area = 7.97 ft²
  Amount = 9 x 5 = 45
  Length = 1 ft

- Tee Girder: (Existing)
  Area = 2.64 ft²
  Amount = 9
  Length = 100.67 ft

- Diaphgram: (widened)
  Area = 7.57 ft²
  Amount = 6 x 5 = 30
  Length = 1 ft

- Deck:
  Area = 36.40 ft²
  Length = 100.67 ft

▲ Collection of Data Continued:

- Compressed Air Pocket: ($\gamma_{seawater} = 64\ lb/ft^3$)
  - Widened:
    Area = 8.015 ft^2
    Amount = 6
    Length = 100.67 ft
  - Existing:
    Area = 8.437 ft^2
    Amount = 9
    Length = 100.67 ft

▲ Self Weight:

Self Weight = Concrete + AC Pavement

$$= (150\ lb/ft^3)[7(2.60)+9(2.64)+36.4+2.19](100.67) + (150\ lb/ft^3)(30)(7.57)(1) + (150\ lb/ft^3)(45)(7.97)(1) + (152\ lb/ft^3)(6.70)(100.67)$$

$$SW = 1.40669 \times 10^6\ lbs$$

▲ Buoyant Force:

Submerged Volume = Air Pocket + Concrete Volume + AC Pavement Volume

$$= (8.015)(6)(100.67) + (8.437)(9)(100.67) + (100.67)[(7)(2.60) + 9(2.64) + 36.4 - \underbrace{(6/12)(10/12)}_{\text{Raised curb on existing section}}] + (6.70)(100.67)$$

$$SV = 20621.3\ ft^3$$

Buoyant Force = $\gamma_{seawater} \times SV$

$$= (64)(20621.3)$$

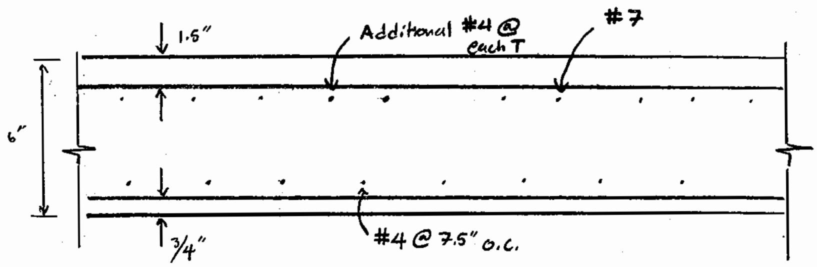
$$BF = 1.31976 \times 10^6\ lbs$$

▲ Summary:

Total Weight = 1406.7 kips
Buoyant Force = 1319.8 kips
Residual Weight = 86.9 kips
% Weight Retained = 6.18%

∴ Bridge is NOT Buoyant

■ Deck Capacity: (Widened Section)



$f_c' = 3000\text{psi}$
$f_y = 40{,}000\text{psi}$

$b = 26'-2'' = 314''$
Top: $14 - \#7$
$12 - \#4$
Bot: $42 - \#4$

○ Positive Bending:

I.) $A_s = (42)(0.20) = 8.4 \text{ in}^2$

II.) $a = \dfrac{A_s f_y}{0.85 f_c' b} = \dfrac{(8.4)(40)}{(.85(3)(314))}$

$a = 0.419633 \text{ in}$

III.) $d = 6 - 3/4 - 1/2(0.5) = 5 \text{in}$

IV.) $\varepsilon_s = .0274 \geq \varepsilon_y = .0014$ (o.k.)

V.) $M_n = A_s f_y (d - a/2)$
$= (8.4)(40)(5 - .41963/2)$
$M_n = 1609.5$ k-in $= 134.125$ k-ft

VI.) $\phi M_n = 0.90(134.125) = 120.713$ k-ft

> Positive Bending:
> $M_n = 134.13$ k-ft
> $\phi M_n = 120.71$ k-ft

○ Negative Bending:

I.) $A_s = (12)(0.20) + (14)(0.60) = 10.8 \text{ in}^2$

II.) $a = \dfrac{A_s f_y}{0.85 f_c' b} = \dfrac{(10.8)(40)}{0.85(3)(314)}$

$a = 0.53953 \text{ in}$

III.) $d = 6 - 1.5 - (1/2)(0.4) = 4.3 \text{in}$

IV.) $\varepsilon_s = .017 \geq \varepsilon_y = .0014$ (ok)

V.) $M_n = A_s f_y (d - a/2)$
$= (10.8)(40)(4.3 - .54/2)$
$M_n = 1741.06$ k-in $= 145.088$ k-ft

VI.) $\phi M_n = 0.90(145.088) = 130.58$ k-ft

> Negative Bending:
> $M_n = 145.088$ k-ft
> $\phi M_n = 130.58$ k-ft

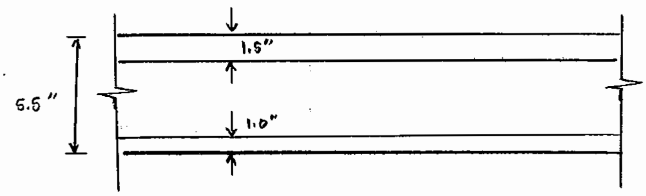
○ Negative Shear:

$V_c = 2\lambda\sqrt{f_c'}\, b_w d$
$= (2)(1.0)(3000)^{1/2}(314)(4.3)$
$V_c = 147907$ lbs $= 147.907$ kips

$\phi V_c = 0.75(147.907) = 110.93$ kips

> Negative Shear:
> $V_c = 147.9$ kips
> $\phi V_c = 110.9$ kips

▪ Deck Capacity: (Existing Section)



$f_c' = 3000$ psi
$f_y = 40{,}000$ psi
$b = 38'-2'' = 458$ in

Top: #4 @ 15" o.c. (30)
Bot: #5 @ 12" o.c. (38)

○ Positive Bending:

I.) $A_s = 38\,(0.31) = 11.78\text{ in}^2$

II.) $a = \dfrac{A_s f_y}{0.85 f_c' b} = \dfrac{(11.78)(40)}{0.85\,(3)(458)}$

$a = .403459$ in

III.) $d = 5.5 - 1.0 - .625/2 = 4.1875$ in

IV.) $\varepsilon_s = .0234 \geq \varepsilon_y = .0014$ (o.k.)

V.) $M_n = A_s f_y (d - a/2)$

$= (11.78)(40)(4.1875 - .403459/2)$

$M_n = 1878.1$ k-in $= 156.508$ k-ft

$\phi M_n = 0.90\,(156.508) = 140.857$ k-ft

Positive Bending:
$M_n = 156.508$ k-ft
$\phi M_n = 140.857$ k-ft

○ Negative Bending:

I.) $A_s = 30\,(0.20) = 6.0\text{ in}^2$

II.) $a = \dfrac{A_s f_y}{0.85 f_c' b} = \dfrac{(6.0)(40)}{0.85\,(3)(458)}$

$a = 0.205497$ in

III.) $d = 5.5 - 1.5 - (1/2)(0.5) = 3.75$ in

IV.) $\varepsilon_s > \varepsilon_y$ (o.k.)

V.) $M_n = A_s f_y (d - a/2)$

$= (6.0)(40)(3.75 - .205/2)$

$M_n = 875.34$ K-in $= 72.945$ k-ft

$\phi M_n = 0.90\,(72.945) = 65.6505$ k-ft

Negative Bending:
$M_n = 72.95$ k-ft
$\phi M_n = 65.65$ k-ft

○ Negative Shear:

I.) $V_c = 2\lambda \sqrt{f_c'}\, b_w d$

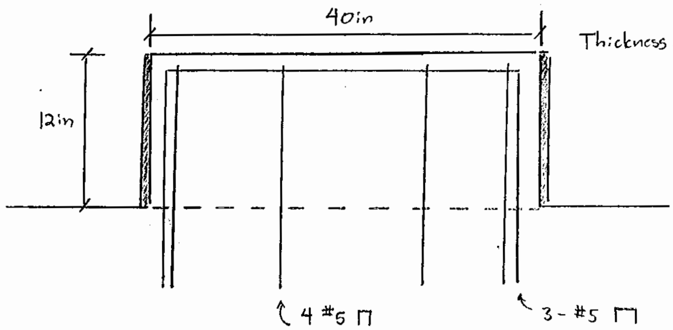
$= 2\,(1.0)(3000)^{1/2}(458)(3.75)$

$V_c = 188143$ lbs

II.) $\phi V_c = 0.75\,(188.143) = 141.107$ kips

Negative Shear:
$V_c = 188.14$ kips
$\phi V_c = 141.11$ kips

- <u>Creep Block Capacity</u>:



Thickness = 12in

$f_y = 40{,}000$ psi

$f_c' = 3{,}000$ psi

o <u>Shear Friction Capacity</u>: (ACI 318-08: Sec 11.6.4)

- $V_n = A_{vf} f_y \mu$

I.) $A_{vf} = (3)(\#5\ ⊓) + (4)(\#5\ ⊓)$
$= (3)(0.31)(2) + (4)(0.31)(2)$
$A_{vf} = 4.34\ in^2$

II.) Creep Block was poured monolithically
$\therefore\ \mu = 1.4\lambda = 1.4(1.0)$
$\mu = 1.4$

III.) $V_n = A_{vf} f_y \mu$
$= (4.34)(40{,}000)(1.4)$
$V_n = 243040$ lbs

IV.) Check:

① $0.2 f_c' A_c = 0.2(3000)(40)(12) = 288{,}000$ lbs $> V_n$ (o.k.)

② $(480 + 0.08 f_c')A_c = (480 + 0.08(3000))(40 \times 12) = 345600$ lbs $> V_n$ (o.k.)

③ $1600 A_c = 1600(40 \times 12) = 768000 > V_n$ (o.k.)

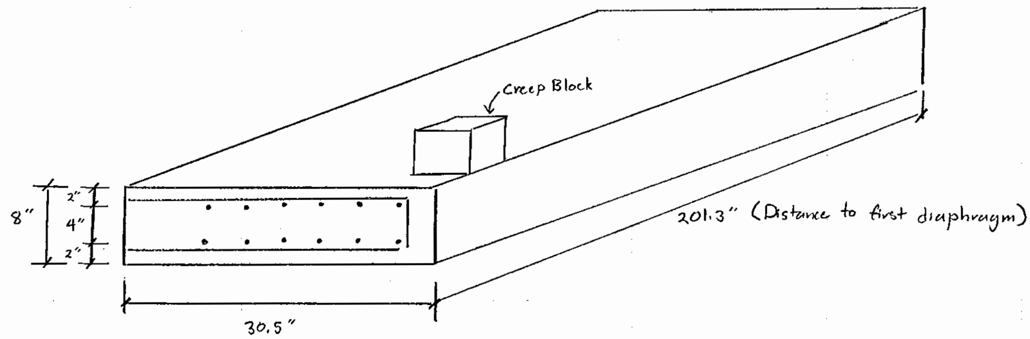
$\therefore\ V_n = 243040$ lbs

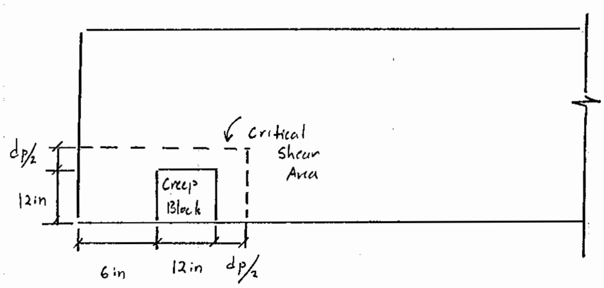
<u>Creep Block Capacity</u>:
$V_n = 243.04$ kips

- **Beam Web Capacity:**

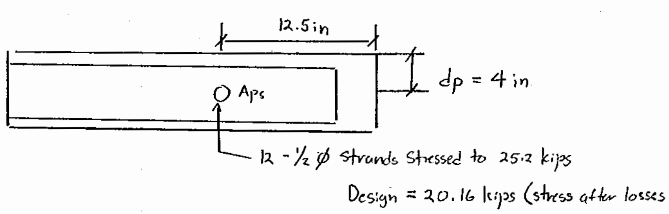
  - Punch Out Failure:

    * The web of the beam is analyzed as a flat slab with prestressing





Plan View



- Beam Web Capacity:
  - o Punchout Failure: cont.
  - Properties:

Concrete:

$f_c' = 6000\,psi$

$f_{ci}' = 4000\,psi$

$f_c = 0.45 f_c' = 2700\,psi$

$f_{ci} = 0.6 f_{ci}' = 2400\,psi$

$f_t = 12\sqrt{f_c'} = 929.5\,psi$

Prestressing:

$A_{ps} = 0.153\ in^2/strand$

$f_{pe} = \frac{(20160)}{0.153} = 131765\,psi$

- $V_c = (\beta_p \sqrt{f_c'} + 0.3\bar{f_c})\, b_o d + V_p$

I.) $\beta_p = \frac{\alpha_s d}{b_o} + 1.5$ For Corner column $\alpha_s = 20$

$\beta_p = \frac{(20)(4)}{b_o} + 1.5$

II.) $b_o = (6 + 12 + {}^{dp}\!/_2)(2) + (12 + {}^{dp}\!/_2)(2)$

$= (18 + 4/2)(2) + (12 + 4/2)(2)$

$b_o = 68\,in$

$\therefore\ \beta_p = {}^{(20)(4)}\!/_{68} + 1.5 = 2.676 < 3.5$

$\Rightarrow \beta_p = 2.676$

III.) $P_e = A_{ps} \times f_{pe} = (12 \times 0.153)(131765)$

$P_e = 241921\ lbs$

IV.) $\bar{f_c} = \frac{P_e}{A_c} = \frac{(241921)}{(30.5)(8)} = 991.478\,psi$

V.) $V_c = (\beta_p \sqrt{f_c'} + 0.3\bar{f_c})\, b_o d + V_p$

prestress is not harped $\therefore = 0$

$= ((2.676)(6000)^{1/2} + 0.3(991.478))(68)(4) + 0$

$V_c = 137295\ lbs$

$V_n = 137.3\ kips$

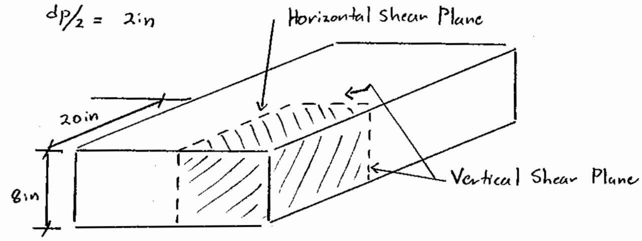
Punching Shear:
$V_n = 137.3\,kips$

■ Beam Web Capacity:

O Shear Failure of Independent Surfaces:

$d_p = 4 in$

$d_p/2 = 2 in$



• Horizontal Shear Plane:

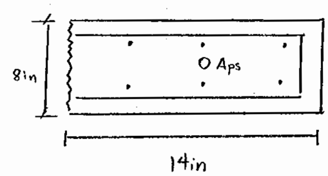
* Note: No prestressing or stirrups provide shear resistance

$V_c = 2\lambda\sqrt{f_c'}\, b_w d$

$= 2(1.0)(6000)^{1/2}(8)(20)$

$V_c = 24787.1$ lbs $= 24.787$ kips

• Vertical Shear Plane:



$A_{ps} = 6 \times 0.153 = 0.918\ in^2$

$A_c = (14)(8) = 112\ in^2$

$I = 1/12\, bh^3 = 597.3\ in^4$

$c = 1/2\, h = 4 in$

$S = I/c = 149.3\ in^3$

$r^2 = I/A = 5.33\ in^2$

Mild steel:

$f_y = 40$ ksi

$A_s = 0.20$

• $M_{cr} = S_b(6\lambda\sqrt{f_c'} + f_{ce} - f_d)$

I.) $f_d = 0$ (No dead load)

II.) $P_e = (0.918)(131765) = 120960$ lbs

III.) $f_{ce} = \frac{-P_e}{A_c}\left(1 + \frac{\cancel{ee}}{\cancel{r^2}}\right)^{\to 0} = \frac{(120960)}{112}$

$f_{ce} = 1080$ psi

IV.) $M_{cr} = S_b(6\lambda\sqrt{f_c'} + f_{ce})$

$= (149.3)(6 \times 1 \times (6000)^{1/2} + 1080)$

$M_{cr} = 230632$ lb-in

• $V_{ci} = 0.60\lambda\sqrt{f_c'}\, b_w d_p + V_d + \frac{V_i(M_{cr})}{M_{max}}$

I.) $V_d = 0$

II.) $V_i = 2.0\, F_R$ (Impact load ⇒ 2.0)

F_R = force from wave

$M_{max} = 12(2.0 F_R) = 24 F_R$

III.) $V_{ci} = 0.60\lambda\sqrt{f_c'}\, b_w d_p + \frac{V_i(M_{cr})}{M_{max}}$

$= 0.60(1.0)(6000)^{1/2}(14)(4) + \frac{(230632)}{12}$

$V_{ci} = 21822$ lbs $= 21.8$ kips

IV.) Check:

$1.7\lambda\sqrt{f_c'}\, b_w d_p = 7374.16$ lbs $< V_{ci}$ (ok)

$5.0\lambda\sqrt{f_c'}\, b_w d_p = 21688.7$ lbs $< V_{ci}$ (N.G.)

⇒ $V_{ci} = 21688.7$ lbs $= 21.6887$ kips

- Beam Web Capacity:
  - Vertical Shear Plane: cont.
    - $V_{cw} = (3.5\lambda\sqrt{f_c'} + 0.3\bar{f_c})\, b_w d_p + V_p$

I) $V_p = 0$

II) $\bar{f_c} = \frac{P_e}{A_c} = \frac{(120960)}{112}$

$\bar{f_c} = 1080$ psi

III) $V_{cw} = (3.5\lambda\sqrt{f_c'} + 0.3\bar{f_c})\, b_w d_p + V_p$

$= (3.5\,(1.0)(6000)^{1/2} + 0.3\,(1080))(14)(4)$

$V_{cw} = 33326$ lbs

$\therefore\ V_{cw} > V_{ci}$

$\Rightarrow\ V_c = V_{ci} = 21.689$ kips

- Stirrups:

#4 stirrups @ 6" o.c.

$V_s = \frac{A_v f_y d_p}{s} = \frac{(0.20)(40{,}000)(4)}{6}$

$V_s = 5333.3$ lbs $= 5.33$ kips

- Total Shear Capacity:

$V_n =$ Horizontal + Vertical

$= (24.787) + (21.689) + 5.33$

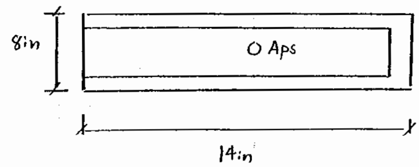
$V_n = 51.8093$ kips

$V_n = 51.81$ kips

$$\boxed{\begin{array}{l}\text{Web Shear Capacity:}\\ V_n = 51.81 \text{ kips}\end{array}}$$

Independent failure surfaces

- **Beam Web Capacity:**
  - Independent Failure Surfaces:
    - Vertical Plane Bending: (Flextural Capacity)



Assume $f_{pu} = 270{,}000$ psi

$f_c' = 6000$ psi

I.) $a = \dfrac{A_{ps} f_{ps}}{0.85 f_c' b}$

II.) $f_{ps} = f_{pu}\left(1 - \dfrac{\gamma_p}{\beta_1}\left[\rho_p \dfrac{f_{pu}}{f_c'} + \dfrac{d}{d_p}(\omega - \omega')\right]\right)$

- For low-relaxation strands
  $\gamma_p = 0.28$
- $4000 \leq f_c' \leq 8000$
  $\beta_1 = 0.85 - 0.05\left\{\dfrac{f_c' - 4000}{1000}\right\}$
  $\beta_1 = 0.75$
- $\omega = 0$
  $\omega' = 0$
- $\Rightarrow f_{ps} = f_{pu}\left(1 - \gamma_p/\beta_1 \left(\rho_p \, f_{pu}/f_c'\right)\right)$
  $= (270.)(1 - (0.28/0.75)[.016393(270/6)])$
  $f_{ps} = 195.642$ ksi

- $\rho_p = \dfrac{A_{ps}}{b\,d_p} = \dfrac{0.918}{(14)(4)}$
  $\rho_p = .016393$

III.) $a = \dfrac{A_{ps} f_{ps}}{0.85 f_c' b} = \dfrac{(0.918)(195.642)}{(.85)(6)(14)}$

$a = 2.5154$ in

IV.) $M_n = A_{ps} f_{ps} (d_p - a/2)$
$= (0.918)(195.642)(4 - 2.5154/2)$
$M_n = 492.516$ k-in
$M_n = 41.043$ k-ft

V.) Force from creep block will act $(6 + 4/2) = 8$ in away from failure surface:

$$V = \frac{M_n}{(8/12)} = \frac{41.043}{(8/12)}$$

$$V = 61.56 \text{ kips}$$

VI.) Total Capacity = Horizontal Shear + Vertical Flexture
= 24.787 + 61.56

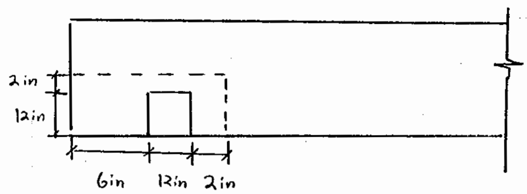
$\boxed{V_n = 86.3514 \text{ kips}}$

Horizontal Shear
Vertical Flexture

- Beam Web Capacity:
  - For Nonprestressed Slabs: (ACI 11.11.2.1)

    * 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



$d_p = 4\,in$

$d_p/2 = 2\,in$

① $V_c = (2 + 4/\beta)\lambda\sqrt{f_c'}\, b_o d$

I.) $\beta = 12/12 = 1$

II.) $b_o = 2(2+12) + 2(6+12+2) = 68\,in$

III.) $V_c = (2 + 4/\beta)\lambda\sqrt{f_c'}\, b_o d$

$= (2+4)(1.0)(6000)^{1/2}(68)(4)$

$V_c = 126414\ lbs = 126.414\ kips$

② $V_c = \left\{\left(\frac{\alpha_s d}{b_o}\right) + 2\right\}\lambda\sqrt{f_c'}\, b_o d$

I.) $\alpha_s = 20$ (corner)

II.) $V_c = \left(\frac{\alpha_s d}{b_o} + 2\right)\lambda\sqrt{f_c'}\, b_o d$

$= \left(\frac{20(4)}{68} + 2\right)(1.0)(6000)^{1/2}(68)(4)$

$V_c = 66925.2\ lbs = 66.9252\ kips$

③ $V_c = 4\lambda\sqrt{f_c'}\, b_o d$

$= 4(1.0)(6000)^{1/2}(68 \times 4)$

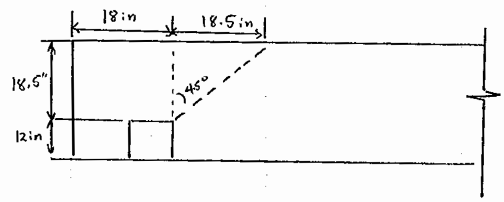
$V_c = 84276.1\ lbs = 84.276\ kips$

∴ ② Controls

$$\boxed{V_n = 66.93\ kips}$$

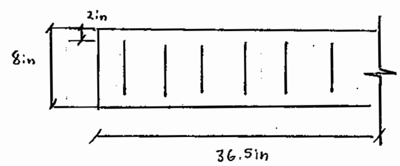
Nonprestressed Slab
Analysis Capacity

- **Beam Web Capacity:**
  - o Flexture Capacity at Top of Web:



Flexture Length = 36.5 in

⇒ 6 #4 stirrups



#4 Stirrups

*Note only the top will provide flextural resistance

$f_c' = 6000\,psi$

$f_y = 40{,}000\,psi$

I.) $A_s = 6(.20) = 1.2\ in^2$

II.) $a = \dfrac{A_s f_y}{0.85 f_c' b} = \dfrac{(1.2)(40)}{0.85(6)(36.5)}$

$a = 0.257857\ in$

III.) $d = 8 - 2 = 6\,in$

IV.) $\beta_1 = 0.85 - 0.05\left\{\dfrac{f_c' - 4000}{1000}\right\}$

$\beta_1 = 0.75$ for $4000 \leq f_c' \leq 8000\,psi$

V.) $\varepsilon_s = .0494 \geq \varepsilon_y = .001$ (o.k.)

VI.) $M_n = A_s f_y (d - a/2)$

$= (1.2)(40)(6 - 0.257/2)$

$M_n = 281.811\ k\text{-}in$

VII.) Force will act = 6 + 18.5 = 24.5 in away from flexture

$V_n = \dfrac{M_n}{24.5} = \dfrac{281.811}{24.5}$

$V_n = 11.502\ kips$

$\boxed{V_n = 11.502\ kips}$

Flexture Capacity at Top of Web

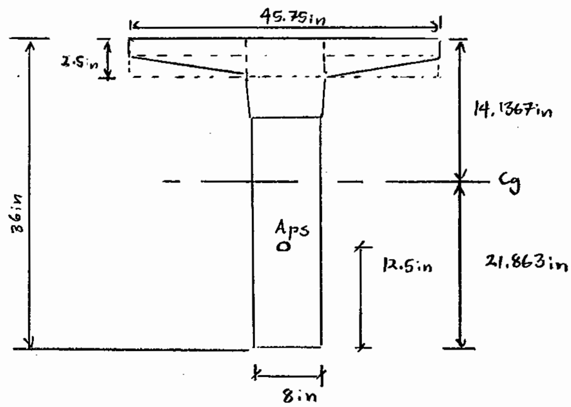
Summary of Results:

- Creep Block (Shear Friction) = 243.04 kips
- Beam Web Prestressed Slab Punchout Failure = 137.3 kips
- Beam Web Independent Shear Surfaces = 50.48 kips
- Beam Web Independent Shear-Flextural Surfaces = 86.35 kips
- Beam Web (Nonprestressed) Shear Failure = 66.93 kips
- Beam Web Flextural capacity = 11.502 kips ∴ controls

$\boxed{\text{Total Capacity For Bridge} = 8(11.502) = 92.016 \text{ kips}}$

(8 total creep Blocks)

Negative Bending Capacity:



Properties:

- $f_c' = 6000\,psi$
- $A_{ps} = 12 \times .153 = 1.836\ in^2$
- $P_e = 211680\ kips$
- $f_t = 12\sqrt{f_c'} = 929.516\ psi$
- $f_c = -.85 f_c' = -5100\ psi$

Compute Geometric Properties:

- Centroid: (see buoyancy calculations for areas)
  - $A_1 = 0.1667\ ft^2$
    $y_1 = 35.25\ in$
  - $A_2 = 0.2387\ ft^2$
    $y_2 = 34.75\ in$
  - $A_3 = .1667\ ft^2$
    $y_3 = 35.25\ in$
  - $A_4 = 0.111\ ft^2$
    $y_4 = 34.1667\ in$
  - $A_5 = 1.8661\ ft^2$
    $y_5 = 16.75\ in$
  - $A_6 = .0599\ ft^2$
    $y_6 = 32.5\ in$

$$\bar{y} = \frac{\Sigma A_i y_i}{\Sigma A_i} = 21.863\ in \text{ (from bottom)}$$

- Moment of inertia:
  - Top: $I = \frac{1}{12} b h^3$
    $= \frac{1}{12}(45.75)(2.5)^3$
    $I = 59.5703\ in^4$
    $\Rightarrow I + Ad^2 = 59.5703 + (45.75 \times 2.5)(14.1367 - 1.25)^2$
    $= 19058.5\ in^4$
  - Center Rectangle:
    $I = \frac{1}{12} b h^3$
    $= \frac{1}{12}(8)(30.5)^3$
    $I = 18915.1\ in^4$
    $\Rightarrow I + Ad^2 = 18915.1 + (8 \times 30.5)(21.863 - 15.25)^2$
    $= 29585.6\ in^4$
  - Remove triangles:
    $I = bh^3/36 = (16)(1^3/36) = .444\ in^4$
    $\Rightarrow I + Ad^2 = (.444) + (\frac{1}{2})(16)(1)(9.30337)$
    $= 74.8714\ in^4$

$I_c = 19053.5 + 29585.6 - (2)(74.8714) + (2)(490.07)$

$I_c = 49469.5\ in^4$

  - Small Triangles:
    $I = bh^3/36 = (2.875)(3)^3/36 = 2.15625\ in^4$
    $\Rightarrow I + Ad^2 = 2.15625 + (\frac{1}{2} \times 2.875 \times 3)(10.6367)^2 = 490.07\ in^4$

Negative Bending Capacity:

- **Geometric Properties:**

$A_c = 374.4 \text{ in}^2$

$I_c = 49469.5 \text{ in}^4$

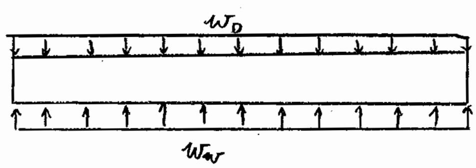
$C_t = 14.1367 \text{ in}$ $\quad S_t = 3499.37 \text{ in}^3$

$C_b = 21.863 \text{ in}$ $\quad S_b = 2262.7 \text{ in}^3$

$r^2 = 132.13 \text{ in}^2$

$e_c = 9.363 \text{ in}$

$L = 50 \text{ ft} = 600 \text{ in}$



$w_D = (7.5'' \text{ topping}) + \text{self weight}$

$= ((7.5/12)(45.75/12) + 2.60)\,150 = 747.42 \text{ lb/ft}$

$w_D = 62.285 \text{ lb/in}$ (unsubmerged)

I.) Tensile Limit: (unsubmerged)

$$W_{wt} = \frac{8}{L^2}\left(M_D + S_t f_t - \frac{S_t P_e}{A_c}\left(\frac{e C_b}{r^2} - 1\right)\right)$$

$$M_D = w_D L^2/8 = (62.285)(600)^2/8 = 2.80283 \times 10^6 \text{ lb-in}$$

$$\Rightarrow W_{wt} = \frac{8}{(600)^2}\left(2.80283 \times 10^6 + (3499.37)(929.516) - \frac{(3499.37)(211680)}{374.4}\left(\frac{9.36 \times 14.1367}{132.13} - 1\right)\right)$$

$W_{wt} = 134.505 \text{ lb/in}$

$\Rightarrow F_w = 80.703$ kips (tensile Limit)

II.) Compression Limit: (unsubmerged)

$$W_{wc} = \frac{8}{L^2}\left(M_D - (S_b f_c) - \left(\frac{S_b P_e}{A_c}\right)\left(1 + \frac{e C_b}{r^2}\right)\right)$$

$W_{wc} = 246.25 \text{ lb/in}$

$\Rightarrow F_w = 147.75$ kips (compression limit)

III.) Tensile Limit: (submerged)

$w_D = ((7.5/12)(45.75/12) + 2.60)(150 - 64) = 428.522 \text{ lb/ft}$

$w_D = 35.71 \text{ lb/in}$

$\Rightarrow M_D = 1.60696 \times 10^6 \text{ lb-in}$

$\Rightarrow W_{wt} = 107.93 \text{ lb/in}$

$\Rightarrow F_w = 64.76$ kips (Tensile Limit)

- **Negative Bending Capacity:**

  IV.) Compression Limit: (submerged)

  $w_{wc} = 219.677\ \text{lb/in}$

  $\Rightarrow F_w = 131.8$ kips (compression limit)

- **Summary of Results:** (32 girders)

  - Unsubmerged:
    - Tensile: $F_w = 80.703$ kips
    - Compression: $F_w = 147.75$ kips
  - Submerged:
    - Tensile: $F_w = 64.76$ kips
    - Compression: $F_w = 131.8$ kips
  - Loss once submerged:
    - Tensile: 19.76 %
    - Compression: 10.80 %

Buoyancy Calculations:

Self Weight:

From Sheet 1

Item	Superstructure	Railing & End Post	Total	Total Weight
(γ = 145 pcf) Class A Concrete	748.01 cy	28.90 cy	776.91 cy (20976.6 ft^3)	3.04161×10^6 lbs
Reinf. Steel	150,873 lbs	5183 lbs	- NA -	156056 lbs
(γ = 152 pcf) 1" AC Pavement				70.25 Tons (140,500 lbs)

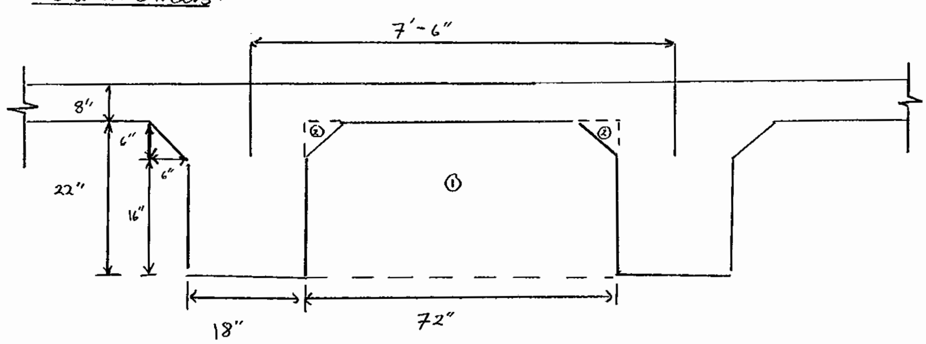
Total = 3.33816×10^6 lbs

Self Weight = 3.33816×10^6 lbs

$\boxed{\Rightarrow \text{Self Weight} = 3338.2 \text{ kips}}$

Air Pocket Calculation:

- Between Girders:



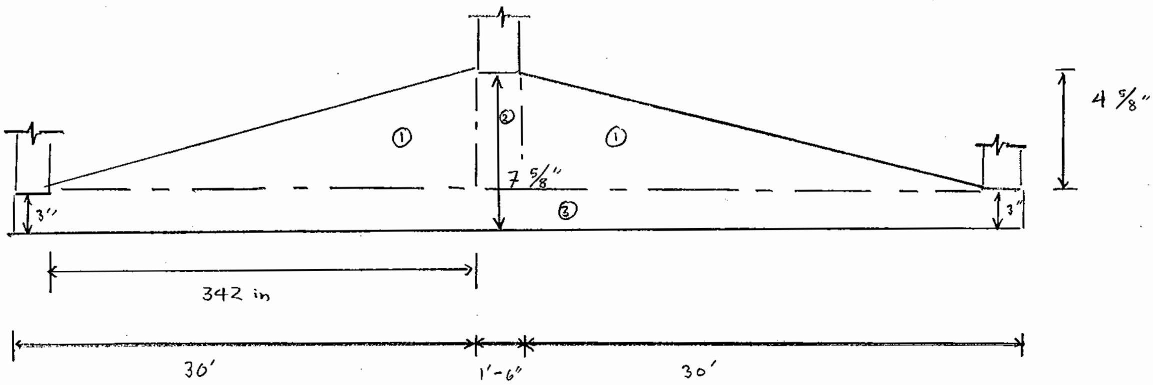
$A_1 = (72)(22)(1/144) = 11\ ft^2$

$A_2 = 2\left[\tfrac{1}{2}(6/12)(6/12)\right] = 0.25\ ft^2$

- $A_T = A_1 - A_2 = 10.75\ ft^2$

$\boxed{A_T = 10.75\ ft^2}$ Air Pocket Between Girders
Amount = 8
Length = 215 ft

- **Air Pocket Calculation:**
  - Between Deck and Pier Cap Reinforcement:



$A_1 = 2\left[\frac{1}{2}(30)\left(\frac{4.625}{12}\right)\right] = 11.5625\ ft^2$

$A_2 = \left(1+\frac{6}{12}\right)\left(\frac{4.625}{12}\right) = 0.578125\ ft^2$

$A_3 = (61.5)\left(\frac{3}{12}\right) = 15.375\ ft^2$

- $A_T = A_1 + A_2 + A_3 = 27.5156\ ft^2$

$\boxed{A_T = 27.52\ ft^2}$ Air Between Girders & Pier Cap

Amount = 1

Length = 215 ft

- Compression of Air Pocket:
  - $h = (7 + \frac{5}{8}) + 30 + 1 = 38.625\text{ in} = 3.21875\ ft$
  - $P_2 = P_1 + h_2\left(\frac{64}{144}\right)$

    $= 14.7 + (3.219)\left(\frac{64}{144}\right)$

    $P_2 = 16.1306\ psi$
  - Total Air Pocket $= 8(10.75) + 27.52 = 113.516\ ft^2$
  - $A_2 = \dfrac{P_1 A_1}{P_2} = \dfrac{(14.7)(113.516)}{(16.1306)}$

$A_2 = 103.449\ ft^2$

$\boxed{A_T = 103.45\ ft^2}$ Compressed Air Pocket

Amount = 1

Length = 215 ft

▲ Buoyant Force: ($\gamma_{sw} = 64\ ^{lb}/_{ft^3}$)

* Assume the bridge is submerged to the top of the deck

o Submerged Volume = Air Pocket + concrete + Ac

$= (103.45)(215) + 748.01(27) + (140500/152)$

$S\forall = 43362.1\ ft^3$

o Buoyant Force $= 43362.1\,(64)$

$BF = 2.77517 \times 10^6$ lbs

o Residual Weight $= 3.33816 \times 10^6 - 2.77517 \times 10^6$

$RW = 562987$ lbs

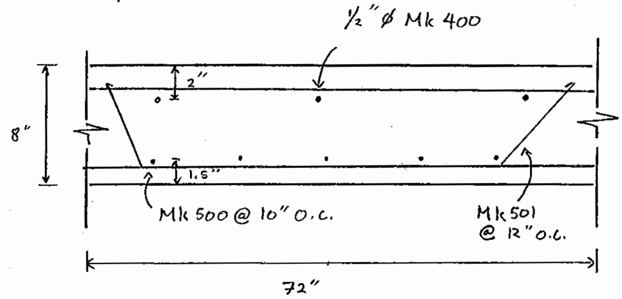
% Retained = 16.8652 %

▲ Summary of Results:

- Self Weight = 3338 kips
- Buoyant Force = 2775 kips
- Residual Weight = 563.0 kips
- % Retained = 16.9 %

∴ Bridge is NOT Buoyant

Deck Capacity:



$f_y = 40{,}000$ psi
$f_c' = 3{,}000$ psi (class A conc.)

Top: 3 #4 bars
Bot: 5 #5 bars

o Positive Bending:

I.) $A_s = (5)(0.31) = 1.55 \text{ in}^2$

II.) $a = \dfrac{A_s f_y}{0.85 f_c' b} = \dfrac{(1.55)(40)}{(.85)(3)(72)}$

$a = 0.337691$ in

III.) $d = 8 - 1.5 - \tfrac{1}{2}(0.625) = 6.1875$ in

IV.) $\varepsilon_s = .0437 \geq \varepsilon_y = .0014$ (o.k.)

V.) $M_n = A_s f_y (d - a/2)$
$= (1.55)(40)(6.1875 - .3376/2)$
$M_n = 373.157$ k-in $= 31.096$ k-ft

$\phi M_n = 0.9(31.096) = 27.9867$ k-ft

Positive Bending:
$M_n = 31.096$ k-ft
$\phi M_n = 27.987$ k-ft

o Negative Bending:

I.) $A_s = (3)(0.20) = 0.60 \text{ in}^2$

II.) $a = \dfrac{A_s f_y}{0.85 f_c' b} = \dfrac{(0.60)(40)}{0.85(3)(72)}$

$a = 0.130719$ in

III.) $d = 8 - 2 - (\tfrac{1}{2})(0.5) = 5.75$ in

IV.) $\varepsilon_s = .109 \geq \varepsilon_y = .0014$ (o.k.)

V.) $M_n = A_s f_y (d - a/2)$
$= (0.60)(40)(5.75 - .1307/2)$
$M_n = 136.431$ k-in $= 11.369$ k-ft

$\phi M_n = 10.2324$ k-ft

Negative Bending:
$M_n = 11.37$ k-ft
$\phi M_n = 10.23$ k-ft

o Negative Shear:

I.) $V_c = 2\lambda\sqrt{f_c'}\, b_w d$
$= (2)(1.0)(3000)^{1/2}(72)(5.75)$
$V_c = 45351.4$ lbs

II.) $V_s = \dfrac{A_v f_y d}{s} = \dfrac{2(0.31)(40{,}000)(5.75)}{12}$

$V_s = 11883.3$ lbs

III.) $V_n = V_c + V_s = 57.2348$ kips

$\phi V_n = 0.75(57.2348) = 42.9261$ kips

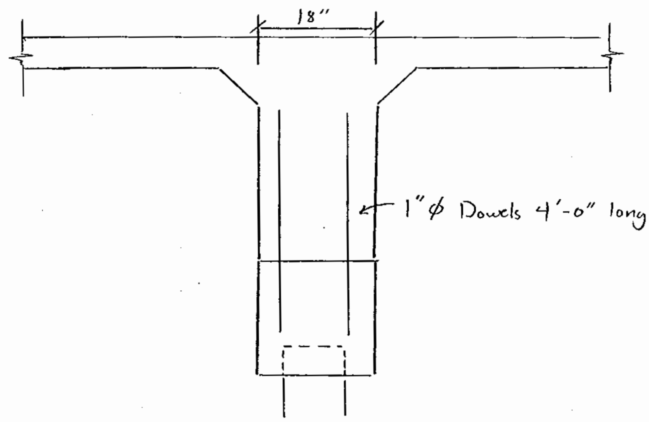
Negative Shear:
$V_n = 57.23$ kips
$\phi V_n = 42.93$ kips

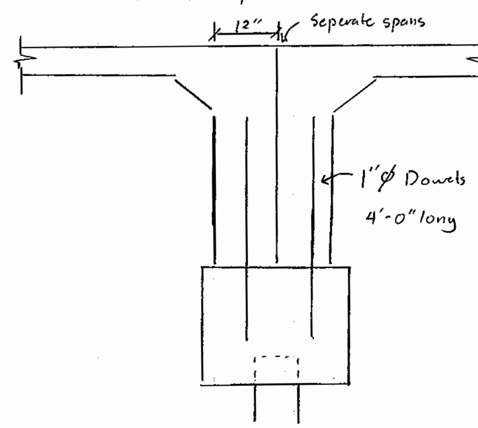
Lateral Resistance:

- **Dowels:**

  * Note: 1" ∅ Dowels 4'-0" long (MK 804) are only provided at fixed abutments/Piers only (Piers No. 2, 3, 4, 5, & 6)

  $f_y = 40{,}000$ psi; $f_c' = 3{,}000$ psi





- **Shear Friction:** (ACI Section 11.6.4)

  I.) $V_n = A_{vf} f_y \mu$

  II.) $A_{vf} = (2)(0.79) = 1.58 \text{ in}^2$

  III.) Concrete placed against hardened concrete (placed monolithically)

  $\mu = 1.4\lambda = 1.4$

  IV.) $V_n = A_{vf} f_y \mu$
  $= (1.58)(40{,}000)(1.4)$
  $V_n = 88480 \text{ lbs} = 88.48 \text{ kips}$

  ⇒ For single span: (span 3, 4, 5 & 6)
  9 girders ⇒ 18 total shear friction

  $V_n = (18)(88.48) = 1592.64$ kips

  ⇒ For single Span: (span 2 & 7)
  9 girders ⇒ 9 total shear friction

  $V_n = 796.32$ kips

- **Self Weight:** (Single Span)

  - $SW = (3.33816 \times 10^6)(1/8) = 417270$ lbs

    $SW = 417.27$ kips

  - Concrete to concrete ⇒ $\mu = 0.8$
    ⇒ Friction = 333.816 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)

Properties one span:
Width = 27 ft
Length = 6[illegible]

- 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:
    $h = 3'$
    width $= 2'-8''$
    Length $= 66''$

    I.) $\forall = (3)(2 + 8/12)(66) = 528\ ft^3$

    II.) $W = \forall \times \gamma_{RC} = (528)(150)$
    $W = 79200\ lbs = 79.2\ kips$

  - Total Vertical Resistance: (spans 3, 4, 5, & 6)

    $VR =$ Self Weight $+ 2$ (pier caps)
    $= (417.27) + (2)(79.2)$
    $VR = 575.67$ 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

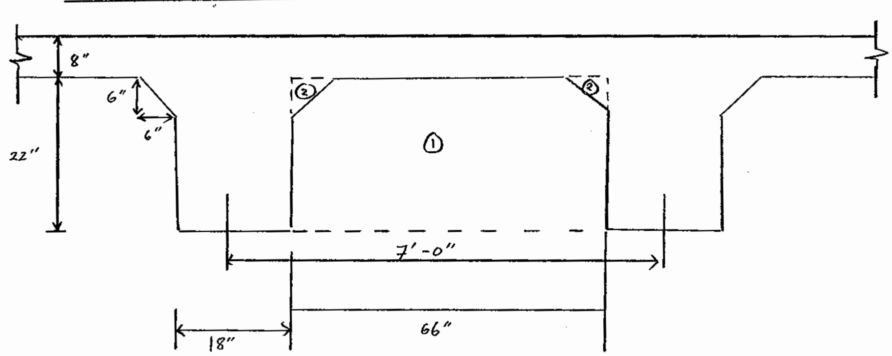
Item	Superstructure	Railing & End Post	Total	Total Weight
Class A Concrete ($\gamma = 145$ pcf)	885.83 cy	25.63 cy	911.46 cy ($24609.4\ ft^3$)	3.56837×10^6 lbs
Reinf. Steel	198,093 lbs	4550 lbs	-NA-	202643 lbs
1" AC Pavement ($\gamma = 152$ pcf)			-NA-	89.8 tons (179600 lbs)
AC Roll ($\gamma = 152$ pcf)			-NA-	2.5 tons (5000 lbs)

Total $= 3.95561 \times 10^6$ lbs

Self Weight $= 3.95561 \times 10^6$ lbs

$\boxed{\text{Self Weight} = 3955.6 \text{ kips}}$

Air Pocket Calculation:



$$A_1 = (66/12)(22/12) = 10.083\ ft^2$$

$$A_2 = 2[(1/2)(6/12)(6/12)] = 0.25\ ft^2$$

$$\bullet\ A_T = A_1 - A_2 = 9.833\ ft^2$$

$\boxed{A_T = 9.83\ ft^2}$ Air Pocket Between Girders

Amount = 12

Length = 188 ft

▲ Air Pocket Calculations: cont.

- Compression of Air Pocket:

  * Assume bridge is submerged to the top of deck

  - $h = (30) + 1 = 31 \text{ in} = 2.583 \text{ ft}$

  - $P_2 = P_1 + h\,(64/144)$
    $= 14.7 + (2.583)(64/144)$
    $P_2 = 15.8481 \text{ psi}$

  - $A_2 = \dfrac{P_1 A_1}{P_2} = \dfrac{(14.7)(9.83)}{(15.8481)}$
    $A_2 = 9.121 \text{ ft}^2$

    $\boxed{A_2 = 9.121 \text{ ft}^2}$ Compressed Air

▲ Buoyant Force: ($\gamma_{sw} = 64 \text{ lb/ft}^3$)

- Submerged Volume = Air + Concrete + AC
  $= (9.121)(8)(188) + (885.83 \times 27) + (179600/152)$
  $S\forall = 38816.9 \text{ ft}^3$

- Buoyant Force $= (38816.9)(64)$
  $BF = 2.48428 \times 10^6 \text{ lbs}$

- Residual Weight $= 3.95561 \times 10^6 - 2.48428 \times 10^6$
  $RW = 1.47133 \times 10^6 \text{ lbs}$

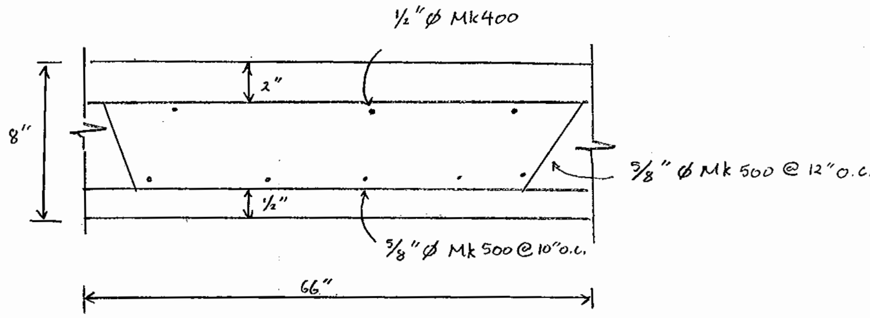
  % Retained = 37.196 %

▲ Summary of Results:

- Self Weight = 3956 kips
- Buoyant Force = 2484 kips
- Residual Weight = 1471 kips
- % Retained = 37.2 %

∴ Bridge is NOT Buoyant

Deck Capacity:



$f_y = 40{,}000$ psi
$f_c' = 3000$ psi

Positive Bending:

I.) $A_s = 1.55\,in^2$

II.) $a = \dfrac{A_s f_y}{0.85 f_c' b} = \dfrac{(1.55)(40)}{0.85(3)(66)}$

$a = .36839$ in

III.) $d = 6.1875$ in

IV.) $\varepsilon_s > \varepsilon_y$ (o.k.)

V.) $M_n = A_s f_y (d - a/2)$

$= (1.55)(40)(6.1875 - .36839/2)$

$M_n = 372.205$ k-in $= 31.0171$ k-ft

$\phi M_n = 27.9154$ k-ft

$$\boxed{\begin{array}{l}\text{Positive Bending:}\\ M_n = 31.02 \text{ k-ft}\\ \phi M_n = 27.92 \text{ k-ft}\end{array}}$$

Negative Bending:

I.) $A_s = 0.60\,in^2$

II.) $a = \dfrac{A_s f_y}{0.85 f_c' b} = \dfrac{(0.60)(40)}{0.85(3)(66)}$

$a = .1426$ in

III.) $d = 5.75$ in

IV.) $\varepsilon_s \geq \varepsilon_y$ (o.k.)

V.) $M_n = A_s f_y (d - a/2)$

$= (0.60)(40)(5.75 - .1426/2)$

$M_n = 136.289$ k-in $= 11.3574$ k-ft

$\phi M_n = 10.2217$ k-ft

$$\boxed{\begin{array}{l}\text{Negative Bending:}\\ M_n = 11.36 \text{ k-ft}\\ \phi M_n = 10.22 \text{ k-ft}\end{array}}$$

Negative Shear:

I.) $V_c = 2\lambda\sqrt{f_c'}\, b_w d$

$= (2)(1.0)(3000)^{1/2}(66)(5.75)$

$V_c = 41572.1$ lbs

II.) $V_s = 11883.3$ lbs (same as Moanalua Bridge)

III.) $V_n = V_c + V_s = 53455.4$ lbs $= 53.46$ kips

$\phi V_n = 40.09$ kips

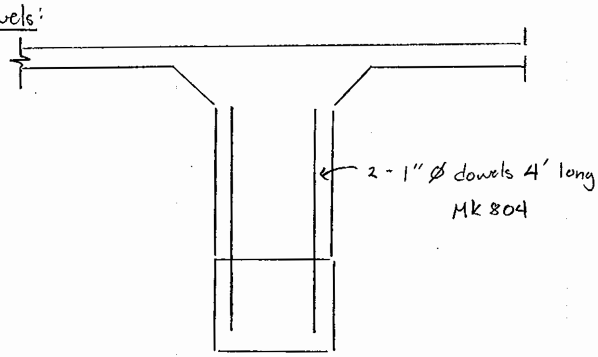
$$\boxed{\begin{array}{l}\text{Negative Shear:}\\ V_n = 53.46 \text{ kips}\\ \phi V_n = 40.09 \text{ kips}\end{array}}$$

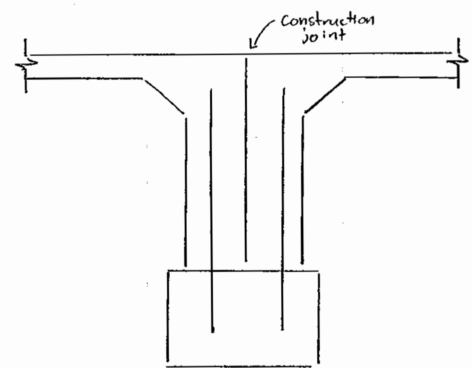
■ Lateral Resistance:

* Note: Piers 2, 3, 4 & 6 have 3 Girders secured to the pier caps with dowels
There are 13 girders ∴ The dowels will only provide some lateral resistance but no vertical resistance.
↳ The front will lift causing a moment at the fixed end failing the dowels.

$f_y = 40{,}000$ psi
$f_c' = 3{,}000$ psi

o Dowels:





• Shear friction:

I.) $V_n = A_{vf} f_y \mu$

II.) $A_{vf} = (2)(0.79) = 1.58 \text{ in}^2$

III.) $\mu = 1.4\lambda = 1.4$
(concrete placed monolithically)

IV.) $V_n = A_{vf} f_y \mu$
$= (1.58)(40)(1.4)$
$V_n = 88.48$ kips

• For typical span
3 Girders restrained by dowels ⇒ 6 total

$V_n = 6(88.48) = 530.88$ kips

o Self Weight: (Single Span)

$SW = (3.95561 \times 10^6)/7 = 565087 \text{ lbs} = 565.087$ kips

* Girders sit on 4 layers of 2ply roofing paper:
$\mu \approx 0.5$
Friction = $(565.087)(0.5) = 282.544$ kips

o Total Lateral Resistance: (Typical Span)

o TLR = $(530.88) + 282.544 = 813.424$ kips

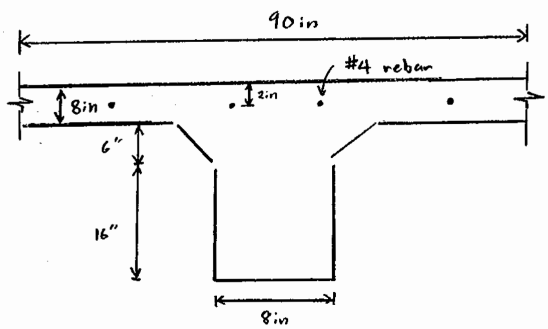
- **Vertical Resistance:**

  * Dowels will not provide additional vertical resistance

  Total Vertical Resistance = self Weight = 565.087 kips

- **Summary:** (Typical Single Span)

  $$\boxed{\begin{array}{l}\text{Lateral Resistance} = 813.424 \text{ kips}\\ \text{Vertical Resistance} = 565.087 \text{ kips}\end{array}}$$



$f_c' = 3000 psi$
$f_y = 40{,}000 psi$
$A_s = 4(0.2) = 0.8 in^2$
$L = 27 ft = 324 in$

- Negative Bending Capacity:

I.) $a = \frac{A_s f_y}{0.85 f_c' b_e} = \frac{(0.8)(40)}{.85(3)(90)}$

$a = .139434 in$

II.) $d = 28$ in

$\beta_1 = 0.85$

III.) $\varepsilon_s = .509 \geq \varepsilon_y = .0014$ (ok)

IV.) $M_n = A_s f_y (d - a/2)$

$= (0.8)(40)(28 - .1394/2)$

$M_n = 893.769$ k-in

$M_n = 893769$ lb-in

IV.) Distributed Load:

$M = \frac{w_w L^2}{8}$

$\Rightarrow w_w = 68.1123$ lb/in

$F_w = 22.0684$ kips

- Negative Bending Capacity:

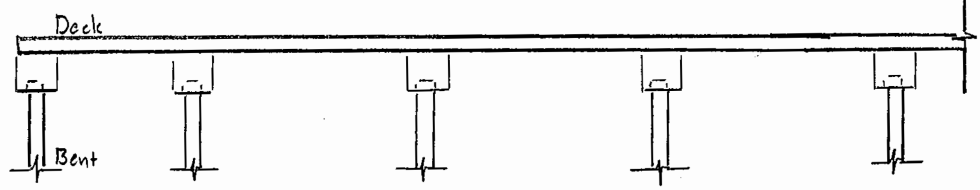
$w_w = 68.1123$ lb/in

$F_w = 22.07$ kips

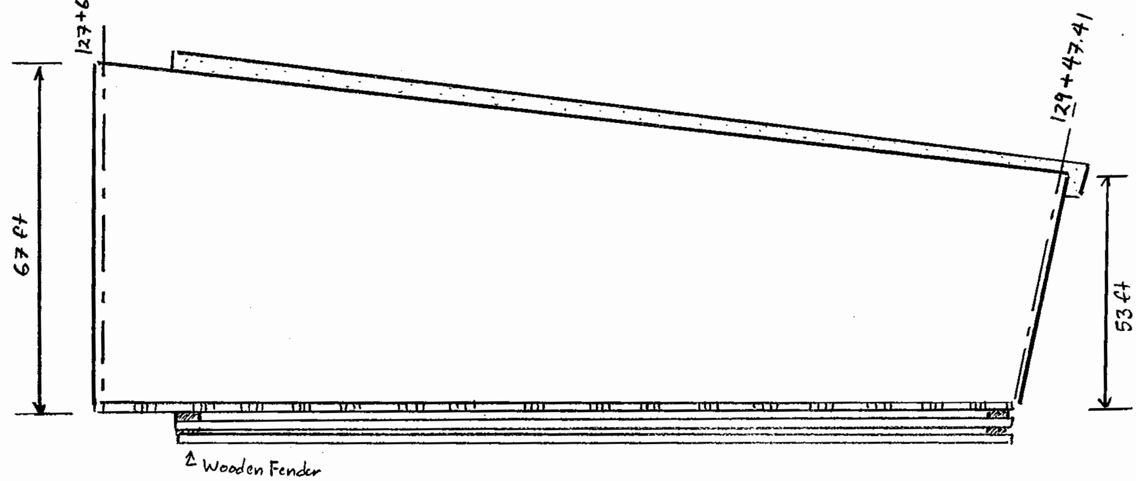
■ Buoyancy Calculations:

* Note: Reinforcement: Nk 400s = $\frac{1}{2}$"ϕ, Nk 500s = $\frac{5}{8}$"ϕ, Nk 600s = $\frac{3}{4}$"ϕ, Nk 700s = $\frac{7}{8}$"ϕ, Nk 800s = 1"ϕ

▲ Self Weight:



Front Elevation (Sheet 10)



Layout Plan (Sheet 10)

o Concrete: (From sheet 1)

Volume Superstructure and Railing = 499 + 12 = 511 yd^3 = 13797 ft^3

o Reinforcing Steel: (From Sheet 1)

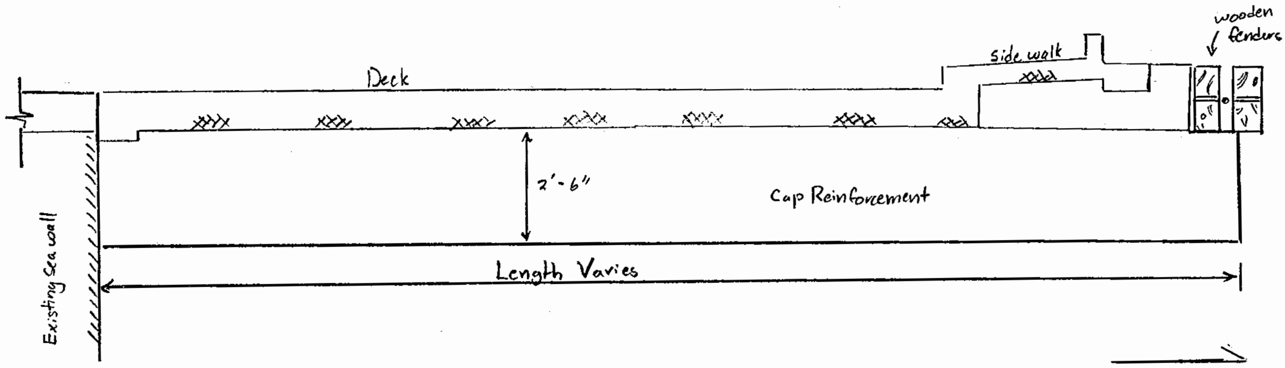
Weight = (Deck and Framing) + (Railing and Rail Post)
= (66,368) + (2,520)
Weight = 68888 lbs

o Total Self Weight: ($\gamma_{concrete}$ = 145 lb/ft^3)

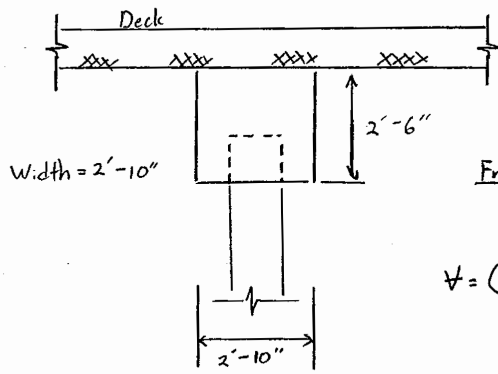
Self Weight = Concrete + Steel
= (13797)(145) + 68888
Self Weight = 2069453 lbs

$\boxed{\text{Self Weight} = 2069.45 \text{ kips}}$

- ▲ Air Pocket Calculations: (Sheet No. 12)
  - o Typical Cap Reinforcing:



- o Concrete Cap volume: (Intermediate Span)



Front View

$$\forall = (2+10/12)(2+10/12)(2+6/12) = 20.0694\ ft^3$$

$$\boxed{\forall = 20.07\ ft^3}$$

* 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: ($\gamma_{sea\ water} = 64\ lb/ft^3$)
  - o Submerged Volume = submerged Concrete + air pocket
    $= 499 + 0$
    $S\forall = 499\ cy = 13473\ ft^3$

* Note: Assume the bridge is submerged to the top of the deck.

  - o Buoyant Force:

$$BF = S\forall \times \gamma_{sw}$$
$$= (13473)(64)$$
$$\boxed{BF = 862272\ lbs}$$

- ▲ Residual Weight: (Once submerged)

$$\text{Residual Weight} = SW - BF$$
$$= 2069453 - 862272$$
$$\boxed{RW = 1207181\ lbs}$$

- <u>Summary of Results:</u>

  - Self Weight = 2069.5 kips
  - Buoyant Force = 862.3 kips
  - Residual Weight = 1207.2 kips
  - % Retained Weight = 58.3 %

$\therefore$ <u>Bridge is NOT Buoyant</u>

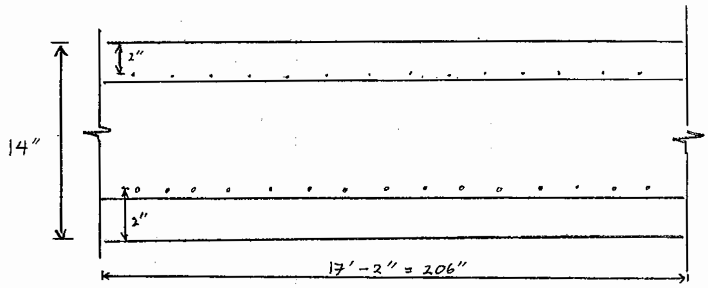
National® Brand 42-182 100 SHEETS

Deck Capacity:

$f_y = 40{,}000\ psi$

$f_c' = 3500\ psi$

* Note: No. Spans = 10
Max Length = 67 ft
Min Length = 53 ft



14 - ½" ϕ Mk 400 Top (#4 bars)

15 - ⅝" ϕ Mk 500 Bot (#5 bars)

○ Positive Bending:

I.) $A_s = (15)(0.31) = 4.65\ in^2$

II.) $a = \frac{A_s f_y}{0.85 f_c' b} = \frac{(4.65)(40)}{0.85(3.5)(206)}$

$a = 0.3035\ in$

III.) $d = 14 - 2 - (\tfrac{1}{2})(.625) = 11.6875\ in$

IV.) $\varepsilon_s = .0952 \geq \varepsilon_s = .0014$ (o.k.)

V.) $M_n = A_s f_y (d - a/2)$

$= (4.65)(40)(11.6875 - .3035/2)$

$M_n = 2145.65\ k\text{-}in = 178.8\ k\text{-}ft$

$\phi M_n = 0.9(178.8) = 160.924\ k\text{-}ft$

$$\boxed{\begin{array}{l}\text{Positive Bending:}\\ M_n = 178.8\ k\text{-}ft\\ \phi M_n = 160.9\ k\text{-}ft\end{array}}$$

○ Negative Bending:

I.) $A_s = (14)(0.20) = 2.8\ in^2$

II.) $a = \frac{A_s f_y}{0.85 f_c' b} = \frac{(2.8)(40)}{0.85(3.5)(206)}$

$a = 0.18275\ in$

III.) $d = 14 - 2 - (\tfrac{1}{2})(0.5) = 11.75\ in$

IV.) $\varepsilon_s = .161 > \varepsilon_y = .0014$ (o.k.)

V.) $M_n = A_s f_y (d - a/2)$

$= (2.8)(40)(11.75 - .1827/2)$

$M_n = 1305.77\ k\text{-}in = 108.8\ k\text{-}ft$

$\phi M_n = 0.90(108.8) = 97.9\ k\text{-}ft$

$$\boxed{\begin{array}{l}\text{Negative Bending:}\\ M_n = 108.8\ k\text{-}ft\\ \phi M_n = 97.9\ k\text{-}ft\end{array}}$$

○ Negative Shear:

$V_c = 2\lambda\sqrt{f_c'}\, b_w d$

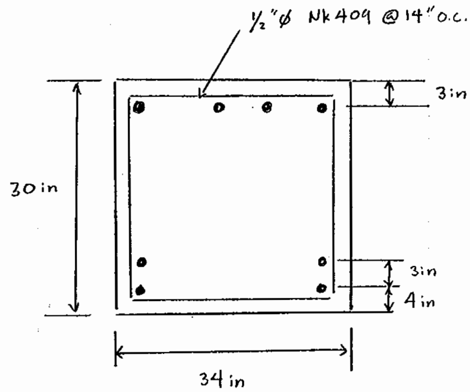
$= 2(1.0)(3500)^{1/2}(206)(11.75)$

$V_c = 286397\ lbs = 286.397\ kips$

$\phi V_c = 0.75(286.397) = 214.798\ kips$

$$\boxed{\begin{array}{l}\text{Negative Shear:}\\ V_n = 286.4\ kips\\ \phi V_n = 214.8\ kips\end{array}}$$

- Bent Cap Capacity:



4 - 1"∅ Nk 801 Top
4 - 1"∅ Nk 803 Bot
(#8 bars)
No. Bent Caps = 11
Max Length = 67 ft
Min Length = 53 ft

$f_y = 40{,}000$ psi
$f_c' = 3500$ psi

o Positive Bending:

I.) $A_s = 4(0.79) = 3.16 \text{ in}^2$

II.) $a = \dfrac{A_s f_y}{0.85 f_c' b} = \dfrac{(3.16)(40)}{(.85)(3.5)(34)}$

$a = 1.24963$ in

III.) $d = 30 - 4 - 1.5 = 24.5$ in

IV.) $\varepsilon_s = .047 \geq \varepsilon_y = .001$ (o.k.)

V.) $M_n = A_s f_y (d - a/2)$
$= (3.16)(40)(24.5 - 1.249/2)$
$M_n = 3017.82$ k-in $= 251.485$ k-ft

$\phi M_n = 226.337$ k-ft

$$\boxed{\begin{array}{l}\text{Positive Bending:}\\ M_n = 251.49 \text{ k-ft}\\ \phi M_n = 226.34 \text{ k-ft}\end{array}}$$

o Negative Bending:

I.) $A_s = 4(0.79) = 3.16 \text{ in}^2$

II.) $a = 1.24963$ in

III.) $d = 30 - 3 = 27$ in

IV.) $\varepsilon_s \geq \varepsilon_y$ (o.k.)

V.) $M_n = A_s f_y (d - a/2)$
$= (3.16)(40)(27 - 1.249/2)$
$M_n = 3333.82$ k-in $= 277.819$ k-ft

$\phi M_n = 250.037$ k-ft

$$\boxed{\begin{array}{l}\text{Negative Bending:}\\ M_n = 277.819 \text{ k-ft}\\ \phi M_n = 250.04 \text{ k-ft}\end{array}}$$

o Shear: (Negative)

I.) $V_c = 2\lambda\sqrt{f_c'}\, b_w d$
$= 2(1.0)(3500)^{1/2}(34)(27)$
$V_c = 108619$ lbs

II.) $V_s = \dfrac{A_v f_{yt} d}{s}$

* ½"∅ @ 14" o.c.

$V_s = \dfrac{(0.20)(40{,}000)(27)}{14}$

$V_s = 15428.6$ lbs

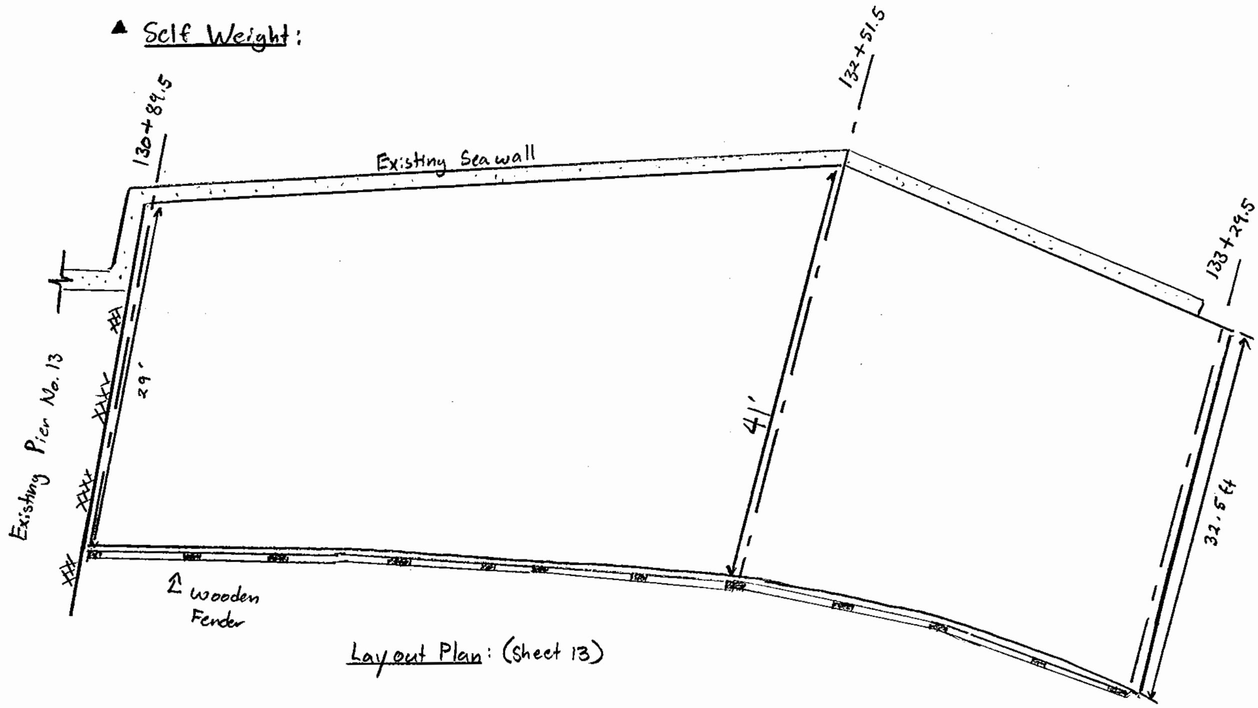
III.) $V_n = V_c + V_s$
$= (108619 + 15428.6)$
$V_n = 124048$ lbs $= 124.048$ kips

$\phi V_n = 93.0357$ kips

$$\boxed{\begin{array}{l}\text{Shear: (Negative)}\\ V_n = 124.05 \text{ kips}\\ \phi V_n = 93.04 \text{ kips}\end{array}}$$

■ Buoyancy Calculations:

▲ Self Weight:



Layout Plan: (Sheet 13)

○ Concrete: (From sheet 1)

Volume = (Deck & Framing) + (Non-Monolithic Side Walk) + Railing
= (521) + (1) + (15)
Volume = 537 yd^3 = 14,499 ft^3

○ Reinforcing Steel:

Weight = (Deck & Framing) + Railing
= 74,631 + 3,037
Weight = 77668 lbs

○ Total Self Weight: ($\gamma_{concrete} = 145\ lb/ft^3$)

Self Weight = Concrete + Steel
= (14499)(145)+(77668)
Self Weight = 2180023 lbs

$\boxed{\text{Total Self Weight} = 2180.0 \text{ kips}}$

- **Buoyant Force:** ($\gamma_{seawater} = 64\ lb/ft^3$)

  * Note: No air will be trapped under the bridge deck as the front (facing the harbor) is open to air, allowing air to escape

  ∴ Air Pocket = 0

  - Submerged Volume = Submerged Concrete + Air Pocket
    $= (521 \times 27) + 0$
    $S\forall = 14067\ ft^3$

  * Note: Assume bridge is submerged to the top of the deck.

  - Buoyant Force:
    $BF = S\forall \times \gamma_{sw}$
    $= (14067)(64)$
    $\boxed{BF = 900288\ lbs}$

- **Residual Weight:**

  Residual Weight = SW − BF
  $= 2180023 - 900288$
  $\boxed{RW = 1279735\ lbs}$

- **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

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Deck Capacity:

$f_y = 40{,}000$ psi
$f_c' = 3500$ psi

15 - ½" ∅ Nk 422 Top
15 - ⅝" ∅ Nk 511 Bot

$b = 206$ in
$h = 14$ in
Cover = 2 in (top & Bot)

* Note: No. Spans = 12
Max Length = 41 ft
Int. Length = 32.5 ft
Min Length = 29 ft

○ Positive Bending: (same as slip #2)

I.) $A_s = 4.65\,in^2$

II.) $a = 0.3035$ in

III.) $d = 11.6875$ in

IV.) $M_n = 178.8$ k-ft
$\phi M_n = 160.9$ k-ft

Positive Bending:
$M_n = 178.8$ k-ft
$\phi M_n = 160.9$ k-ft

○ Negative Bending:

I.) $A_s = (15)(0.20) = 3\,in^2$

II.) $a = \dfrac{A_s f_y}{0.85 f_c' b} = \dfrac{(3)(40{,}000)}{0.85(3500)(206)}$
$a = .195806$ in

III.) $d = 11.75$ in

IV.) $\varepsilon_s > \varepsilon_y$ (o.k.)

V.) $M_n = A_s f_y (d - a/2)$
$= (3)(40)(11.75 - .195806/2)$
$M_n = 1398.25$ k-in $= 116.5$ k-ft

$\phi M_n = 104.869$ k-ft

Negative Bending:
$M_n = 116.521$ k-ft
$\phi M_n = 104.869$ k-ft

○ Shear: (Same as Slip #2)

Shear:
$V_n = 286.4$ kips
$\phi V_n = 214.8$ kips

- Bent Cap Capacity:

No. Bent Caps = 13
Max Length = 41 ft
Int. Length = 32.5 ft
Min Length = 29 ft

- Positive Bending:

M_n = 251.49 k-ft
ϕM_n = 226.34 k-ft

- Negative Bending:

M_n = 277.819 k-ft
ϕM_n = 250.04 k-ft

- Negative Shear:

V_n = 124.05 kips
ϕV_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:

$$F_v = c_{v-va} * F_v^*$$
$$F_v^* = \gamma * (\Delta z_v) * A_v$$

Horizontal Force Equation:

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

Constant Coefficients:

C_r	0.4	
γ	64	lb/cubic ft
c_{v-va}	1	Nonconserv.
c_{v-va}	2	Conserv.
c_{h-va}	1	Nonconserv.
c_{h-va}	2	Conserv.

♦ **Vertical Force Calculations:**

Bridge Deck Width = 68.75 ft
Bridge Deck Length = 48.40 ft

Water Depth = 3.94 ft
Water surface to bot. of girder = 1.00 ft
Height of girder = 3.00 ft

Av = 3327.21 sq ft
N = 12 girders

Elevation to bot. of girder = 4.94 ft
Elevation to bot. of deck = 7.94 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*storm surge depth)*

Wave Height = 2.50 ft *(above storm surge elevation)*

● **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 =	723777	lbs	723.777	kips
Bearing Plates =	433453	lbs	433.453	kips

• Total Vertical Resistance:

Rv = 1157.23 kips

● **Comparison of Vertical Wave Force to Vertical Resistance:**

							Bridge Condition:
Fv =	332.19	kips	<	Rv =	1157.23	kips	O.K.
Fv =	664.38	kips	<	Rv =	1157.23	kips	O.K.

♦ **Horizontal Force Calculations:**

Railing Height = 2.708 ft
Deck Height = 0.667 ft
Girder Height = 3.000 ft

Water Depth = 3.94 ft
Water surface to bot. of girder = 1.00 ft

Bridge Total Height = 6.375 ft
Bridge Length = 48.396 ft

Elevation to bot. of girder = 4.94 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:**

Fh =	52.34	kips	26.17	tons
Fh =	104.68	kips	52.34	tons

- **Horizontal Resistance:** *(from hand calculations)*

  - Sources of Horizontal Resistance:

Self Weight =	723777	lbs
Girder Seat Interface =	Neoprene Pads	
μs =	0.1	
Frictional Resistance =	72377.7	lbs
Bearing Plates =	345312	lbs
Beam Web Capacity =	41011.6	lbs
Koko Head wing wall =	59400	lbs

  - Total Horizontal Resistance:

Rh = 518.10 kips

- **Comparison of Horizontal Wave Force to Horizontal Resistance:**

							Bridge Condition:
Fh =	52.34	kips	<	Rh =	518.10	kips	O.K.
Fh =	104.68	kips	<	Rh =	518.10	kips	O.K.

<u>Kahaluu Bridge:</u>

Method For Estimating Wave Forces on Bridge Superstructures

Douglass (2006)

Vertical Force Equation:

$$F_v = c_{v-va} * F_v^*$$
$$F_v^* = \gamma * (\Delta z_v) * A_v$$

Horizontal Force Equation:

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

Constant Coefficients:

C_r	0.4	
γ	64	lb/cubic ft
c_{v-va}	1	Nonconserv.
c_{v-va}	2	Conserv.
c_{h-va}	1	Nonconserv.
c_{h-va}	2	Conserv.

♦ **<u>Vertical Force Calculations:</u>**

Bridge Deck Width = 46.00 ft
Bridge Deck Length = 318.00 ft

Water Depth = 5.75 ft
Water surface to bot. of girder = 5.00 ft
Height of girder = 4.50 ft

Av = 14628.00 sq ft
N = 8 girders

Elevation to bot. of girder = 10.75 ft
Elevation to bot. of deck = 15.25 ft

Storm Surge Depth + Wave Height = 16.00 ft *(from NFIP Flood Hazard Assessment Tool)*
Storm Surge Depth = 11.00 ft

● **Largest Unbroken Wave:** *(0.455*storm surge depth)*

Wave Height = 5.00 ft *(above storm surge elevation)*

● **Resulting Vertical Force:**

Fv = 2808.58 kips 1404.29 tons (Non Conservative Value)

Fv = 5617.15 kips 2808.58 tons (Conservative Value)

● **Vertical Resistance:** *(from hand calculations)*

• <u>Sources of Vertical Resistance</u>:
Self Weight = 3811550 lbs 3811.55 kips

• <u>Total Vertical Resistance</u>:
Rv = 3811.55 kips

● **Comparison of Vertical Wave Force to Vertical Resistance:**

				Bridge Condition:
Fv = 2808.58 kips	<	Rv = 3811.55 kips		O.K.
Fv = 5617.15 kips	>	Rv = 3811.55 kips		Failure

♦ **<u>Horizontal Force Calculations:</u>**

Railing Height = 2.125 ft
Deck Height = 0.500 ft
Girder Height = 4.500 ft

Water Depth = 5.75 ft
Water surface to bot. of girder = 5.00 ft

Bridge Total Height = 7.125 ft
Bridge Length = 318.000 ft

Elevation to bot. of girder = 10.75 ft
Elevation of centroid of Ah = 14.31 ft

Ah = 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:**

Fh =	929.86	kips	464.93	tons
Fh =	1859.73	kips	929.86	tons

- **Horizontal Resistance:** *(from hand calculations)*

  - Sources of Horizontal Resistance:

Self Weight =	3811550	lbs
Girder Seat Interface =	Steel to Concrete	
μs =	0.4	
Frictional Resistance =	1524620	lbs
Wing Walls =	299260	lbs

  - Total Horizontal Resistance:

Rh = 1823.88 kips

- **Comparison of Horizontal Wave Force to Horizontal Resistance:**

							Bridge Condition:
Fh =	929.86	kips	<	Rh =	1823.88	kips	O.K.
Fh =	1859.73	kips	>	Rh =	1823.88	kips	Failure

New South Punaluu Bridge: *(Span #2)*
Method For Estimating Wave Forces on Bridge Superstructures

Douglass (2006)

Vertical Force Equation:

$$F_v = c_{v-va} * F_v^*$$
$$F_v^* = \gamma * (\Delta z_v) * A_v$$

Horizontal Force Equation:

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

Constant Coefficients:

C_r	0.4	
γ	64	lb/cubic ft
c_{v-va}	1	Nonconserv.
c_{v-va}	2	Conserv.
c_{h-va}	1	Nonconserv.
c_{h-va}	2	Conserv.

♦ **Vertical Force Calculations:**

Bridge Deck Width = 50.00 ft
Bridge Deck Length = 66.00 ft

Water Depth = 4.92 ft
Water surface to bot. of girder = 0.00 ft
Height of girder = 1.75 ft

Av = 3300.00 sq ft
N = 30 girders

Elevation to bot. of girder = 4.92 ft
Elevation to bot. of deck = 6.67 ft

Storm Surge Depth + Wave Height = 12.00 ft *(from NFIP Flood Hazard Assessment Tool)*
Storm Surge Depth = 8.25 ft

● **Largest Unbroken Wave:** *(0.455*storm surge depth)*

Wave Height = 3.75 ft *(above storm surge elevation)*

● **Resulting Vertical Force:**

Fv = 1125.70 kips 562.85 tons (Non Conservative Value)

Fv = 2251.39 kips 1125.70 tons (Conservative Value)

● **Vertical Resistance:** *(from hand calculations)*

• Sources of Vertical Resistance:
Deck Capacity = 1262990 lbs 1262.99 kips

• Total Vertical Resistance:
Rv = 1262.99 kips

● **Comparison of Vertical Wave Force to Vertical Resistance:**

						Bridge Condition:
Fv =	1125.70 kips	<	Rv =	1262.99 kips		O.K.
Fv =	2251.39 kips	>	Rv =	1262.99 kips		Failure

♦ **Horizontal Force Calculations:**

Railing Height = 3.000 ft
Deck Height = 0.875 ft
Girder Height = 1.750 ft

Water Depth = 4.92 ft
Water surface to bot. of girder = 0.00 ft

Bridge Total Height = 5.625 ft
Bridge Length = 66.000 ft

Elevation to bot. of girder = 4.92 ft
Elevation of centroid of Ah = 7.73 ft

Ah = 371.25 sq ft
N = 30 girders

● **Largest Unbroken Wave:** *(0.455*storm surge depth)*

Wave Height = 3.75 ft *(above storm surge elevation)*

● **Resulting Horizontal Force:**

Fh =	1277.59	kips	638.79	tons
Fh =	2555.17	kips	1277.59	tons

● **Horizontal Resistance:** *(from hand calculations)*

• Sources of Horizontal Resistance:

Self Weight = 907306 lbs *(for span #2)*

Girder Seat Interface = Concrete to Concrete

μs = 0.8

Frictional Resistance = 725844.8 lbs

• Total Horizontal Resistance:

Rh = 725.84 kips

● **Comparison of Horizontal Wave Force to Horizontal Resistance:**

							Bridge Condition:
Fh =	1277.59	kips	>	Rh =	725.84	kips	Failure
Fh =	2555.17	kips	>	Rh =	725.84	kips	Failure

New Makaha No. 3A Bridge:

Method For Estimating Wave Forces on Bridge Superstructures

Douglass (2006)

Vertical Force Equation:

$$F_v = c_{v-va} * F_v^*$$
$$F_v^* = \gamma * (\Delta z_v) * A_v$$

Horizontal Force Equation:

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

Constant Coefficients:

C_r	0.4	
γ	64	lb/cubic ft
c_{v-va}	1	Nonconserv.
c_{v-va}	2	Conserv.
c_{h-va}	1	Nonconserv.
c_{h-va}	2	Conserv.

♦ **Vertical Force Calculations:**

Bridge Deck Width = 46.83 ft
Bridge Deck Length = 70.00 ft

Water Depth = 1.00 ft
Water surface to bot. of girder = 9.59 ft
Height of girder = 2.33 ft

Av = 3278.33 sq ft
N = 1 girders

Elevation to bot. of girder = 10.59 ft
Elevation to bot. of deck = 10.59 ft

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*storm surge depth)*

Wave Height = 4.07 ft *(above storm surge elevation)*

● **Resulting Vertical Force:**

Fv = 505.65 kips 252.83 tons (Non Conservative Value)

Fv = 1011.30 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:
Rv = 1127.13 kips

● **Comparison of Vertical Wave Force to Vertical Resistance:**

						Bridge Condition:
Fv =	505.65	kips	<	Rv =	1127.13 kips	O.K.
Fv =	1011.30	kips	<	Rv =	1127.13 kips	O.K.

♦ **Horizontal Force Calculations:**

Railing Height = 1.167 ft
Deck Height = 0.458 ft
Girder Height = 2.333 ft

Water Depth = 1.00 ft
Water surface to bot. of girder = 9.59 ft

Bridge Total Height = 3.958 ft
Bridge Length = 70.000 ft

Elevation to bot. of girder = 10.59 ft
Elevation of centroid of Ah = 12.57 ft

Ah = 277.08 sq ft
N = 1 girders

● **Largest Unbroken Wave:** *(0.455*storm surge depth)*

Wave Height = 4.07 ft *(above storm surge elevation)*

• **Resulting Horizontal Force:**

Fh =	7.64	kips	3.82	tons
Fh =	15.28	kips	7.64	tons

• **Horizontal Resistance:** *(from hand calculations)*

• Sources of Horizontal Resistance:

Self Weight = 1161770 lbs

Girder Seat Interface = Concrete to Concrete

μs = 0.8

Frictional Resistance = 929416 lbs

Shear Friction = 8870400 lbs

• Total Horizontal Resistance:

Rh = 9799.82 kips

• **Comparison of Horizontal Wave Force to Horizontal Resistance:**

							Bridge Condition:
Fh =	7.64	kips	<	Rh =	9799.82	kips	O.K.
Fh =	15.28	kips	<	Rh =	9799.82	kips	O.K.

Old Makaha No. 3A Bridge:

Method For Estimating Wave Forces on Bridge Superstructures

Douglass (2006)

Vertical Force Equation:

$$F_v = c_{v-va} * F_v^*$$
$$F_v^* = \gamma * (\Delta z_v) * A_v$$

Horizontal Force Equation:

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

Constant Coefficients:

C_r	0.4	
γ	64	lb/cubic ft
c_{v-va}	1	Nonconserv.
c_{v-va}	2	Conserv.
c_{h-va}	1	Nonconserv.
c_{h-va}	2	Conserv.

♦ **Vertical Force Calculations:**

Bridge Deck Width = 32.83 ft
Bridge Deck Length = 78.83 ft

Water Depth = 4.00 ft
Water surface to bot. of girder = 7.24 ft
Height of girder = 1.50 ft

Av = 2588.36 sq ft
N = 12 girders

Elevation to bot. of girder = 11.24 ft
Elevation to bot. of deck = 12.74 ft

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*storm surge depth)*

Wave Height = 4.07 ft *(above storm surge elevation)*

● **Resulting Vertical Force:**

Fv = 43.07 kips 21.54 tons (Non Conservative Value)

Fv = 86.14 kips 43.07 tons (Conservative Value)

● **Vertical Resistance:** *(from hand calculations)*

• Sources of Vertical Resistance:

Self Weight = 279830 lbs 279.83 kips
Drift Bolt Withdrawal Capacity = 69300 lbs 69.3 kips

• Total Vertical Resistance:

Rv = 349.13 kips

● **Comparison of Vertical Wave Force to Vertical Resistance:**

							Bridge Condition:
Fv =	43.07	kips	<	Rv =	279.83	kips	O.K.
Fv =	86.14	kips	<	Rv =	279.83	kips	O.K.

♦ **Horizontal Force Calculations:**

Railing Height = 0.000 ft
Deck Height = 0.500 ft
Girder Height = 1.500 ft

Water Depth = 4.00 ft
Water surface to bot. of girder = 7.24 ft

Bridge Total Height = 2.000 ft
Bridge Length = 78.833 ft

Elevation to bot. of girder = 11.24 ft
Elevation of centroid of Ah = 12.24 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:**

Fh =	41.41	kips	20.71	tons
Fh =	82.82	kips	41.41	tons

• **Horizontal Resistance:** *(from hand calculations)*

• Sources of Horizontal Resistance:

Self Weight = 279830 lbs

Girder Seat Interface = Wood to Wood (wet)

μs = 0.2

Frictional Resistance = 55966 lbs

Drift Bolts Lateral Capacity = 14405.4 lbs

• Total Horizontal Resistance:

Rh = 70.37 kips

• **Comparison of Horizontal Wave Force to Horizontal Resistance:**

							Bridge Condition:
Fh =	41.41	kips	<	Rh =	70.37	kips	O.K.
Fh =	82.82	kips	>	Rh =	70.37	kips	Failure

Maipalaoa Stream Bridge: (Maili Channel)

Method For Estimating Wave Forces on Bridge Superstructures

Douglass (2006)

Vertical Force Equation:

$$F_v = c_{v-va} * F_v^*$$
$$F_v^* = \gamma * (\Delta z_v) * A_v$$

Horizontal Force Equation:

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

Constant Coefficients:

C_r	0.4	
γ	64	lb/cubic ft
c_{v-va}	1	Nonconserv.
c_{v-va}	2	Conserv.
c_{h-va}	1	Nonconserv.
c_{h-va}	2	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
Av =	6476.22	sq ft	Elevation to bot. of girder =	6.50	ft
N =	16	girders	Elevation to bot. of deck =	9.50	ft
			Storm Surge Depth + Wave Height =	12.00	ft *(from NFIP Flood Hazard Assessment Tool)*
			Storm Surge Depth =	8.25	ft

● **Largest Unbroken Wave:** *(0.455*storm surge depth)*

Wave Height = 3.75 ft *(above storm surge elevation)*

● **Resulting Vertical Force:**

Fv =	1036.20	kips	518.10	tons	(Non Conservative Value)
Fv =	2072.39	kips	1036.20	tons	(Conservative Value)

● **Vertical Resistance:** *(from hand calculations)*

• Sources of Vertical Resistance:

Self Weight = 1406690 lbs 1406.69 kips

• Total Vertical Resistance:

Rv = 1406.69 kips

● **Comparison of Vertical Wave Force to Vertical Resistance:**

								Bridge Condition:
Fv =	1036.20	kips	<	Rv =	1406.69	kips		O.K.
Fv =	2072.39	kips	>	Rv =	1406.69	kips		Failure

♦ **Horizontal Force Calculations:**

Railing Height =	2.000	ft	Water Depth =	3.00	ft
Deck Height =	0.500	ft	Water surface to bot. of girder =	3.50	ft
Girder Height =	3.000	ft			
Bridge Total Height =	5.500	ft	Elevation to bot. of girder =	6.50	ft
Bridge Length =	100.667	ft	Elevation of centroid of Ah =	9.25	ft
Ah =	553.67	sq ft			
N =	16	girders			

● **Largest Unbroken Wave:** *(0.455*storm surge depth)*

Wave Height = 3.75 ft *(above storm surge elevation)*

• **Resulting Horizontal Force:**

Fh =	682.12	kips	341.06	tons
Fh =	1364.23	kips	682.12	tons

• **Horizontal Resistance:** *(from hand calculations)*

• Sources of Horizontal Resistance:

Self Weight =	1406690	lbs
Girder Seat Interface =	neoprene pad	
μs =	0.1	
Frictional Resistance =	140669	lbs
Beam Web Flexural Capacity =	92016	lbs

• Total Horizontal Resistance:

Rh = 232.69 kips

• **Comparison of Horizontal Wave Force to Horizontal Resistance:**

							Bridge Condition:
Fh =	682.12	kips	>	Rh =	232.69	kips	Failure
Fh =	1364.23	kips	>	Rh =	232.69	kips	Failure

Moanalua Stream Bridge: (Spans 3, 4, 5 & 6)
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 8.

Douglass (2006)

Vertical Force Equation:

$$F_v = c_{v-va} * F_v^*$$
$$F_v^* = \gamma * (\Delta z_v) * A_v$$

Horizontal Force Equation:

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

Constant Coefficients:

C_r	0.4	
γ	64	lb/cubic ft
c_{v-va}	1	Nonconserv.
c_{v-va}	2	Conserv.
c_{h-va}	1	Nonconserv.
c_{h-va}	2	Conserv.

♦ **Vertical Force Calculations:**

Bridge Deck Width = 64.33 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

Av = 1737.00 sq ft
N = 9 girders

Elevation to bot. of girder = 6.50 ft
Elevation to bot. of deck = 8.33 ft

Storm Surge Depth + Wave Height = 10.00 ft *(from NFIP Flood Hazard Assessment Tool)*
Storm Surge Depth = 6.87 ft

● **Largest Unbroken Wave:** *(0.455*storm surge depth)*

Wave Height = 3.13 ft *(above storm surge elevation)*

● **Resulting Vertical Force:**

Fv = 185.28 kips 92.64 tons (Non Conservative Value)

Fv = 370.56 kips 185.28 tons (Conservative Value)

● **Vertical Resistance:** *(from hand calculations)*

• Sources of Vertical Resistance:
Self Weight = 417270 lbs 417.27 kips
Pier Cap Weight = 158400 lbs

• Total Vertical Resistance:
Rv = 575.67 kips

● **Comparison of Vertical Wave Force to Vertical Resistance:**

						Bridge Condition:
Fv =	185.28 kips	<	Rv =	575.67	kips	O.K.
Fv =	370.56 kips	<	Rv =	575.67	kips	O.K.

♦ **Horizontal Force Calculations:**

Railing Height = 3.729 ft
Deck Height = 0.667 ft
Girder Height = 1.833 ft

Water Depth = 2.50 ft
Water surface to bot. of girder = 4.00 ft

Bridge Total Height = 6.229 ft
Bridge Length = 27.000 ft

Elevation to bot. of girder = 6.50 ft
Elevation of centroid of Ah = 9.61 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)*

● **Resulting Horizontal Force:**

Fh =	17.42	kips	8.71	tons
Fh =	34.85	kips	17.42	tons

● **Horizontal Resistance:** *(from hand calculations)*

• Sources of Horizontal Resistance:

Self Weight = 417270 lbs
Girder Seat Interface = concrete to concrete
μs = 0.8
Frictional Resistance = 333816 lbs

Dowel Shear Friction = 1592640 lbs

• Total Horizontal Resistance:

Rh = 1926.46 kips

● **Comparison of Horizontal Wave Force to Horizontal Resistance:**

							Bridge Condition:
Fh =	17.42	kips	<	Rh =	1926.46	kips	O.K.
Fh =	34.85	kips	<	Rh =	1926.46	kips	O.K.

*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.

Kalihi 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:

$$F_v = c_{v-va} * F_v^*$$
$$F_v^* = \gamma * (\Delta z_v) * A_v$$

Horizontal Force Equation:

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

Constant Coefficients:

C_r	0.4	
γ	64	lb/cubic ft
c_{v-va}	1	Nonconserv.
c_{v-va}	2	Conserv.
c_{h-va}	1	Nonconserv.
c_{h-va}	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

Av = 2385.00 sq ft
N = 13 girders

Elevation to bot. of girder = 6.50 ft
Elevation to bot. of deck = 8.33 ft

Storm Surge Depth + Wave Height = 10.00 ft *(from NFIP Flood Hazard Assessment Tool)*
Storm Surge Depth = 6.87 ft

● **Largest Unbroken Wave:** *(0.455*storm surge depth)*

Wave Height = 3.13 ft *(above storm surge elevation)*

● **Resulting Vertical Force:**

Fv = 254.40 kips 127.20 tons (Non Conservative Value)

Fv = 508.80 kips 254.40 tons (Conservative Value)

● **Vertical Resistance:** *(from hand calculations)*

• Sources of Vertical Resistance:
Self Weight = 565087 lbs 565.087 kips

• Total Vertical Resistance:
Rv = 565.087 kips

● **Comparison of Vertical Wave Force to Vertical Resistance:**

						Bridge Condition:
Fv =	254.40 kips	<	Rv =	565.087	kips	O.K.
Fv =	508.80 kips	<	Rv =	565.087	kips	O.K.

♦ **Horizontal Force Calculations:**

Railing Height = 3.729 ft
Deck Height = 0.667 ft
Girder Height = 1.833 ft

Water Depth = 2.50 ft
Water surface to bot. of girder = 4.00 ft

Bridge Total Height = 6.229 ft
Bridge Length = 27.000 ft

Elevation to bot. of girder = 6.50 ft
Elevation of centroid of Ah = 9.61 ft

Ah = 168.19 sq ft
N = 13 girders

● **Largest Unbroken Wave:** *(0.455*storm surge depth)*

Wave Height = 3.13 ft *(above storm surge elevation)*

• **Resulting Horizontal Force:**

Fh =	24.06	kips	12.03	tons
Fh =	48.12	kips	24.06	tons

• **Horizontal Resistance:** *(from hand calculations)*

• Sources of Horizontal Resistance:

Self Weight = 565087 lbs

Girder Seat Interface = concrete to concrete

μs = 0.8

Frictional Resistance = 452069.6 lbs

• Total Horizontal Resistance:

Rh = 452.07 kips

• **Comparison of Horizontal Wave Force to Horizontal Resistance:**

							Bridge Condition:
Fh =	24.06	kips	<	Rh =	452.07	kips	O.K.
Fh =	48.12	kips	<	Rh =	452.07	kips	O.K.

Nimitz Highway at Aloha Tower Slip Cover #2:
Method For Estimating Wave Forces on Bridge Superstructures

Douglass (2006)

Vertical Force Equation:

$$F_v = c_{v-va} * F_v^*$$
$$F_v^* = \gamma * (\Delta z_v) * A_v$$

Horizontal Force Equation:

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

Constant Coefficients:

C_r	0.4	
γ	64	lb/cubic ft
c_{v-va}	1	Nonconserv.
c_{v-va}	2	Conserv.
c_{h-va}	1	Nonconserv.
c_{h-va}	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
Av =	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*storm surge depth)*

Wave Height = 2.50 ft *(above storm surge elevation)*

• **Resulting Vertical Force:**

Fv =	0.00	kips	0.00	tons	(Non Conservative Value)
Fv =	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:
Rv = 2069.45 kips

• **Comparison of Vertical Wave Force to Vertical Resistance:**

							Bridge Condition:
Fv =	0.00	kips	<	Rv =	2069.45	kips	O.K.
Fv =	0.00	kips	<	Rv =	2069.45	kips	O.K.

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	Water Depth =	5.33	ft
Deck Height =	1.292	ft	Water surface to bot. of girder =	4.50	ft
Girder Height =	2.500	ft			
Bridge Total Height =	5.458	ft	Elevation to bot. of girder =	9.83	ft
Bridge Length =	178.410	ft	Elevation of centroid of Ah =	12.56	ft
Ah =	973.82	sq ft			
N =	11	girders			

• **Largest Unbroken Wave:** *(0.455*storm surge depth)*

Wave Height = 2.50 ft *(above storm surge elevation)*

• **Resulting Horizontal Force:**

Fh =	0.00	kips	0.00	tons
Fh =	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	
μs =	0.8	
Frictional Resistance =	1655562	lbs

• Total Horizontal Resistance:

Rh = 1655.56 kips

• **Comparison of Horizontal Wave Force to Horizontal Resistance:**

							Bridge Condition:
Fh =	0.00	kips	<	Rh =	1655.56	kips	O.K.
Fh =	0.00	kips	<	Rh =	1655.56	kips	O.K.

Note: With 100 year flood data from the NFIP,
the waves will miss the bottom elevation of the bridge

Nimitz Highway at Aloha Tower Slip Cover #3:
Method For Estimating Wave Forces on Bridge Superstructures

Douglass (2006)

Vertical Force Equation:

$$F_v = c_{v-va} * F_v^*$$
$$F_v^* = \gamma * (\Delta z_v) * A_v$$

Horizontal Force Equation:

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

Constant Coefficients:

C_r	0.4	
γ	64	lb/cubic ft
c_{v-va}	1	Nonconserv.
c_{v-va}	2	Conserv.
c_{h-va}	1	Nonconserv.
c_{h-va}	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				

Av =	8536.50	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*storm surge depth)*

Wave Height = 2.50 ft *(above storm surge elevation)*

● **Resulting Vertical Force:**

Fv =	0.00	kips	0.00	tons	(Non Conservative Value)
Fv =	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:
Rv = 2180.02 kips

● **Comparison of Vertical Wave Force to Vertical Resistance:**

							Bridge Condition:
Fv =	0.00	kips	<	Rv =	2180.02	kips	O.K.
Fv =	0.00	kips	<	Rv =	2180.02	kips	O.K.

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		Water Depth =	5.33	ft
Deck Height =	1.292 ft		Water surface to bot. of girder =	4.50	ft
Girder Height =	2.500 ft				
Bridge Total Height =	5.458 ft		Elevation to bot. of girder =	9.83	ft
Bridge Length =	240.000 ft		Elevation of centroid of Ah =	12.56	ft

Ah = 1310.00 sq ft
N = 13 girders

● **Largest Unbroken Wave:** *(0.455*storm surge depth)*

Wave Height = 2.50 ft *(above storm surge elevation)*

● **Resulting Horizontal Force:**

Fh =	0.00	kips	0.00	tons
Fh =	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

μs = 0.8

Frictional Resistance = 1744018 lbs

• Total Horizontal Resistance:

Rh = 1744.02 kips

● **Comparison of Horizontal Wave Force to Horizontal Resistance:**

							Bridge Condition:
Fh =	0.00	kips	<	Rh =	1744.02	kips	O.K.
Fh =	0.00	kips	<	Rh =	1744.02	kips	O.K.

Note: With 100 year flood data from the NFIP,
the waves will miss the bottom elevation of the bridge

Appendix C: McPherson Wave Estimation Method Calculations

Kuliouou Bridge:
Method For Estimating Wave Forces on Bridge Superstructures

McPherson (2008)

Vertical Force Equation:

- $F_{Total} = F_{Hydrostatic} + F_{Bridge} + F_{AirEntrapment}$

- $F_{Hydrostatic} = \gamma \delta_z A - F_w$

  If $h \leq h\_model$

  $F_w = \frac{1}{2}\gamma\delta A$

  If $h > h\_model$

  $F_w = \frac{1}{2}\gamma\delta A + \gamma(h - h_{model})A$

- $F_{Bridge} = \gamma Vol_{Bridge}$

- $F_{AirEntrapment} = (n-1)0.5\gamma\delta_G A_G$

Horizontal Force Equation:

- $F_{Total} = F_{Hydrostatic\_Front} + F_{Hydrostatic\_Back}$

If $\eta_{max} < h_{deck}$

$F_{Hydrostatic\_Front} = 0.5 * (\eta_{max} + h - h_{girders}) H_{bridge} L_{bridge} \gamma$

If $\eta_{max} > h_{deck}$

$F_{Hydrostatic\_Front} = 0.5 * [(\eta_{max} + h - h_{girders}) + (\eta_{max} - h_{deck})] H_{bridge} L_{bridge} \gamma$

If $SWL < h_{girders}$

$F_{Hydrostatic\_back} = 0$

If $SWL > h_{girders}$

$F_{Hydrostatic\_back} = 0.5(h - h_{girder})^2 L_{bridge} \gamma$

Constant Coefficients:

γ 64 lb/cubic ft

♦ **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
Area (A) =	3327.21	sq ft	Height of girder =	3.00	ft
Vol_bridge =	4825.18	cubic ft	Deck Height =	0.667	ft
Vol_trapped air/2 =	3475.35	cubic ft	Railing Height =	2.708	ft
n =	12	girders	Elevation to bot. of girder =	4.94	ft
			Elevation to bot. of deck =	7.94	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*storm surge depth)*

Wave Height = 2.50 ft *(above storm surge elevation)*

● **Equation Values:**

δz = -0.61 ft *(Difference between top of deck and highest point on wave)*

Fw = 0 lbs
Fhydrostatic = 0 lbs

Fbridge = 308811.52 lbs

Fair_entrapment = 222422.08 lbs

● **Resulting Vertical Force:**

Fv = 531.23 kips 265.62 tons

● **Vertical Resistance:** *(from hand calculations)*

- Sources of Vertical Resistance:

Self Weight =	723777	lbs	723.78	kips
Bearing Plates =	433453	lbs	433.45	kips

- Total Vertical Resistance:

  Rv = 1157.23 kips

● **Comparison of Vertical Wave Force to Vertical Resistance:**

Fv = 531.23 kips < Rv = 1157.23 kips

Bridge Condition:
O.K.

♦ **Horizontal Force Calculations:**

Railing Height =	2.708	ft	Water Depth =	3.94	ft
Deck Height =	0.667	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 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:**

η_max =	2.50	ft	*(Wave height above storm surge elevation)*
H_bridge =	6.38	ft	
h =	5.50	ft	*(SWL)*
h_girder =	4.94	ft	
h_deck =	7.94	ft	

F_hydrostatic_front =	30210.615	lbs	η_max	<	h_deck
F_hydrostatic_back =	482.686	lbs	SWL	>	h_girder

● **Resulting Vertical Force:**

Fh = 29.73 kips 14.86 tons

● **Horizontal Resistance:** *(from hand calculations)*

• Sources of Horizontal Resistance:

Self Weight =	723777	lbs
Girder Seat Interface =	Neoprene Pads	
μs =	0.1	
Frictional Resistance =	72377.7	lbs
Bearing Plates =	345312	lbs
Beam Web Capacity =	41010	lbs
Koko Head wing wall =	59400	lbs

• Total Horizontal Resistance:

Rh = 518.10 kips

● **Comparison of Horizontal Wave Force to Horizontal Resistance:**

							Bridge Condition:
Fh =	29.73	kips	<	Rh =	518.10	kips	O.K.

Kahaluu Bridge:

Method For Estimating Wave Forces on Bridge Superstructures

McPherson (2008)

Vertical Force Equation:

- $F_{Total} = F_{Hydrostatic} + F_{Bridge} + F_{AirEntrapment}$
- $F_{Hydrostatic} = \gamma\delta_z A - F_w$

If $h \leq h\_model$

$F_w = \frac{1}{2}\gamma\delta A$

If $h > h\_model$

$F_w = \frac{1}{2}\gamma\delta A + \gamma(h - h_{model})A$

- $F_{Bridge} = \gamma Vol_{Bridge}$
- $F_{AirEntrapment} = (n-1)0.5\gamma\delta_G A_G$

Horizontal Force Equation:

- $F_{Total} = F_{Hydrostatic\_Front} + F_{Hydrostatic\_Back}$

If $\eta_{max} < h_{deck}$

$F_{Hydrostatic\_Front} = 0.5 * (\eta_{max} + h - h_{girders})H_{bridge}L_{bridge}\gamma$

If $\eta_{max} > h_{deck}$

$F_{Hydrostatic\_Front} = 0.5 * [(\eta_{max} + h - h_{girders}) + (\eta_{max} - h_{deck})]H_{bridge}L_{bridge}\gamma$

If $SWL < h_{girders}$

$F_{Hydrostatic\_back} = 0$

If $SWL > h_{girders}$

$F_{Hydrostatic\_back} = 0.5(h - h_{girder})^2 L_{bridge}\gamma$

Constant Coefficients:

γ 64 lb/cubic ft

♦ **Vertical Force Calculations:**

Bridge Deck Width =	46.00	ft	Water Depth =	5.75	ft
Bridge Deck Length =	318.00	ft	Water surface to bot. of girder =	5.00	ft
Area (A) =	14628	sq ft	Height of girder =	4.50	ft
Vol_bridge =	23960.2	cubic ft	Deck Height =	0.500	ft
Vol_trapped air/2 =	19161.1	cubic ft	Railing Height =	2.125	ft
n =	8	girders	Elevation to bot. of girder =	10.75	ft
			Elevation to bot. of deck =	15.25	ft
			Storm Surge Depth + Wave Height =	16.00	ft
			Storm Surge Depth =	11.00	ft

(from NFIP Flood Hazard Assessment Tool)

- **Largest Unbroken Wave:** *(0.455*storm surge depth)*

Wave Height = 5.00 ft *(above storm surge elevation)*

- **Equation Values:**

δz = 0.25 ft *(Difference between top of deck and highest point on wave)*

Fw = 117024 lbs

Fhydrostatic = 117024 lbs

Fbridge = 1533452.8 lbs

Fair_entrapment = 1226310.4 lbs

- **Resulting Vertical Force:**

Fv = 2876.79 kips 1438.39 tons

- **Vertical Resistance:** *(from hand calculations)*

  - Sources of Vertical Resistance:

    Self Weight = 3811550 lbs 3811.55 kips

  - Total Vertical Resistance:

    Rv = 3811.55 kips

- **Comparison of Vertical Wave Force to Vertical Resistance:**

Fv = 2876.79 kips < Rv = 3811.55 kips

Bridge Condition:
O.K.

♦ **Horizontal Force Calculations:**

Railing Height =	2.125	ft	Water Depth =	5.75	ft
Deck Height =	0.500	ft	Water surface to bot. of girder =	5.00	ft
Girder Height =	4.500	ft			
Bridge Total Height =	7.125	ft	Elevation to bot. of girder =	10.75	ft
Bridge Length =	318.000	ft	Elevation to bot. of deck =	15.25	ft

Ah =	2265.75	sq ft
N =	8	girders

● **Largest Unbroken Wave:** *(0.455*storm surge depth)*

Wave Height =	5.00	ft	*(above storm surge elevation)*

● **Equation Values:**

η_max =	5.00	ft	*(Wave height above storm surge elevation)*
H_bridge =	7.13	ft	
h =	11.00	ft	*(SWL)*
h_girder =	10.75	ft	
h_deck =	15.25	ft	

F_hydrostatic_front =	380646	lbs	η_max	<	h_deck
F_hydrostatic_back =	618.636	lbs	SWL	>	h_girder

● **Resulting Vertical Force:**

Fh = 380.03 kips 190.01 tons

● **Horizontal Resistance:** *(from hand calculations)*

- Sources of Horizontal Resistance:

Self Weight =	3811550	lbs
Girder Seat Interface =	Steel to Concrete	
μs =	0.4	
Frictional Resistance =	1524620	lbs
Wing Walls =	299260	lbs

- Total Horizontal Resistance:

Rh = 1823.88 kips

● **Comparison of Horizontal Wave Force to Horizontal Resistance:**

							Bridge Condition:
Fh =	380.03	kips	<	Rh =	1823.88	kips	O.K.

New South Punaluu Bridge: (span #2)
Method For Estimating Wave Forces on Bridge Superstructures

McPherson (2008)

Vertical Force Equation:

○ $F_{Total} = F_{Hydrostatic} + F_{Bridge} + F_{AirEntrapment}$

○ $F_{Hydrostatic} = \gamma \delta_z A - F_w$

If $h \leq h\_model$

$F_w = \frac{1}{2}\gamma\delta A$

If $h > h\_model$

$F_w = \frac{1}{2}\gamma\delta A + \gamma(h - h_{model})A$

○ $F_{Bridge} = \gamma Vol_{Bridge}$

○ $F_{AirEntrapment} = (n-1)0.5\gamma\delta_G A_G$

Horizontal Force Equation:

○ $F_{Total} = F_{Hydrostatic\_Front} + F_{Hydrostatic\_Back}$

If $\eta_{max} < h_{deck}$

$F_{Hydrostatic\_Front} = 0.5 * (\eta_{max} + h - h_{girders})H_{bridge}L_{bridge}\gamma$

If $\eta_{max} > h_{deck}$

$F_{Hydrostatic\_Front} = 0.5 * [(\eta_{max} + h - h_{girders}) + (\eta_{max} - h_{deck})]H_{bridge}L_{bridge}\gamma$

If $SWL < h_{girders}$

$F_{Hydrostatic\_back} = 0$

If $SWL > h_{girders}$

$F_{Hydrostatic\_back} = 0.5(h - h_{girder})^2 L_{bridge}\gamma$

Constant Coefficients:

γ 64 lb/cubic ft

♦ **Vertical Force Calculations:**

Bridge Deck Width =	50.00	ft	Water Depth =	4.92	ft
Bridge Deck Length =	66.00	ft	Water surface to bot. of girder =	0.00	ft
Area (A) =	3300	sq ft	Height of girder =	1.75	ft
Vol_bridge =	14662.83	cubic ft	Deck Height =	0.875	ft
Vol_trapped air/2 =	4402.245	cubic ft	Railing Height =	3.000	ft
n =	30	girders	Elevation to bot. of girder =	4.92	ft
			Elevation to bot. of deck =	6.67	ft
			Storm Surge Depth + Wave Height =	12.00	ft
			Storm Surge Depth =	8.25	ft

(from NFIP Flood Hazard Assessment Tool)

● **Largest Unbroken Wave:** *(0.455*storm surge depth)*

Wave Height = 3.75 ft *(above storm surge elevation)*

● **Equation Values:**

δz = 4.46 ft *(Difference between top of deck and highest point on wave)*

Fw = 470448 lbs
Fhydrostatic = 470448 lbs

Fbridge = 938421.12 lbs

Fair_entrapment = 281743.68 lbs

● **Resulting Vertical Force:**

Fv = 1690.61 kips 845.31 tons

● **Vertical Resistance:** *(from hand calculations)*

• Sources of Vertical Resistance:
Deck Capacity = 1262990 lbs 1262.99 kips

• Total Vertical Resistance:
Rv = 1262.99 kips

● **Comparison of Vertical Wave Force to Vertical Resistance:**

Fv = 1690.61 kips > Rv = 1262.99 kips

Bridge Condition:
Failure

♦ **Horizontal Force Calculations:**

Railing Height =	3.000	ft	Water Depth =	4.92	ft
Deck Height =	0.875	ft	Water surface to bot. of girder =	0.00	ft
Girder Height =	1.750	ft			
Bridge Total Height =	5.625	ft	Elevation to bot. of girder =	4.92	ft
Bridge Length =	66.000	ft	Elevation to bot. of deck =	6.67	ft

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:**

η_max =	3.20	ft	*(Wave height above storm surge elevation)*
H_bridge =	5.63	ft	
h =	8.25	ft	
h_girder =	4.92	ft	
h_deck =	6.67	ft	

F_hydrostatic_front =	77522.0214	lbs	η_max	<	h_deck
F_hydrostatic_back =	23383.518	lbs	SWL	>	h_girder

- **Resulting Vertical Force:**

Fh =	54.14	kips	27.07	tons

- **Horizontal Resistance:** *(from hand calculations)*

  - Sources of Horizontal Resistance:

Self Weight =	907302	lbs
Girder Seat Interface =	Concrete to Concrete	
μs =	0.8	
Frictional Resistance =	725841.6	lbs
Bearing Pads =	766770	lbs

  - Total Horizontal Resistance:

Rh =	1492.61	kips

- **Comparison of Horizontal Wave Force to Horizontal Resistance:**

							Bridge Condition:
Fh =	54.14	kips	<	Rh =	1492.61	kips	O.K.

New Makaha No.3A:

Method For Estimating Wave Forces on Bridge Superstructures

McPherson (2008)

Vertical Force Equation:

- $F_{Total} = F_{Hydrostatic} + F_{Bridge} + F_{AirEntrapment}$
- $F_{Hydrostatic} = \gamma \delta_z A - F_w$

If $h \leq h\_model$

$F_w = \frac{1}{2}\gamma\delta A$

If $h > h\_model$

$F_w = \frac{1}{2}\gamma\delta A + \gamma(h - h_{model})A$

- $F_{Bridge} = \gamma Vol_{Bridge}$
- $F_{AirEntrapment} = (n-1)0.5\gamma\delta_G A_G$

Horizontal Force Equation:

- $F_{Total} = F_{Hydrostatic\_Front} + F_{Hydrostatic\_Back}$

If $\eta_{max} < h_{deck}$

$F_{Hydrostatic\_Front} = 0.5 * (\eta_{max} + h - h_{girders}) H_{bridge} L_{bridge} \gamma$

If $\eta_{max} > h_{deck}$

$F_{Hydrostatic\_Front} = 0.5 * [(\eta_{max} + h - h_{girders}) + (\eta_{max} - h_{deck})] H_{bridge} L_{bridge} \gamma$

If $SWL < h_{girders}$

$F_{Hydrostatic\_back} = 0$

If $SWL > h_{girders}$

$F_{Hydrostatic\_back} = 0.5(h - h_{girder})^2 L_{bridge} \gamma$

Constant Coefficients:

γ 64 lb/cubic ft

♦ **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
Area (A) =	3278.33	sq ft	Height of girder =	2.33	ft
Vol_bridge =	7745.12	cubic ft	Deck Height =	0.458	ft
Vol_trapped air/2 =	0	cubic ft	Railing Height =	1.167	ft
n =	1	girders	Elevation to bot. of girder =	10.59	ft
			Elevation to bot. of deck =	12.92	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*storm surge depth)*

Wave Height = 4.07 ft *(above storm surge elevation)*

- **Equation Values:**

δz = -0.38 ft *(Difference between top of deck and highest point on wave)*

Fw = 0 lbs

Fhydrostatic = 0 lbs

Fbridge = 495687.68 lbs

Fair_entrapment = 0 lbs

- **Resulting Vertical Force:**

Fv = 495.69 kips 247.84 tons

- **Vertical Resistance:** *(from hand calculations)*

  - Sources of Vertical Resistance:

  Deck Flexture Capacity = 1127130 lbs 1127.13 kips

  - Total Vertical Resistance:

  Rv = 1127.13 kips

- **Comparison of Vertical Wave Force to Vertical Resistance:**

						Bridge Condition:
Fv =	495.69	kips	<	Rv =	1127.13 kips	O.K.

♦ **Horizontal Force Calculations:**

Railing Height =	1.167	ft	Water Depth =	1.00	ft
Deck Height =	0.458	ft	Water surface to bot. of girder =	9.59	ft
Girder Height =	2.333	ft			
Bridge Total Height =	3.958	ft	Elevation to bot. of girder =	10.59	ft
Bridge Length =	70.000	ft	Elevation to bot. of deck =	12.92	ft

Ah =	277.08	sq ft
N =	1	girders

● **Largest Unbroken Wave:** *(0.455*storm surge depth)*

Wave Height = 4.07 ft *(above storm surge elevation)*

● **Equation Values:**

η_max =	4.07	ft	*(Wave height above storm surge elevation)*
H_bridge =	3.96	ft	
h =	8.93	ft	
h_girder =	10.59	ft	
h_deck =	12.92	ft	

F_hydrostatic_front =	21368.6667	lbs	η_max	<	h_deck
F_hydrostatic_back =	0.000	lbs	SWL	<	h_girder

● **Resulting Vertical Force:**

Fh = 21.37 kips 10.68 tons

● **Horizontal Resistance:** *(from hand calculations)*

- Sources of Horizontal Resistance:

Self Weight =	1161770	lbs
Girder Seat Interface =	Concrete to Concrete	
μs =	0.8	
Frictional Resistance =	929416	lbs
Shear Friction =	8870400	lbs

- Total Horizontal Resistance:

Rh = 9799.82 kips

● **Comparison of Horizontal Wave Force to Horizontal Resistance:**

							Bridge Condition:
Fh =	21.37	kips	<	Rh =	9799.82	kips	O.K.

Old Makaha No.3A:
Method For Estimating Wave Forces on Bridge Superstructures

McPherson (2008)

Vertical Force Equation:

- $F_{Total} = F_{Hydrostatic} + F_{Bridge} + F_{AirEntrapment}$

- $F_{Hydrostatic} = \gamma \delta_z A - F_w$

If $h \leq h\_model$

$F_w = \frac{1}{2}\gamma\delta A$

If $h > h\_model$

$F_w = \frac{1}{2}\gamma\delta A + \gamma(h - h_{model})A$

- $F_{Bridge} = \gamma Vol_{Bridge}$

- $F_{AirEntrapment} = (n-1)0.5\gamma\delta_G A_G$

Horizontal Force Equation:

- $F_{Total} = F_{Hydrostatic\_Front} + F_{Hydrostatic\_Back}$

If $\eta_{max} < h_{deck}$

$F_{Hydrostatic\_Front} = 0.5 * (\eta_{max} + h - h_{girders}) H_{bridge} L_{bridge} \gamma$

If $\eta_{max} > h_{deck}$

$F_{Hydrostatic\_Front} = 0.5 * [(\eta_{max} + h - h_{girders}) + (\eta_{max} - h_{deck})] H_{bridge} L_{bridge} \gamma$

If $SWL < h_{girders}$

$F_{Hydrostatic\_back} = 0$

If $SWL > h_{girders}$

$F_{Hydrostatic\_back} = 0.5(h - h_{girder})^2 L_{bridge} \gamma$

Constant Coefficients:

γ 64 lb/cubic ft

♦ **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	
Area (A) =	2588.36 sq ft		Height of girder =	1.50 ft	
Vol_bridge =	6933.77 cubic ft		Deck Height =	0.500 ft	
Vol_trapped air/2 =	1167.17 cubic ft		Railing Height =	0.000 ft	
n =	12 girders		Elevation to bot. of girder =	11.24 ft	
			Elevation to bot. of deck =	12.74 ft	
			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*storm surge depth)*

Wave Height = 4.07 ft *(above storm surge elevation)*

- **Equation Values:**

δz = -0.24 ft *(Difference between top of deck and highest point on wave)*

Fw = 0.00 lbs
Fhydrostatic = 0.00 lbs

Fbridge = 443761.28 lbs

Fair_entrapment = 74698.88 lbs

- **Resulting Vertical Force:**

Fv = 518.46 kips 259.23 tons

- **Vertical Resistance:** *(from hand calculations)*

  - Sources of Vertical Resistance:

Self Weight =	279830 lbs	279.83 kips
Drift Bolt Withdrawal Capacity =	69300 lbs	69.3 kips

  - Total Vertical Resistance:

Rv = 349.13 kips

- **Comparison of Vertical Wave Force to Vertical Resistance:**

Fv = 518.46 kips > Rv = 349.13 kips

Bridge Condition:
Failure

♦ **Horizontal Force Calculations:**

Railing Height =	0.000 ft	Water Depth =	4.00 ft
Deck Height =	0.500 ft	Water surface to bot. of girder =	7.24 ft
Girder Height =	1.500 ft		
Bridge Total Height =	2.000 ft	Elevation to bot. of girder =	11.24 ft

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:**

η_max =	4.07	ft	*(Wave height above storm surge elevation)*
H_bridge =	2.000	ft	
h =	8.935	ft	
h_girder =	11.240	ft	
h_deck =	12.740	ft	

F_hydrostatic_front =	8879.78667	lbs	η_max	<	h_deck
F_hydrostatic_back =	0.000	lbs	SWL	<	h_girder

- **Resulting Vertical Force:**

Fh =	8.88	kips	4.44	tons

- **Horizontal Resistance:** *(from hand calculations)*

  - Sources of Horizontal Resistance:

Self Weight =	279830	lbs
Girder Seat Interface =	Wood to Wood (wet)	
μs =	0.2	
Frictional Resistance =	55966	lbs
Drift Bolts Lateral Capacity =	14405.4	lbs

  - Total Horizontal Resistance:

Rh =	70.37	kips

- **Comparison of Horizontal Wave Force to Horizontal Resistance:**

							Bridge Condition:
Fh =	8.88	kips	<	Rh =	70.37	kips	O.K.

Maipalaoa Bridge: (Maili Channel)

Method For Estimating Wave Forces on Bridge Superstructures

McPherson (2008)

Vertical Force Equation:

○ $F_{Total} = F_{Hydrostatic} + F_{Bridge} + F_{AirEntrapment}$

○ $F_{Hydrostatic} = \gamma \delta_z A - F_w$

If $h \leq h\_model$

$F_w = \frac{1}{2}\gamma \delta A$

If $h > h\_model$

$F_w = \frac{1}{2}\gamma \delta A + \gamma(h - h_{model})A$

○ $F_{Bridge} = \gamma Vol_{Bridge}$

○ $F_{AirEntrapment} = (n-1)0.5\gamma \delta_G A_G$

Horizontal Force Equation:

○ $F_{Total} = F_{Hydrostatic\_Front} + F_{Hydrostatic\_Back}$

If $\eta_{max} < h_{deck}$

$F_{Hydrostatic\_Front} = 0.5 * (\eta_{max} + h - h_{girders}) H_{bridge} L_{bridge} \gamma$

If $\eta_{max} > h_{deck}$

$F_{Hydrostatic\_Front} = 0.5 * [(\eta_{max} + h - h_{girders}) + (\eta_{max} - h_{deck})] H_{bridge} L_{bridge} \gamma$

If $SWL < h_{girders}$

$F_{Hydrostatic\_back} = 0$

If $SWL > h_{girders}$

$F_{Hydrostatic\_back} = 0.5(h - h_{girder})^2 L_{bridge} \gamma$

Constant Coefficients:

γ 64 lb/cubic ft

♦ **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
Area (A) =	6476.22222 sq ft		Height of girder =	3.00	ft
Vol_bridge =	7745.12 cubic ft		Deck Height =	0.500	ft
Vol_trapped air/2 =	6242.7 cubic ft		Railing Height =	2.000	ft
n =	16 girders		Elevation to bot. of girder =	6.50	ft
			Elevation to bot. of deck =	9.50	ft
			Storm Surge Depth + Wave Height =	12.00	ft *(from NFIP Flood Hazard Assessment Tool)*
			Storm Surge Depth =	8.25	ft

● **Largest Unbroken Wave:** *(0.455*storm surge depth)*

Wave Height = 3.75 ft *(above storm surge elevation)*

● **Equation Values:**

δz = 2.00 ft *(Difference between top of deck and highest point on wave)*

Fw = 414478.222 lbs

Fhydrostatic = 414478.222 lbs

Fbridge = 495687.68 lbs

Fair_entrapment = 399532.8 lbs

● **Resulting Vertical Force:**

Fv = 1309.70 kips 654.85 tons

● **Vertical Resistance:** *(from hand calculations)*

• Sources of Vertical Resistance:

Self Weight = 1406690 lbs 1406.69 kips

• Total Vertical Resistance:

Rv = 1406.69 kips

● **Comparison of Vertical Wave Force to Vertical Resistance:**

Fv = 1309.70 kips < Rv = 1406.69 kips

Bridge Condition:
O.K.

♦ **Horizontal Force Calculations:**

Railing Height =	2.000	ft	Water Depth =	3.00	ft
Deck Height =	0.500	ft	Water surface to bot. of girder =	3.50	ft
Girder Height =	3.000	ft			
Bridge Total Height =	5.500	ft	Elevation to bot. of girder =	6.50	ft
Bridge Length =	100.667	ft	Elevation to bot. of deck =	9.50	ft

Ah =	553.67	sq ft
N =	16	girders

● **Largest Unbroken Wave:** *(0.455*storm surge depth)*

Wave Height = 3.75 ft *(above storm surge elevation)*

● **Equation Values:**

η_max =	3.75	ft	*(Wave height above storm surge elevation)*
H_bridge =	5.50	ft	
h =	8.25	ft	
h_girder =	6.50	ft	
h_deck =	9.50	ft	

F_hydrostatic_front =	97445.3333	lbs	η_max	<	h_deck
F_hydrostatic_back =	9836.296	lbs	SWL	>	h_girder

● **Resulting Vertical Force:**

Fh = 87.61 kips 43.80 tons

● **Horizontal Resistance:** *(from hand calculations)*

• Sources of Horizontal Resistance:

Self Weight =	1406690	lbs
Girder Seat Interface =	neoprene pad	
μs =	0.1	
Frictional Resistance =	140669	lbs
Beam Web Flexural Capacity =	92016	lbs

• Total Horizontal Resistance:

Rh = 232.69 kips

● **Comparison of Horizontal Wave Force to Horizontal Resistance:**

							Bridge Condition:
Fh =	87.61	kips	<	Rh =	232.69	kips	O.K.

Moanalua Stream Bridge: (Spans 3, 4, 5 & 6)

Method For Estimating Wave Forces on Bridge Superstructures

*Note: The calculations that follow are for one span only, therefore bridge volumes are divided by 8.

McPherson (2008)

Vertical Force Equation:

- $F_{Total} = F_{Hydrostatic} + F_{Bridge} + F_{AirEntrapment}$

- $F_{Hydrostatic} = \gamma\delta_z A - F_w$

If $h \leq h\_model$

$F_w = \frac{1}{2}\gamma\delta A$

If $h > h\_model$

$F_w = \frac{1}{2}\gamma\delta A + \gamma(h - h_{model})A$

- $F_{Bridge} = \gamma Vol_{Bridge}$

- $F_{AirEntrapment} = (n-1)0.5\gamma\delta_G A_G$

Horizontal Force Equation:

- $F_{Total} = F_{Hydrostatic\_Front} + F_{Hydrostatic\_Back}$

If $\eta_{max} < h_{deck}$

$F_{Hydrostatic\_Front} = 0.5 * (\eta_{max} + h - h_{girders})H_{bridge}L_{bridge}\gamma$

If $\eta_{max} > h_{deck}$

$F_{Hydrostatic\_Front} = 0.5 * [(\eta_{max} + h - h_{girders}) + (\eta_{max} - h_{deck})]H_{bridge}L_{bridge}\gamma$

If $SWL < h_{girders}$

$F_{Hydrostatic\_back} = 0$

If $SWL > h_{girders}$

$F_{Hydrostatic\_back} = 0.5(h - h_{girder})^2 L_{bridge}\gamma$

Constant Coefficients:

γ 64 lb/cubic ft

♦ **Vertical Force Calculations:**

Bridge Deck Width =	64.33 ft		Water Depth =	2.50	ft
Bridge Deck Length =	27.00 ft		Water surface to bot. of girder =	4.00	ft
Area (A) =	1737 sq ft		Height of girder =	1.83	ft
Vol_bridge =	2737.62 cubic ft		Deck Height =	0.667	ft
Vol_trapped air/2 =	1390.11 cubic ft		Railing Height =	3.729	ft
n =	9 girders		Elevation to bot. of girder =	6.50	ft
			Elevation to bot. of deck =	8.33	ft
			Storm Surge Depth + Wave Height =	10.00	ft *(from NFIP Flood Hazard Assessment Tool)*
			Storm Surge Depth =	6.87	ft

- **Largest Unbroken Wave:** *(0.455*storm surge depth)*

Wave Height = 3.13 ft *(above storm surge elevation)*

- **Equation Values:**

δz = 1.00 ft *(Difference between top of deck and highest point on wave)*

Fw = 55584 lbs

Fhydrostatic = 55584 lbs

Fbridge = 175207.537 lbs

Fair_entrapment = 88967 lbs

- **Resulting Vertical Force:**

Fv = 319.76 kips 159.88 tons

- **Vertical Resistance:** *(from hand calculations)*

  - Sources of Vertical Resistance:

Self Weight =	417270	lbs	417.27	kips
Pier Cap Weight =	158400	lbs	158.4	kips

  - Total Vertical Resistance:

Rv = 575.67 kips

- **Comparison of Vertical Wave Force to Vertical Resistance:**

Fv = 319.76 kips < Rv = 575.67 kips

Bridge Condition:
O.K.

♦ **Horizontal Force Calculations:**

Railing Height =	3.729	ft	Water Depth =	2.50	ft
Deck Height =	0.667	ft	Water surface to bot. of girder =	4.00	ft
Girder Height =	1.833	ft			
Bridge Total Height =	6.229	ft	Elevation to bot. of girder =	6.50	ft

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:**

η_max =	3.13	ft	*(Wave height above storm surge elevation)*
H_bridge =	6.23	ft	
h =	6.87	ft	
h_girder =	6.50	ft	
h_deck =	8.33	ft	

F_hydrostatic_front =	18837	lbs	η_max	<	h_deck
F_hydrostatic_back =	120.112	lbs	SWL	>	h_girder

• **Resulting Vertical Force:**

Fh = 18.72 kips 9.36 tons

• **Horizontal Resistance:** *(from hand calculations)*

• Sources of Horizontal Resistance:

Self Weight =	417270	lbs
Girder Seat Interface =	concrete to concrete	
μs =	0.8	
Frictional Resistance =	333816	lbs
Dowel Shear Friction =	1592640	lbs

• Total Horizontal Resistance:

Rh = 1926.46 kips

• **Comparison of Horizontal Wave Force to Horizontal Resistance:**

							Bridge Condition:
Fh =	18.72	kips	<	Rh =	1926.46	kips	O.K.

*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.

Kalihi 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:

- $F_{Total} = F_{Hydrostatic} + F_{Bridge} + F_{AirEntrapment}$
- $F_{Hydrostatic} = \gamma \delta_z A - F_w$

  If $h \leq h\_model$

  $F_w = \frac{1}{2}\gamma\delta A$

  If $h > h\_model$

  $F_w = \frac{1}{2}\gamma\delta A + \gamma(h - h_{model})A$
- $F_{Bridge} = \gamma Vol_{Bridge}$
- $F_{AirEntrapment} = (n-1)0.5\gamma\delta_G A_G$

Horizontal Force Equation:

- $F_{Total} = F_{Hydrostatic\_Front} + F_{Hydrostatic\_Back}$

If $\eta_{max} < h_{deck}$

$F_{Hydrostatic\_Front} = 0.5 * (\eta_{max} + h - h_{girders}) H_{bridge} L_{bridge} \gamma$

If $\eta_{max} > h_{deck}$

$F_{Hydrostatic\_Front} = 0.5 * [(\eta_{max} + h - h_{girders}) + (\eta_{max} - h_{deck})] H_{bridge} L_{bridge} \gamma$

If $SWL < h_{girders}$

$F_{Hydrostatic\_back} = 0$

If $SWL > h_{girders}$

$F_{Hydrostatic\_back} = 0.5(h - h_{girder})^2 L_{bridge} \gamma$

Constant Coefficients:

γ 64 lb/cubic ft

♦ **Vertical Force Calculations:**

Bridge Deck Width =	88.33	ft	Water Depth =	2.50	ft
Bridge Deck Length =	27.00	ft	Water surface to bot. of girder =	4.00	ft
Area (A) =	2385	sq ft	Height of girder =	1.83	ft
Vol_bridge =	3684.43	cubic ft	Deck Height =	0.667	ft
Vol_trapped air/2 =	979.86	cubic ft	Railing Height =	3.729	ft
n =	13	girders	Elevation to bot. of girder =	6.50	ft
			Elevation to bot. of deck =	8.33	ft

Storm Surge Depth + Wave Height = 10.00 ft *(from NFIP Flood Hazard Assessment Tool)*

Storm Surge Depth = 6.87 ft

- **Largest Unbroken Wave:** *(0.455*storm surge depth)*

  Wave Height = 3.13 ft *(above storm surge elevation)*

- **Equation Values:**

  δz = 1.00 ft *(Difference between top of deck and highest point on wave)*

  Fw = 76320 lbs

  Fhydrostatic = 76320 lbs

  Fbridge = 235803.236 lbs

  Fair_entrapment = 62710.784 lbs

- **Resulting Vertical Force:**

  Fv = 374.83 kips 187.42 tons

- **Vertical Resistance:** *(from hand calculations)*

  - Sources of Vertical Resistance:

    Self Weight = 565087.1 lbs 565.09 kips

  - Total Vertical Resistance:

    Rv = 565.09 kips

- **Comparison of Vertical Wave Force to Vertical Resistance:**

							Bridge Condition:
Fv =	374.83	kips	<	Rv =	565.09	kips	O.K.

♦ **Horizontal Force Calculations:**

Railing Height =	3.729	ft	Water Depth =	2.50	ft
Deck Height =	0.667	ft	Water surface to bot. of girder =	4.00	ft
Girder Height =	1.833	ft			
Bridge Total Height =	6.229	ft	Elevation to bot. of girder =	6.50	ft
Bridge Length =	27.000	ft	Elevation to bot. of deck =	8.33	ft

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:**

η_max =	4.23	ft	*(Wave height above storm surge elevation)*
H_bridge =	6.23	ft	
h =	6.87	ft	
h_girder =	6.50	ft	
h_deck =	8.33	ft	

F_hydrostatic_front =	24745.6407	lbs	η_max	<	h_deck
F_hydrostatic_back =	120.112	lbs	SWL	>	h_girder

- **Resulting Vertical Force:**

Fh = 24.63 kips 12.31 tons

- **Horizontal Resistance:** *(from hand calculations)*

  - Sources of Horizontal Resistance:
    - Self Weight = 565087.1 lbs
    - Girder Seat Interface = concrete to concrete
    - μs = 0.8
    - Frictional Resistance = 452069.7 lbs

  - Total Horizontal Resistance:
    - Rh = 452.07 kips

- **Comparison of Horizontal Wave Force to Horizontal Resistance:**

								Bridge Condition:
Fh =	24.63	kips	<	Rh =	452.07	kips		O.K.

Nimitz Highway Slip Cover #2:
Method For Estimating Wave Forces on Bridge Superstructures

McPherson (2008)

Vertical Force Equation:

- $F_{Total} = F_{Hydrostatic} + F_{Bridge} + F_{AirEntrapment}$

- $F_{Hydrostatic} = \gamma\delta_z A - F_w$

If $h \le h\_model$

$F_w = \frac{1}{2}\gamma\delta A$

If $h > h\_model$

$F_w = \frac{1}{2}\gamma\delta A + \gamma(h - h_{model})A$

- $F_{Bridge} = \gamma Vol_{Bridge}$

- $F_{AirEntrapment} = (n-1)0.5\gamma\delta_G A_G$

Horizontal Force Equation:

- $F_{Total} = F_{Hydrostatic\_Front} + F_{Hydrostatic\_Back}$

If $\eta_{max} < h_{deck}$

$F_{Hydrostatic\_Front} = 0.5 * (\eta_{max} + h - h_{girders})H_{bridge}L_{bridge}\gamma$

If $\eta_{max} > h_{deck}$

$F_{Hydrostatic\_Front} = 0.5 * [(\eta_{max} + h - h_{girders}) + (\eta_{max} - h_{deck})]H_{bridge}L_{bridge}\gamma$

If $SWL < h_{girders}$

$F_{Hydrostatic\_back} = 0$

If $SWL > h_{girders}$

$F_{Hydrostatic\_back} = 0.5(h - h_{girder})^2 L_{bridge}\gamma$

Constant Coefficients:

γ 64 lb/cubic ft

♦ **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			
Area (A) =	10704.6	sq ft	Height of girder =	2.50	ft
Vol_bridge =	13797	cubic ft	Deck Height =	1.292	ft
Vol_trapped air/2 =	0	cubic ft	Railing Height =	1.667	ft
n =	11	girders	Elevation to bot. of girder =	9.83	ft
			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*storm surge depth)*

Wave Height = 2.50 ft *(above storm surge elevation)*

- **Equation Values:**

δz = -5.63 ft *(Difference between top of deck and highest point on wave)*

Fw = 0 lbs
Fhydrostatic = 0 lbs

Fbridge = 0 lbs *(bridge is not submerged, water level is too low)*

Fair_entrapment = 0 lbs

- **Resulting Vertical Force:**

Fv = 0.00 kips 0.00 tons

- **Vertical Resistance:** *(from hand calculations)*

  - Sources of Vertical Resistance:
    Self Weight = 2069453 lbs 2069.453 kips

  - Total Vertical Resistance:
    Rv = 2069.453 kips

- **Comparison of Vertical Wave Force to Vertical Resistance:**

				Bridge Condition:
Fv = 0.00 kips	<	Rv = 2069.453 kips		O.K.

♦ **Horizontal Force Calculations:**

Railing Height =	1.667	ft	Water Depth =	5.33	ft
Deck Height =	1.292	ft	Water surface to bot. of girder =	4.50	ft
Girder Height =	2.500	ft			

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*storm surge depth)*

Wave Height = 2.50 ft *(above storm surge elevation)*

- **Equation Values:**

η_max =	2.50	ft	*(Wave height above storm surge elevation)*
H_bridge =	5.46	ft	
h =	8.00	ft	
h_girder =	9.83	ft	
h_deck =	12.33	ft	

F_hydrostatic_front =	0	lbs	η_max	<	h_deck	*(bridge is not submerged)*
F_hydrostatic_back =	0.000	lbs	SWL	<	h_girder	

- **Resulting Vertical Force:**

Fh = 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

    μs = 0.8

    Frictional Resistance = 1655562 lbs

  - Total Horizontal Resistance:

    Rh = 1655.56 kips

- **Comparison of Horizontal Wave Force to Horizontal Resistance:**

							Bridge Condition:
Fh =	0.00	kips	<	Rh =	1655.56	kips	O.K.

Nimitz Highway Slip Cover #3:

Method For Estimating Wave Forces on Bridge Superstructures

McPherson (2008)

Vertical Force Equation:

- $F_{Total} = F_{Hydrostatic} + F_{Bridge} + F_{AirEntrapment}$
- $F_{Hydrostatic} = \gamma\delta_z A - F_w$

If $h \le h\_model$

$F_w = \frac{1}{2}\gamma\delta A$

If $h > h\_model$

$F_w = \frac{1}{2}\gamma\delta A + \gamma(h - h_{model})A$

- $F_{Bridge} = \gamma Vol_{Bridge}$
- $F_{AirEntrapment} = (n-1)0.5\gamma\delta_G A_G$

Horizontal Force Equation:

- $F_{Total} = F_{Hydrostatic\_Front} + F_{Hydrostatic\_Back}$

If $\eta_{max} < h_{deck}$

$F_{Hydrostatic\_Front} = 0.5 * (\eta_{max} + h - h_{girders})H_{bridge}L_{bridge}\gamma$

If $\eta_{max} > h_{deck}$

$F_{Hydrostatic\_Front} = 0.5 * [(\eta_{max} + h - h_{girders}) + (\eta_{max} - h_{deck})]H_{bridge}L_{bridge}\gamma$

If $SWL < h_{girders}$

$F_{Hydrostatic\_back} = 0$

If $SWL > h_{girders}$

$F_{Hydrostatic\_back} = 0.5(h - h_{girder})^2 L_{bridge}\gamma$

Constant Coefficients:

γ 64 lb/cubic ft

♦ **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			
Bridge Deck Length (1) =	162.00	ft	Height of girder =	2.50	ft
Bridge Deck Length (2) =	78.00	ft	Deck Height =	1.292	ft
Av =	8536.5	sq ft	Railing Height =	1.667	ft
Vol_bridge =	14499	cubic ft	Elevation to bot. of girder =	9.83	ft
Vol_trapped air/2 =	0	cubic ft	Elevation to bot. of deck =	12.33	ft
n =	13	girders	Storm Surge Depth + Wave Height =	8.00	ft
			Storm Surge Depth =	5.50	ft

(from NFIP Flood Hazard Assessment Tool)

- **Largest Unbroken Wave:** *(0.455*storm surge depth)*

Wave Height = 2.50 ft *(above storm surge elevation)*

- **Equation Values:**

δz = -5.63 ft *(Difference between top of deck and highest point on wave)*

Fw = 0 lbs

Fhydrostatic = 0 lbs

Fbridge = 0 lbs *(bridge is not submerged, water level is too low)*

Fair_entrapment = 0 lbs

- **Resulting Vertical Force:**

Fv = 0.00 kips 0.00 tons

- **Vertical Resistance:** *(from hand calculations)*

  - Sources of Vertical Resistance:

    Self Weight = 2069453 lbs 2069.453 kips

  - Total Vertical Resistance:

    Rv = 2069.453 kips

- **Comparison of Vertical Wave Force to Vertical Resistance:**

Fv = 0.00 kips < Rv = 2069.453 kips

Bridge Condition:
O.K.

♦ **Horizontal Force Calculations:**

Railing Height =	1.667	ft	Water Depth =	5.33	ft
Deck Height =	1.292	ft	Water surface to bot. of girder =	4.50	ft
Girder Height =	2.500	ft			
Bridge Total Height =	5.458	ft	Elevation to bot. of girder =	9.83	ft

Bridge Length =	240.000	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:**

η_max =	6.39	ft	*(Wave height above storm surge elevation)*
H_bridge =	5.46	ft	
h =	8.00	ft	
h_girder =	9.83	ft	
h_deck =	12.33	ft	

F_hydrostatic_front =	0	lbs	η_max	<	h_deck	*(bridge is not submerged)*
F_hydrostatic_back =	0.000	lbs	SWL	<	h_girder	

• **Resulting Vertical Force:**

Fh = 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

μs = 0.8

Frictional Resistance = 1744018 lbs

• Total Horizontal Resistance:

Rh = 1744.02 kips

• **Comparison of Horizontal Wave Force to Horizontal Resistance:**

							Bridge Condition:
Fh =	0.00	kips	<	Rh =	1744.02	kips	O.K.

Appendix D: AASHTO Guide Specifications Calculations

Average Water Depth Over Fetch Length:

	Water Depths are from Google Earth									
	Water Depth (10000 ft away)	Water Depth (8000 ft away)	Water Depth (6000 ft away)	Water Depth (4000 ft	Water Depth (2000 ft away)	Water Depth (1000 ft away)	Water Depth (500 ft	Water Depth (200 ft away)	Average Water Depth (ft)	Total Average Water Depth
Kuliouou Bridge:	105	43	27	8	5.50	5.50	5.50	5.50	25.63	25.63
Kahaluu Stream Bridge:	32	30	14	11.00	11.00	11.00	11.00	11.00	16.38	16.38
New South Punaluu Bridge:	238	135	53	17	22	8.25	8.25	8.25	61.22	61.22
Moanalua Bridge:	10	8	7	6.87	6.87	6.87	6.87	6.87	7.42	7.42
Kalihi Bridge:	10	8	7	6.87	6.87	6.87	6.87	6.87	7.42	7.42

	Water Depths are from Google Earth									
	Water Depth (3250 ft away)	Water Depth (2750 ft away)	Water Depth (2250 ft away)	Water Depth (1750 ft away)	Water Depth (1250 ft away)	Water Depth (1000 ft away)	Water Depth (500 ft away)	Water Depth (200 ft away)	Average Water Depth (ft)	Total Average Water Depth (ft)
New Makaha Stream #3A Bridge:	72	53	35	20	10	8.93	8.93	8.93	27.10	27.10
Old Makaha Stream #3A Bridge:	72	53	35	20	10	8.93	8.93	8.93	27.10	27.10

(*Note: For the these locations the change in depth is rapid, therefore depths were measured closer to the shore where the continental shelf begins)

	Water Depths are from Google Earth									
	Water Depth (4500 ft away)	Water Depth (4000 ft away)	Water Depth (3500 ft away)	Water Depth (3000 ft away)	Water Depth (2000 ft away)	Water Depth (1000 ft away)	Water Depth (500 ft away)	Water Depth (200 ft away)	Average Water Depth (ft)	Total Average Water Depth (ft)
Maipalaoa Bridge:	40	35	29	23	13	8.25	8.25	8.25	20.59	20.59

(*Note: For the these locations the change in depth is rapid, therefore depths were measured closer to the shore where the continental shelf begins)

	Water Depths are from Google Earth									
	Water Depth (6700 ft away)	Water Depth (5500 ft away)	Water Depth (4500 ft away)	Water Depth (3500 ft away)	Water Depth (3000 ft away)	Water Depth (2000 ft away)	Water Depth (1000 ft away)	Water Depth (200 ft away)	Average Water Depth (ft)	Total Average Water Depth (ft)
Nimitz Hwy. Slip Cover #2:	5.50	5.50	5.50	5.50	13	9	5.50	5.50	6.88	6.88
Nimitz Hwy. Slip Cover #3:	5.50	5.50	5.50	5.50	13	9	5.50	5.50	6.88	6.88

Kuliouou Bridge:
Method For Estimating Wave Forces on Bridge Superstructures

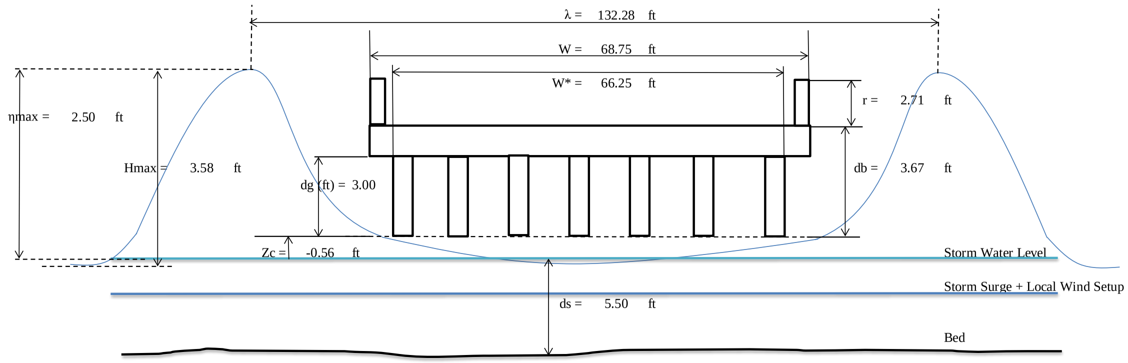
AASHTO (2008)
Constant Coefficients:

50 year wind speed =	105	mph	*(from AASHTO Fig 6.2.2.2-1 b)*
specific weight water =	0.064	kip/cubic ft	
g =	32.2	ft/sec^2	

♦ **Wave Calculations:**

● **Bridge Properties:**

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
Girder to Girder Width =	66.25	ft	Height of girder =	3.00	ft
Deck Thickness =	0.67	ft	Height of railing =	2.71	ft
Av =	3327.21	sq ft	Elevation to bot. of girder =	4.94	ft
N =	12	girders	Elevation to bot. of deck =	7.94	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)*
d =	25.63	ft	*(average water depth over fetch length)*
Fetch Length =	4767610.00	ft	
[(g*d)/(Ut*)^2] =	0.010		
Tp =	9.86	sec	

○ **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: *(until 't' converges)*

First Iteration:
Ut = 169.45 ft/sec

Second Iteration:
Ut* = 297.38 ft/sec
[(g*d)/(Ut*)^2] = 0.009
Tp = 9.95 sec
t = 5896.69 sec

Third Iteration:
Ut = 169.49 sec
Ut* = 297.47 ft/sec
[(g*d)/(Ut*)^2] = 0.009
Tp = 9.95 sec
t = 5895.42 sec

Fourth Iteration:
Ut = 169.49 sec
Ut* = 297.47 ft/sec
[(g*d)/(Ut*)^2] = 0.009
Tp = 9.95 sec
t = 5895.41 sec

Fifth Iteration:
Ut = 169.49 sec
Ut* = 297.47 ft/sec
[(g*d)/(Ut*)^2] = 0.009
Tp = 9.95 sec
t = 5895.41 sec

Sixth Iteration:
Ut = 169.49 sec
Ut* = 297.47 ft/sec
[(g*d)/(Ut*)^2] = 0.009
Tp = 9.95 sec
t = 5895.41 sec *(the value of 't' has converged)*

○ **Wave height and wavelength:**

Hs = 12.37 ft
Wave Length (λ) = 132.28 ft
Hmax = 22.26 ft

Maximum Wave Height: *(least of the following)*

Hmax = 22.26 ft
Hmax ≤ 0.65*ds = 3.58 ft
Hmax ≤ λ/7.0 = 18.90 ft

Therefore Hmax = 3.58 ft

○ **Resulting Storm Wave Properties:** *(see Fig)*

Tp = 9.95 sec

Hmax =	3.58	ft
Wave Length (λ) =	132.28	ft
η max =	2.50	ft

■ **<u>Maximum Quasi-Static Vertical Force and Associated Forces and Moments:</u>** *(AASHTO Sec 6.1.2.2)*

● **<u>Maximum Quasi-Static Vertical Force:</u>** *(AASHTO Sec 6.1.2.2.1)*

○ **<u>Determination of Fv-max parameters:</u>** *(eq 6.1.2.2.1-1)*

W_hat =	86.86	ft				
W_hat / W =	1.26		>	0.15	Therefore W_hat = W_hat	
W_hat =	86.86	ft				
η max - Zc =	3.06		<	db =	3.67	ft
β =	0.84					
x =	0.0270269					
y =	0.6566434					

<u>For Girder Spans:</u> *(eq 6.1.2.2.1 a)*

b0 =	-0.888	b4 =	-0.00048
b1 =	56.16	b5 =	-0.056
b2 =	0.0538	b6 =	7.86
b3 =	-192.8944		

<u>Tapped Air Factor:</u>

A_air =	0.0057553			
B_air =	0.3945618			
(η max - Zc) / dg =	1.0208333	>	1	
%Air =	100.00			

Assume 50% air pocket:

%Air =	50			
TAF =	0.682325	<	1	(O.K.)
TAF =	0.682325			

<u>Quasi-Static Vertical Force:</u> *(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

● **<u>Associated Vertical Slamming Force:</u>** *(AASHTO Sec 6.1.2.2.2)*

B =	-1.2801331		
Zc / η max =	-0.2237762	<	0
A =	0.0282657		

<u>Vertical Slamming Force:</u> *(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

● **<u>Associated Horizontal Quasi-Static Wave Force:</u>** *(AASHTO Sec 6.1.2.2.3)*

*Note: Girders used on the Kuliouou Bridge are similar to the AASHTO Type III)

<u>From Table 6.1.2.2.3-1:</u> *(for AASHTO Type III Girder)*

a0 =	-0.0938	a5 =	0.0054
a1 =	1.6197	a6 =	0.019
a2 =	-1.4792	a7 =	0.6044
a3 =	0.5367	a8 =	-0.283
a4 =	-0.0877		

x = 0.4803922

y = 0.0270269

<u>Horizontal Quasi-Static Wave Force:</u> *(eq 6.1.2.2.3)*

Fh-av = 0.1231737 kip/ft

Length of Bridge = 48.40 ft
Fh-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 6.1.2.2.4-1)*

Mt-av = 108.58 (kip/ft)-ft

Length of Bridge = 48.40 ft
Mt-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:** *(AASHTO Sec 6.1.2.3.1)*

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 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)*

Fs = 2.3523513 kip/ft

Length of Bridge = 48.40 ft

Fs Total = 113.84 kips 56.92 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 = 277.01574 (kip/ft)-ft

Associated Moment about Trailing Edge: *(eq 6.1.2.3.4-1)*
Mt-ah = 316.06586 (kip/ft)-ft

Length of Bridge = 48.40 ft
Mt-ah Total = 15296.27 kip-ft 7648.14 ton-ft

● **Resulting Maximum Horizontal Wave Force and Associated Forces and Moments:** *(AASHTO sec 6.1.2.3)*

Fh-max Total =	27.61	kips	*(Maximum Horizontal Wave Force)*
Fv-ah Total =	174.82	kips	*(Quasi-Static Vertical Force)*
Fs Total =	113.84	kips	*(Vertical Slamming Force)*
Mt-ah Total =	15296.27	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

Kahaluu Bridge:
Method For Estimating Wave Forces on Bridge Superstructures

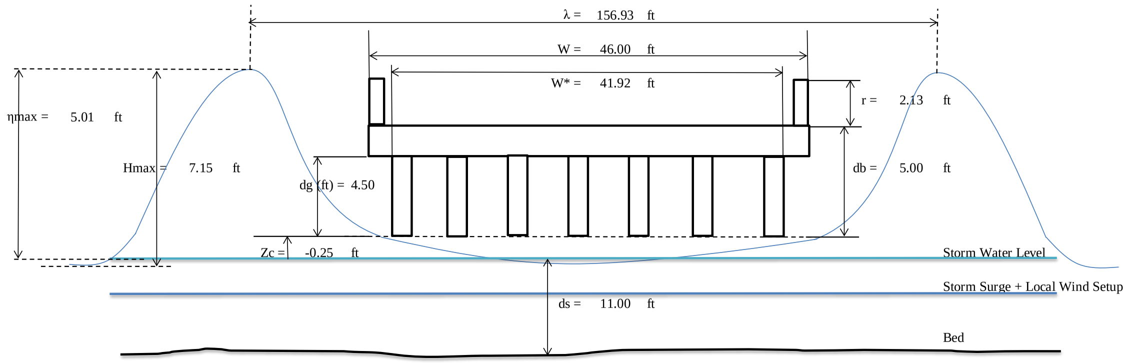
AASHTO (2008)
Constant Coefficients:

50 year wind speed =	105	mph	*(from AASHTO Fig 6.2.2.2-1 b)*
specific weight water =	0.064	kip/cubic ft	
g =	32.2	ft/sec^2	

♦ **Wave Calculations:**

● **Bridge Properties:**

Bridge Deck Width =	46.00	ft	Water Depth =	5.75	ft
Bridge Deck Length =	318.00	ft	Water surface to bot. of girder =	5.00	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
Av =	14628.00	sq ft	Elevation to bot. of girder =	10.75	ft
N =	8	girders	Elevation to bot. of deck =	15.25	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 =	11.00	ft	*(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	

○ **Determination of time duration to develop fetch limited waves:**

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)*

First Iteration:

Ut = 165.91 ft/sec

Second Iteration:

Ut* =	289.75	ft/sec
[(g*d)/(Ut*)^2] =	0.006	
Tp =	8.39	sec
t =	4102.79	sec

Third Iteration:

Ut =	165.92	sec
Ut* =	289.78	ft/sec
[(g*d)/(Ut*)^2] =	0.006	
Tp =	8.39	sec
t =	4102.50	sec

Fourth Iteration:

Ut =	165.92	sec
Ut* =	289.78	ft/sec
[(g*d)/(Ut*)^2] =	0.006	
Tp =	8.39	sec
t =	4102.50	sec

Fifth Iteration:

Ut =	165.92	sec
Ut* =	289.78	ft/sec
[(g*d)/(Ut*)^2] =	0.006	
Tp =	8.39	sec
t =	4102.50	sec

Sixth Iteration:

Ut =	165.92	sec
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 wavelength:**

Hs =	8.72	ft
Wave Length (λ) =	156.93	ft
Hmax =	15.70	ft

Maximum Wave Height: *(least of the following)*

Hmax =	15.70	ft
Hmax ≤ 0.65*ds =	7.15	ft
Hmax ≤ λ/7.0 =	22.42	ft

Therefore Hmax = 7.15 ft

○ **Resulting Storm Wave Properties:** *(see Fig)*

Tp = 8.39 sec

Hmax = 7.15 ft
Wave Length (λ) = 156.93 ft
η max = 5.01 ft

■ **<u>Maximum Quasi-Static Vertical Force and Associated Forces and Moments:</u>** *(AASHTO Sec 6.1.2.2)*

● **<u>Maximum Quasi-Static Vertical Force:</u>** *(AASHTO Sec 6.1.2.2.1)*

○ **<u>Determination of Fv-max parameters:</u>** *(eq 6.1.2.2.1-1)*

W_hat = 83.95 ft
W_hat / W = 1.83 > 0.15 Therefore W_hat = W_hat
W_hat = 83.95 ft

η max - Zc = 5.26 > db = 5.00 ft
β = 1.00

x = 0.0455613

y = 0.534965

<u>For Girder Spans:</u> *(eq 6.1.2.2.1 a)*

b0 =	-1.038	b4 =	-0.00057
b1 =	55.89	b5 =	0.22
b2 =	0.058	b6 =	11.01
b3 =	-192.5416		

<u>Tapped Air Factor:</u>

A_air = 0.0058515
B_air = 0.4069234

(η max - Zc) / dg = 1.1677778 > 1
%Air = 100.00

Assume 50% air pocket:
%Air = 50

TAF = 0.6994975 < 1 (O.K.)
TAF = 0.6994975

<u>Quasi-Static Vertical Force:</u> *(eq 6.1.2.2.1-1)*

Fv-max = 7.4058389 kip/ft

Length of Bridge = 318.00 ft
Fv-max Total = 2355.06 kips 1177.53 tons

● **<u>Associated Vertical Slamming Force:</u>** *(AASHTO Sec 6.1.2.2.2)*

B = -1.218169

Zc / η max = -0.04995 < 0
A = 0.0308557

<u>Vertical Slamming Force:</u> *(eq 6.1.2.2.2-1)*

Fs = 4.3469604 kip/ft

Length of Bridge = 318.00 ft
Fs Total = 1382.33 kips 691.17 tons

● **<u>Associated Horizontal Quasi-Static Wave Force:</u>** *(AASHTO Sec 6.1.2.2.3)*

*Note: Girders used on the Kahaluu Bridge are AASHTO Type IV)

<u>From Table 6.1.2.2.3-1:</u> *(for AASHTO Type IV Girder)*

a0 =	-0.0911	a5 =	0.0048
a1 =	1.5445	a6 =	0.0113
a2 =	-1.4684	a7 =	0.6785
a3 =	0.54	a8 =	-0.2661
a4 =	-0.0861		

x = 0.7375439

y = 0.0455613

<u>Horizontal Quasi-Static Wave Force:</u> *(eq 6.1.2.2.3)*

Fh-av = 0.7988378 kip/ft

Length of Bridge = 318.00 ft
Fh-av Total = 254.03 kips 127.02 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 ft
b_m = -0.045446 ft
c_m = -0.004767 ft
W' = 4.08 ft
W* = 41.92 ft

Associated Moment about Trailing Edge: *(eq 6.1.2.2.4-1)*

Mt-av = 142.22 (kip/ft)-ft

Length of Bridge = 318.00 ft
Mt-av Total = 45224.59 kip-ft 22612.29 tons-ft

● **Resulting Quasi-Static Vertical Force and Associated Forces and Moments:** *(AASHTO sec 6.1.2.2)*

Fv-max Total =	2355.06	kips	*(Quasi-Static Vertical Force)*
Fs Total =	1382.33	kips	*(Vertical Slamming Force)*
Fh-av Total =	254.03	kips	*(Quasi-Static Horizontal Force)*
Mt-av =	45224.59	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)*

check = 120.44 > W = 46.00 ft Use eq 6.1.2.3.1-4 for omega

ω = 46.00 ft

Reference Horizontal Force: *(eq 6.1.2.3.1-2)*

F*h-max = 3.24 kip/ft

Horizontal Wave Force: *(eq 6.1.2.3.1-1)*

Fh-max = 2.0358549 kip/ft

Length of Bridge = 318.00 ft
Fh-max Total = 647.40 kip 323.70 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 = 32.29 < W = 46.00 ft Use eq 6.1.2.3.2-3 for alpha

α = 32.29 ft

Reference Vertical Force: *(eq 6.1.2.3.2-2)*

F*v-ah = 10.86 kip/ft

Quasi-Static Vertical Wave Force: *(eq 6.1.2.3.2-1)*

Fv-ah = 6.5194655 kip/ft

Length of Bridge = 318.00 ft
Fv-ah Total = 2073.19 kip 1036.60 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)*

Fs = 4.3469604 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*t-ah = 347.74253 (kip/ft)-ft

Associated Moment about Trailing Edge: *(eq 6.1.2.3.4-1)*
Mt-ah = 352.75575 (kip/ft)-ft

Length of Bridge =	318.00	ft		
Mt-ah Total =	112176.33	kip-ft	56088.16	ton-ft

● **Resulting Maximum Horizontal Wave Force and Associated Forces and Moments:** *(AASHTO sec 6.1.2.3)*

Fh-max Total =	647.40	kips	*(Maximum Horizontal Wave Force)*
Fv-ah Total =	2073.19	kips	*(Quasi-Static Vertical Force)*
Fs Total =	1382.33	kips	*(Vertical Slamming Force)*
Mt-ah Total =	112176.33	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

New South Punaluu Bridge: *(span #2)*

Method For Estimating Wave Forces on Bridge Superstructures

AASHTO (2008)

Constant Coefficients:

50 year wind speed = 105 mph *(from AASHTO Fig 6.2.2.2-1 b)*

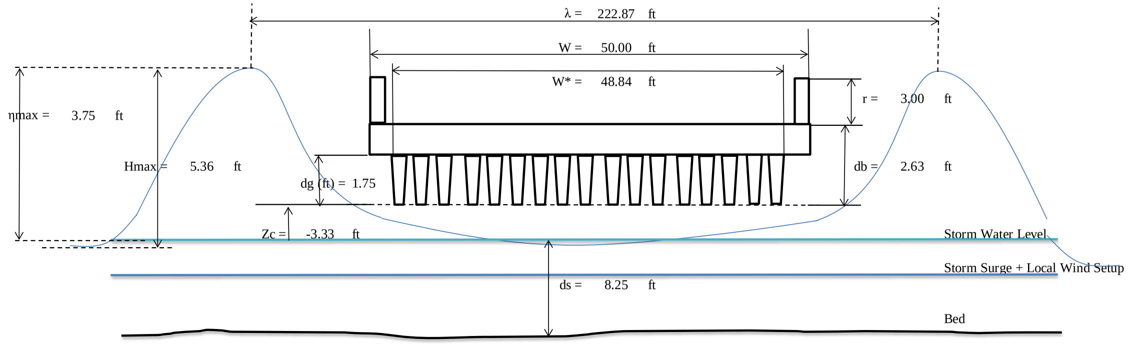
specific weight water = 0.064 kip/cubic ft

g = 32.2 ft/sec^2

♦ **Wave Calculations:**

● **Bridge Properties:**

Bridge Deck Width =	50.00	ft	Water Depth =	4.92	ft
Bridge Deck Length =	66.00	ft	Water surface to bot. of girder =	0.00	ft
Girder to Girder Width =	48.84	ft	Height of girder =	1.75	ft
Deck Thickness =	0.88	ft	Height of railing =	3.00	ft
Av =	3300.00	sq ft	Elevation to bot. of girder =	4.92	ft
N =	30	girders	Elevation to bot. of deck =	6.67	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 = 8.25 ft *(from NFIP Flood Hazard Assessment Tool)*

d = 61.22 ft *(average water depth over fetch length)*

Fetch Length = 4767610.00 ft

[(g*d)/(Ut*)^2] = 0.024

Tp = 13.35 sec

○ **Determination of time duration to develop fetch limited waves:**

t = 12270.88 sec > 3600 sec

U_1hr = 179.10 ft/sec To compute U_1hr use eq 6.2.2.4-5

Iteration Process: *(until 't' converges)*

First Iteration:

Ut = 177.00 ft/sec

Second Iteration:

Ut* = 313.76 ft/sec

[(g*d)/(Ut*)^2] = 0.020

Tp = 13.68 sec

t = 11546.93 sec

Third Iteration:

Ut = 177.06 sec

Ut* = 313.89 ft/sec

[(g*d)/(Ut*)^2] = 0.020

Tp = 13.68 sec

t = 11543.43 sec

Fourth Iteration:

Ut = 177.06 sec

Ut* = 313.89 ft/sec

[(g*d)/(Ut*)^2] = 0.020

Tp = 13.68 sec

t = 11543.42 sec

Fifth Iteration:

Ut = 177.06 sec

Ut* = 313.89 ft/sec

[(g*d)/(Ut*)^2] = 0.020

Tp = 13.68 sec

t = 11543.42 sec

Sixth Iteration:

Ut = 177.06 sec

Ut* = 313.89 ft/sec

[(g*d)/(Ut*)^2] = 0.020

Tp = 13.68 sec

t = 11543.42 sec *(the value of 't' has converged)*

○ **Wave height and wavelength:**

Hs = 24.41 ft

Wave Length (λ) = 222.87 ft

Hmax = 43.93 ft

Maximum Wave Height: *(least of the following)*

Hmax = 43.93 ft

Hmax ≤ 0.65*ds = 5.36 ft

Hmax ≤ λ/7.0 = 31.84 ft

Therefore Hmax = 5.36 ft

○ **Resulting Storm Wave Properties:** *(see Fig)*

Tp = 13.68 sec

Hmax =	5.36	ft
Wave Length (λ) =	222.87	ft
η max =	3.75	ft

■ **<u>Maximum Quasi-Static Vertical Force and Associated Forces and Moments:</u>** *(AASHTO Sec 6.1.2.2)*

● **<u>Maximum Quasi-Static Vertical Force:</u>** *(AASHTO Sec 6.1.2.2.1)*

○ **<u>Determination of Fv-max parameters:</u>** *(eq 6.1.2.2.1-1)*

W_hat = 249.84 ft
W_hat / W = 5.00 > 0.15 Therefore W_hat = W_hat
W_hat = 249.84 ft

η max - Zc = 7.08 > db = 2.63 ft
β = 1.00

x = 0.0240609

y = 1.120979

<u>For Girder Spans:</u> *(eq 6.1.2.2.1 a)*

b0 =	-0.763	b4 =	-0.00041
b1 =	56.385	b5 =	-0.286
b2 =	0.0503	b6 =	5.235
b3 =	-193.1884		

<u>Tapped Air Factor:</u>

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

<u>Quasi-Static Vertical Force:</u> *(eq 6.1.2.2.1-1)*

Fv-max = 13.50676 kip/ft

Length of Bridge = 66.00 ft
Fv-max Total = 891.45 kips 445.72 tons

● **<u>Associated Vertical Slamming Force:</u>** *(AASHTO Sec 6.1.2.2.2)*

B = -1.1507468

Zc / η max = -0.8871129 < 0
A = 0.018382

<u>Vertical Slamming Force:</u> *(eq 6.1.2.2.2-1)*

Fs = 2.4660879 kip/ft

Length of Bridge = 66.00 ft
Fs Total = 162.76 kips 81.38 tons

● **<u>Associated Horizontal Quasi-Static Wave Force:</u>** *(AASHTO Sec 6.1.2.2.3)*

*Note: Girders used on the New South Punaluu Bridge are similar to the AASHTO Type III)

<u>From Table 6.1.2.2.3-1:</u> *(for AASHTO Type III Girder)*

a0 =	-0.0938	a5 =	0.0054
a1 =	1.6197	a6 =	0.019
a2 =	-1.4792	a7 =	0.6044
a3 =	0.5367	a8 =	-0.283
a4 =	-0.0877		

x = 1.2593333

y = 0.0240609

<u>Horizontal Quasi-Static Wave Force:</u> *(eq 6.1.2.2.3)*

Fh-av = 0.3958665 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)*

Mt-av = 87.89 (kip/ft)-ft

Length of Bridge = 66.00 ft
Mt-av Total = 5801.05 kip-ft 2900.53 tons-ft

● **Resulting Quasi-Static Vertical Force and Associated Forces and Moments:** *(AASHTO sec 6.1.2.2)*

Fv-max Total =	891.45	kips	*(Quasi-Static Vertical Force)*
Fs Total =	162.76	kips	*(Vertical Slamming Force)*
Fh-av Total =	26.13	kips	*(Quasi-Static Horizontal Force)*
Mt-av =	5801.05	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)*

check = 236.35 > W = 50.00 ft Use eq 6.1.2.3.1-4 for omega

ω = 50.00 ft

Reference Horizontal Force: *(eq 6.1.2.3.1-2)*

F*h-max = 1.43 kip/ft

Horizontal Wave Force: *(eq 6.1.2.3.1-1)*

Fh-max = 1.1434522 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)*

α check: *(eq 6.1.2.3.2-3 or eq 6.1.2.3.2-4)*

check = 96.09 > W = 50.00 ft Use eq 6.1.2.3.2-4 for alpha

α = 50.00 ft

Reference Vertical Force: *(eq 6.1.2.3.2-2)*

F*v-ah = 22.67 kip/ft

Quasi-Static Vertical Wave Force: *(eq 6.1.2.3.2-1)*

Fv-ah = 10.825717 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 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)*

Fs = 2.4660879 kip/ft

Length of Bridge = 66.00 ft

Fs Total = 162.76 kips 81.38 tons

● **<u>Associated Moment About Trailing Edge:</u>** *(AASHTO Sec 6.1.2.3.4)*

<u>Reference Moment:</u> *(eq 6.1.2.3.4-2)*
M*t-ah = 449.49207 (kip/ft)-ft

<u>Associated Moment about Trailing Edge:</u> *(eq 6.1.2.3.4-1)*
Mt-ah = 218.2946 (kip/ft)-ft

Length of Bridge = 66.00 ft
Mt-ah Total = 14407.44 kip-ft 7203.72 ton-ft

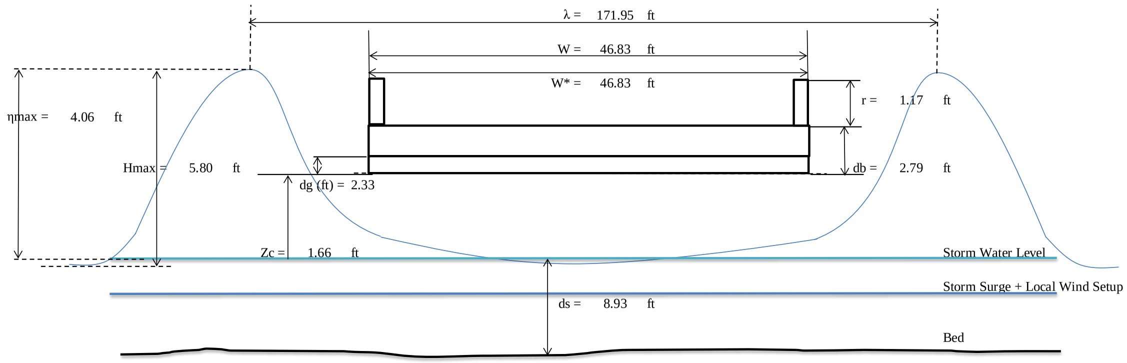
● **<u>Resulting Maximum Horizontal Wave Force and Associated Forces and Moments:</u>** *(AASHTO sec 6.1.2.3)*

Fh-max Total =	75.47	kips	*(Maximum Horizontal Wave Force)*
Fv-ah Total =	714.50	kips	*(Quasi-Static Vertical Force)*
Fs Total =	162.76	kips	*(Vertical Slamming Force)*
Mt-ah Total =	14407.44	kip-ft	*(Associated Moment about Trailing Edge)*

■ **<u>Current Loads on Superstructure:</u>** *(AASHTO Sec 6.1.2.4)*
*Note: Current loads are not considered in this study.

Fhc = 0 kips

New Makaha #3A Bridge:
Method For Estimating Wave Forces on Bridge Superstructures



AASHTO (2008)
Constant Coefficients:

50 year wind speed =	105	mph	*(from AASHTO Fig 6.2.2.2-1 b)*
specific weight water =	0.064	kip/cubic ft	
g =	32.2	ft/sec^2	

♦ **Wave Calculations:**

● **Bridge Properties:**

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
Girder to Girder Width =	46.83	ft	Height of girder =	2.33	ft
Deck Thickness =	0.46	ft	Height of railing =	1.17	ft
Av =	3278.33	sq ft	Elevation to bot. of girder =	10.59	ft
N =	1	girders	Elevation to bot. 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	
Ut* =	287.33	ft/sec	
ds =	8.93	ft	*(from NFIP Flood Hazard Assessment Tool)*
d =	27.10	ft	*(average water depth over fetch length)*
Fetch Length =	4767610.00	ft	
[(g*d)/(Ut*)^2] =	0.011		
Tp =	10.06	sec	

○ **Determination of time duration to develop fetch limited waves:**

t = 6336.05 sec > 3600 sec

U_1hr = 171.09 ft/sec To compute U_1hr use eq 6.2.2.4-5

Iteration Process: *(until 't' converges)*

First Iteration:

Ut = 169.91 ft/sec

Second Iteration:

Ut* = 298.38 ft/sec
[(g*d)/(Ut*)^2] = 0.010
Tp = 10.16 sec
t = 6166.22 sec

Third Iteration:

Ut = 169.96 sec
Ut* = 298.47 ft/sec
[(g*d)/(Ut*)^2] = 0.010
Tp = 10.16 sec
t = 6164.82 sec

Fifth Iteration:

Ut = 169.96 sec
Ut* = 298.47 ft/sec
[(g*d)/(Ut*)^2] = 0.010
Tp = 10.16 sec
t = 6164.81 sec

Fourth Iteration:

Ut = 169.96 sec
Ut* = 298.47 ft/sec
[(g*d)/(Ut*)^2] = 0.010
Tp = 10.16 sec
t = 6164.81 sec

Sixth Iteration:

Ut = 169.96 sec
Ut* = 298.47 ft/sec
[(g*d)/(Ut*)^2] = 0.010
Tp = 10.16 sec
t = 6164.81 sec *(the value of 't' has converged)*

○ **Wave height and wavelength:**

Hs = 12.92 ft
Wave Length (λ) = 171.95 ft
Hmax = 23.25 ft

Maximum Wave Height: *(least of the following)*

Hmax = 23.25 ft
Hmax ≤ 0.65*ds = 5.80 ft
Hmax ≤ λ/7.0 = 24.56 ft

Therefore Hmax = 5.80 ft

○ **Resulting Storm Wave Properties:** *(see Fig)*

Tp = 10.16 sec

Hmax =	5.80	ft
Wave Length (λ) =	171.95	ft
η 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-max parameters:** *(eq 6.1.2.2.1-1)*

W_hat = 36.80 ft
W_hat / W = 0.79 > 0.15 Therefore W_hat = W_hat
W_hat = 36.80 ft

η max - Zc = 2.40 < db = 2.79 ft
β = 0.86

x = 0.0337569

y = 0.214015

For Slab Spans: *(eq 6.1.2.2.1 b)*

b0 =	-0.196975	b4 =	-0.00075
b1 =	35.067917	b5 =	1.06277
b2 =	0.0471417	b6 =	25.83333
b3 =	-62.95042		

Tapped Air Factor:

A_air = 0.0074884
B_air = 0.2845313

(η max - Zc) / dg = 1.0299214 > 1
%Air = variable

However, the bridge is not a girder type bridge
therefore:
%Air = 0

TAF = 1 > 1 (O.K.)
TAF = 1

Quasi-Static Vertical Force: *(eq 6.1.2.2.1-1)*

Fv-max = 4.2371596 kip/ft

Length of Bridge = 70.00 ft
Fv-max Total = 296.60 kips 148.30 tons

● **Associated Vertical Slamming Force:** *(AASHTO Sec 6.1.2.2.2)*

B = -0.863728

Zc / η 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)*

a0 =	-0.0304	a5 =	0.0025
a1 =	1.4247	a6 =	0.0403
a2 =	-1.1168	a7 =	0.5503
a3 =	0.3455	a8 =	-0.3612
a4 =	-0.048		

x = 0.6071116

y = 0.0337569

Horizontal Quasi-Static Wave Force: *(eq 6.1.2.2.3)*

Fh-av =	0.3482357	kip/ft		
Length of Bridge =	70.00	ft		
Fh-av Total =	24.38	kips	12.19	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	ft
b_m =	-0.040658	ft
c_m =	-0.0049	ft
W' =	0.00	ft
W* =	46.83	ft

Associated Moment about Trailing Edge: *(eq 6.1.2.2.4-1)*

Mt-av =	48.10	(kip/ft)-ft		
Length of Bridge =	70.00	ft		
Mt-av Total =	3366.74	kip-ft	1683.37	tons-ft

● **Resulting Quasi-Static Vertical Force and Associated Forces and Moments:** *(AASHTO sec 6.1.2.2)*

Fv-max Total =	296.60	kips	*(Quasi-Static Vertical Force)*
Fs Total =	106.19	kips	*(Vertical Slamming Force)*
Fh-av Total =	24.38	kips	*(Quasi-Static Horizontal Force)*
Mt-av =	3366.74	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)*

check =	104.37	>	W =	46.83	ft	Use eq 6.1.2.3.1-4 for omega
ω =	46.83	ft				

Reference Horizontal Force: *(eq 6.1.2.3.1-2)*

F*h-max = 1.34 kip/ft

Horizontal Wave Force: *(eq 6.1.2.3.1-1)*

Fh-max =	0.76856	kip/ft		
Length of Bridge =	70.00	ft		
Fh-max Total =	53.80	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)*

check =	14.15	<	W =	46.83	ft	Use eq 6.1.2.3.2-3 for alpha
α =	14.15	ft				

Reference Vertical Force: *(eq 6.1.2.3.2-2)*

F*v-ah = 2.18 kip/ft

Quasi-Static Vertical Wave Force: *(eq 6.1.2.3.2-1)*

Fv-ah =	3.7502229	kip/ft		
Length of Bridge =	70.00	ft		
Fv-ah Total =	262.52	kip	131.26	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)*

Fs = 1.5170513 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 (kip/ft)-ft

Associated Moment about Trailing Edge: *(eq 6.1.2.3.4-1)*
Mt-ah = 188.53083 (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 sec 6.1.2.3)*

Fh-max Total =	53.80	kips	*(Maximum Horizontal Wave Force)*
Fv-ah Total =	262.52	kips	*(Quasi-Static Vertical Force)*
Fs Total =	106.19	kips	*(Vertical Slamming Force)*
Mt-ah Total =	13197.16	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

Old Makaha #3A Bridge:
Method For Estimating Wave Forces on Bridge Superstructures

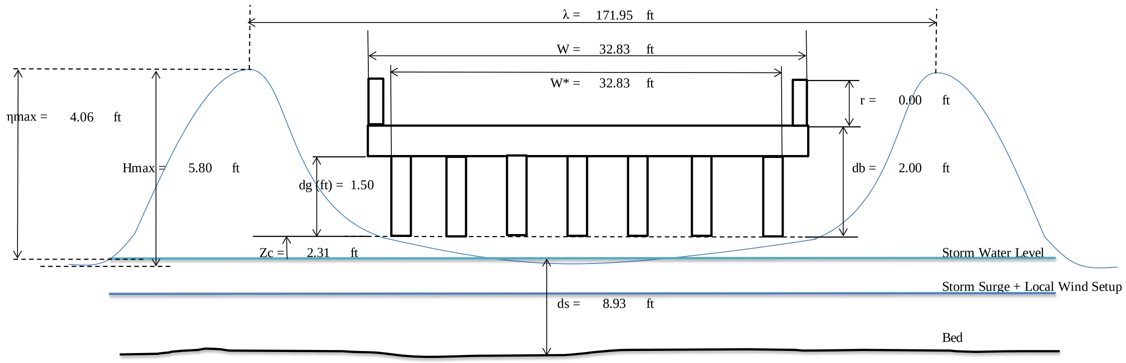
AASHTO (2008)
Constant Coefficients:

50 year wind speed =	105	mph	*(from AASHTO Fig 6.2.2.2-1 b)*
specific weight water =	0.064	kip/cubic ft	
g =	32.2	ft/sec^2	

♦ **Wave Calculations:**

● **Bridge Properties:**

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
Girder to Girder Width =	32.83	ft	Height of girder =	1.50	ft
Deck Thickness =	0.50	ft	Height of railing =	0.00	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



■ **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 =	8.93	ft	*(from NFIP Flood Hazard Assessment Tool)*			
d =	27.10	ft	*(average water depth over fetch length)*			
Fetch Length =	#########	ft				
[(g*d)/(Ut*)^2] =	0.011					
Tp =	10.06	sec				

○ **Determination of time duration to develop fetch limited waves:**

t = 6336.05 sec > 3600 sec

U_1hr = 171.09 ft/sec To compute U_1hr use eq 6.2.2.4-5

Iteration Process: *(until 't' converges)*

First Iteration:

Ut = 169.91 ft/sec

Second Iteration:

Ut* =	298.38	ft/sec
[(g*d)/(Ut*)^2] =	0.010	
Tp =	10.16	sec
t =	6166.22	sec

Third Iteration:

Ut =	169.96	sec
Ut* =	298.47	ft/sec
[(g*d)/(Ut*)^2] =	0.010	
Tp =	10.16	sec
t =	6164.82	sec

Fourth Iteration:

Ut =	169.96	sec
Ut* =	298.47	ft/sec
[(g*d)/(Ut*)^2] =	0.010	
Tp =	10.16	sec
t =	6164.81	sec

Fifth Iteration:

Ut =	169.96	sec
Ut* =	298.47	ft/sec
[(g*d)/(Ut*)^2] =	0.010	
Tp =	10.16	sec
t =	6164.81	sec

Sixth Iteration:

Ut =	169.96	sec
Ut* =	298.47	ft/sec
[(g*d)/(Ut*)^2] =	0.010	
Tp =	10.16	sec
t =	6164.81	sec

(the value of 't' has converged)

○ **Wave height and wavelength:**

Hs =	12.92	ft
Wave Length (λ) =	171.95	ft
Hmax =	23.25	ft

Maximum Wave Height: *(least of the following)*

Hmax =	23.25	ft
Hmax ≤ 0.65*ds =	5.80	ft
Hmax ≤ λ/7.0 =	24.56	ft
Therefore Hmax =	5.80	ft

○ **Resulting Storm Wave Properties:** *(see Fig)*

Tp = 10.16 sec

Hmax =	5.80	ft
Wave Length (λ) =	171.95	ft
η 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-max parameters:** *(eq 6.1.2.2.1-1)*

W_hat =	17.54	ft			
W_hat / W =	0.53	>	0.15	Therefore W_hat = W_hat	
W_hat =	17.54	ft			
η max - Zc =	1.75	<	db =	2.00	ft
β =	0.88				

x = 0.0337569

y = 0.1020329

For Girder Spans: *(eq 6.1.2.2.1 a)*

b0 =	-0.738	b4 =	-0.00039
b1 =	56.43	b5 =	-0.332
b2 =	0.0496	b6 =	4.71
b3 =	-193.2472		

Tapped Air Factor:

A_air = 0.0074333

B_air = 0.2975314

(η max - Zc) / dg = 1.1687667 > 1

%Air = 100.00

Assume 50% air pocket:

%Air = 50

TAF = 0.6691971 < 1 (O.K.)

TAF = 0.6691971

Quasi-Static Vertical Force: *(eq 6.1.2.2.1-1)*

Fv-max = 1.3744929 kip/ft

Length of Bridge = 78.83 ft

Fv-max Total = 108.36 kips 54.18 tons

● **Associated Vertical Slamming Force:** *(AASHTO Sec 6.1.2.2.2)*

B = -0.674879

Zc / η max = 0.5685244 > 0

A = 0.040071

Vertical Slamming Force: *(eq 6.1.2.2.2-1)*

Fs = 0.8505833 kip/ft

Length of Bridge = 78.83 ft

Fs Total = 67.05 kips 33.53 tons

● **Associated Horizontal Quasi-Static Wave Force:** *(AASHTO Sec 6.1.2.2.3)*

*Note: Girders used on the Old Makaha #3A Bridge are similar to 21 in Voided Slabs)

From Table 6.1.2.2.3-1: *(for 21" Voided Slab)*

a0 =	0.0123	a5 =	0.0018
a1 =	1.3927	a6 =	0.0628
a2 =	-1.0131	a7 =	0.5654
a3 =	0.2953	a8 =	-0.4112
a4 =	-0.0385		

x = 0.876575

y = 0.0337569

Horizontal Quasi-Static Wave Force: *(eq 6.1.2.2.3)*

Fh-av = 0.4398445 kip/ft

Length of Bridge = 78.83 ft
Fh-av Total = 34.67 kips 17.34 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 =	0.901	ft
b_m =	-0.07922	ft
c_m =	-0.003568	ft
W' =	0.00	ft
W* =	32.83	ft

Associated Moment about Trailing Edge: *(eq 6.1.2.2.4-1)*
Mt-av = 18.95 (kip/ft)-ft

Length of Bridge = 78.83 ft
Mt-av Total = 1494.13 kip-ft 747.06 tons-ft

● **Resulting Quasi-Static Vertical Force and Associated Forces and Moments:** *(AASHTO sec 6.1.2.2)*

Fv-max Total =	108.36	kips	*(Quasi-Static Vertical Force)*
Fs Total =	67.05	kips	*(Vertical Slamming Force)*
Fh-av Total =	34.67	kips	*(Quasi-Static Horizontal Force)*
Mt-av =	1494.13	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)*
check = 94.75 > W = 32.83 ft Use eq 6.1.2.3.1-4 for omega

ω = 32.83 ft

Reference Horizontal Force: *(eq 6.1.2.3.1-2)*
F*h-max = 0.49 kip/ft

Horizontal Wave Force: *(eq 6.1.2.3.1-1)*
Fh-max = 0.4433121 kip/ft

Length of Bridge = 78.83 ft
Fh-max Total = 34.95 kip 17.47 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 = 6.75 < W = 32.83 ft Use eq 6.1.2.3.2-3 for alpha

α = 6.75 ft

Reference Vertical Force: *(eq 6.1.2.3.2-2)*
F*v-ah = 0.76 kip/ft

Quasi-Static Vertical Wave Force: *(eq 6.1.2.3.2-1)*
Fv-ah = 1.3787925 kip/ft

Length of Bridge = 78.83 ft
Fv-ah Total = 108.69 kip 54.35 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)*
Fs = 0.8505833 kip/ft

Length of Bridge = 78.83 ft

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*t-ah = 49.685183 (kip/ft)-ft

Associated Moment about Trailing Edge: *(eq 6.1.2.3.4-1)*
Mt-ah = 55.454519 (kip/ft)-ft

Length of Bridge = 78.83 ft
Mt-ah Total = 4371.66 kip-ft 2185.83 ton-ft

● **Resulting Maximum Horizontal Wave Force and Associated Forces and Moments:** *(AASHTO sec 6.1.2.3)*

Fh-max Total =	34.95	kips	*(Maximum Horizontal Wave Force)*
Fv-ah Total =	108.69	kips	*(Quasi-Static Vertical Force)*
Fs Total =	67.05	kips	*(Vertical Slamming Force)*
Mt-ah Total =	4371.66	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

Maipalaoa Bridge:
Method For Estimating Wave Forces on Bridge Superstructures

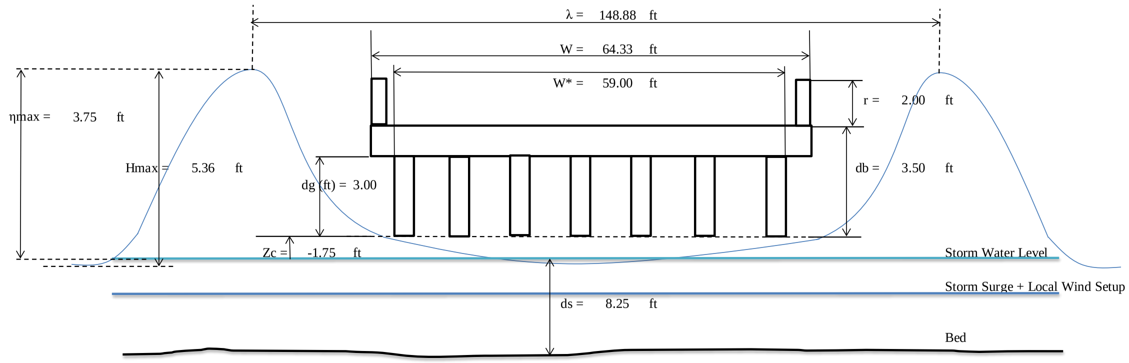
AASHTO (2008)
Constant Coefficients:

50 year wind speed =	105	mph	*(from AASHTO Fig 6.2.2.2-1 b)*
specific weight water =	0.064	kip/cubic ft	
g =	32.2	ft/sec^2	

♦ **Wave Calculations:**

● **Bridge Properties:**

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
Girder to Girder Width =	59.00	ft	Height of girder =	3.00	ft
Deck Thickness =	0.50	ft	Height of railing =	2.00	ft
Av =	6476.22	sq ft	Elevation to bot. of girder =	6.50	ft
N =	16	girders	Elevation to bot. of deck =	9.50	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	ft/sec	
ds =	8.25	ft	*(from NFIP Flood Hazard Assessment Tool)*
d =	20.59	ft	*(average water depth over fetch length)*
Fetch Length =	4767610.00	ft	
[(g*d)/(Ut*)^2] =	0.008		
Tp =	9.10	sec	

○ **Determination of time duration to develop fetch limited waves:**

t = 5022.33 sec > 3600 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

Second Iteration:
Ut* = 293.56 ft/sec
[(g*d)/(Ut*)^2] = 0.008
Tp = 9.16 sec
t = 4944.65 sec

Third Iteration:
Ut = 167.71 sec
Ut* = 293.63 ft/sec
[(g*d)/(Ut*)^2] = 0.008
Tp = 9.16 sec
t = 4943.88 sec

Fourth Iteration:
Ut = 167.71 sec
Ut* = 293.63 ft/sec
[(g*d)/(Ut*)^2] = 0.008
Tp = 9.16 sec
t = 4943.87 sec

Fifth Iteration:
Ut = 167.71 sec
Ut* = 293.63 ft/sec
[(g*d)/(Ut*)^2] = 0.008
Tp = 9.16 sec
t = 4943.87 sec

Sixth Iteration:
Ut = 167.71 sec
Ut* = 293.63 ft/sec
[(g*d)/(Ut*)^2] = 0.008
Tp = 9.16 sec
t = 4943.87 sec *(the value of 't' has converged)*

○ **Wave height and wavelength:**
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:** *(see Fig)*
Tp = 9.16 sec

Hmax =	5.36	ft
Wave Length (λ) =	148.88	ft
η max =	3.75	ft

■ **<u>Maximum Quasi-Static Vertical Force and Associated Forces and Moments:</u>** *(AASHTO Sec 6.1.2.2)*

● **<u>Maximum Quasi-Static Vertical Force:</u>** *(AASHTO Sec 6.1.2.2.1)*

○ **<u>Determination of Fv-max parameters:</u>** *(eq 6.1.2.2.1-1)*

W_hat = 123.02 ft
W_hat / W = 1.91 > 0.15 Therefore W_hat = W_hat
W_hat = 123.02 ft

η max - Zc = 5.50 > db = 3.50 ft
β = 1.00

x = 0.03602

y = 0.8263403

<u>For Girder Spans:</u> *(eq 6.1.2.2.1 a)*

b0 =	-0.888	b4 =	-0.00048
b1 =	56.16	b5 =	-0.056
b2 =	0.0538	b6 =	7.86
b3 =	-192.8944		

<u>Tapped Air Factor:</u>

A_air = 0.0039527
B_air = 0.5994428

(η max - Zc) / dg = 1.8345833 > 1
%Air = -83.46

Assume 50% air pocket:
%Air = 50

TAF = 0.7970769 < 1 (O.K.)
TAF = 0.7970769

<u>Quasi-Static Vertical Force:</u> *(eq 6.1.2.2.1-1)*

Fv-max = 9.6207819 kip/ft

Length of Bridge = 100.67 ft
Fv-max Total = 968.49 kips 484.25 tons

● **<u>Associated Vertical Slamming Force:</u>** *(AASHTO Sec 6.1.2.2.2)*

B = -1.3000709

Zc / η max = -0.4662005 < 0
A = 0.0246536

<u>Vertical Slamming Force:</u> *(eq 6.1.2.2.2-1)*

Fs = 3.4150059 kip/ft

Length of Bridge = 100.67 ft
Fs Total = 343.78 kips 171.89 tons

● **<u>Associated Horizontal Quasi-Static Wave Force:</u>** *(AASHTO Sec 6.1.2.2.3)*

*Note: Girders used on the Maipalaoa Bridge are similar to AASHTO Type III girders)

<u>From Table 6.1.2.2.3-1:</u> *(for AASHTO Type III)*

a0 =	-0.0938	a5 =	0.0054
a1 =	1.6197	a6 =	0.019
a2 =	-1.4792	a7 =	0.6044
a3 =	0.5367	a8 =	-0.283
a4 =	-0.0877		

x = 1.0006818

y = 0.03602

<u>Horizontal Quasi-Static Wave Force:</u> *(eq 6.1.2.2.3)*

Fh-av = 0.3885508 kip/ft

Length of Bridge = 100.67 ft
Fh-av Total = 39.11 kips 19.56 tons

● **<u>Associated Moment about the Trailing Edge Due to the Quasi-static and Slamming Forces:</u>** *(AASHTO Sec 6.1.2.2.4)*

<u>For Girder Spans:</u>

a_m = 0.85725 ft
b_m = -0.056155 ft
c_m = -0.004387 ft
W' = 5.33 ft
W* = 59.00 ft

<u>Associated Moment about Trailing Edge:</u> *(eq 6.1.2.2.4-1)*

Mt-av = 166.69 (kip/ft)-ft

Length of Bridge = 100.67 ft
Mt-av Total = 16779.78 kip-ft 8389.89 tons-ft

● **<u>Resulting Quasi-Static Vertical Force and Associated Forces and Moments:</u>** *(AASHTO sec 6.1.2.2)*

Fv-max Total =	968.49	kips	*(Quasi-Static Vertical Force)*
Fs Total =	343.78	kips	*(Vertical Slamming Force)*
Fh-av Total =	39.11	kips	*(Quasi-Static Horizontal Force)*
Mt-av =	16779.78	kip-ft	*(Associated Moment about Trailing Edge)*

■ **<u>Maximum Horizontal Wave Force and Associated Forces and Moments:</u>** *(AASHTO Sec 6.1.2.3)*

● **<u>Maximum Horizontal Wave Force:</u>** *(AASHTO Sec 6.1.2.3.1)*

<u>ω check:</u> *(eq 6.1.2.3.1-3 or eq 6.1.2.3.1-4)*

check = 135.95 > W = 64.33 ft Use eq 6.1.2.3.1-4 for omega

ω = 64.33 ft

<u>Reference Horizontal Force:</u> *(eq 6.1.2.3.1-2)*

F*h-max = 2.67 kip/ft

<u>Horizontal Wave Force:</u> *(eq 6.1.2.3.1-1)*

Fh-max = 1.2744943 kip/ft

Length of Bridge = 100.67 ft
Fh-max Total = 128.30 kip 64.15 tons

● **<u>Associated Quasi-Static Vertical Force:</u>** *(AASHTO Sec 6.1.2.3.2)*

<u>α check:</u> *(eq 6.1.2.3.2-3 or eq 6.1.2.3.2-4)*

check = 47.32 < W = 64.33 ft Use eq 6.1.2.3.2-3 for alpha

α = 47.32 ft

<u>Reference Vertical Force:</u> *(eq 6.1.2.3.2-2)*

F*v-ah = 16.67 kip/ft

<u>Quasi-Static Vertical Wave Force:</u> *(eq 6.1.2.3.2-1)*

Fv-ah = 8.0648175 kip/ft

Length of Bridge = 100.67 ft
Fv-ah Total = 811.86 kip 405.93 tons

● **<u>Associated Vertical Slamming Forces:</u>** *(AASHTO Sec 6.1.2.3.3)*

*Note: Slamming force is calculated using the same method as AASHTO sec 6.1.2.2.2)

<u>Vertical Slamming Force:</u> *(eq 6.1.2.2.2-1)*

Fs = 3.4150059 kip/ft

Length of Bridge = 100.67 ft

Fs Total = 343.78 kips 171.89 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 = 499.36659 (kip/ft)-ft

Associated Moment about Trailing Edge: *(eq 6.1.2.3.4-1)*
Mt-ah = 384.56577 (kip/ft)-ft

Length of Bridge = 100.67 ft
Mt-ah Total = 38712.95 kip-ft 19356.48 ton-ft

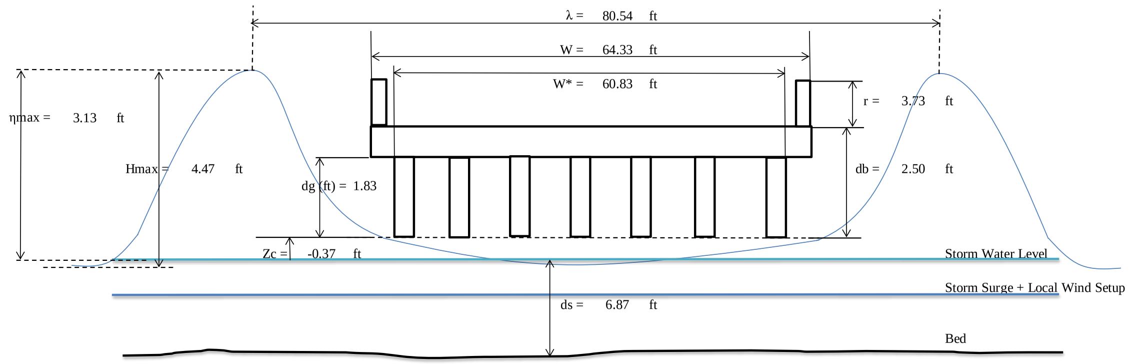
● **Resulting Maximum Horizontal Wave Force and Associated Forces and Moments:** *(AASHTO sec 6.1.2.3)*

Fh-max Total =	128.30	kips	*(Maximum Horizontal Wave Force)*
Fv-ah Total =	811.86	kips	*(Quasi-Static Vertical Force)*
Fs Total =	343.78	kips	*(Vertical Slamming Force)*
Mt-ah Total =	38712.95	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

Moanalua Bridge: *(single span)*
Method For Estimating Wave Forces on Bridge Superstructures



AASHTO (2008)
Constant Coefficients:

50 year wind speed =	105	mph	*(from AASHTO Fig 6.2.2.2-1 b)*
specific weight water =	0.064	kip/cubic ft	
g =	32.2	ft/sec^2	

♦ **Wave Calculations:**

● **Bridge Properties:**

Bridge Deck Width =	64.33	ft	Water Depth =	2.50	ft
Bridge Deck Length =	27.00	ft	Water surface to bot. of girder =	4.00	ft
Girder to Girder Width =	60.83	ft	Height of girder =	1.83	ft
Deck Thickness =	0.67	ft	Height of railing =	3.73	ft
Av =	1737.00	sq ft	Elevation to bot. of girder =	6.50	ft
N =	9	girders	Elevation to bot. of deck =	8.33	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 =	6.87	ft	*(from NFIP Flood Hazard Assessment Tool)*
d =	7.42	ft	*(average water depth over fetch length)*
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

U_1hr = 109.17 ft/sec To compute U_1hr use eq 6.2.2.4-4

Iteration Process: *(until 't' converges)*

First Iteration:

Ut = 110.17 ft/sec

Second Iteration:

Ut* =	175.12	ft/sec
[(g*d)/(Ut*)^2] =	0.008	
Tp =	5.50	sec
t =	2994.53	sec

Third Iteration:

Ut =	109.44	sec
Ut* =	173.68	ft/sec
[(g*d)/(Ut*)^2] =	0.008	
Tp =	5.49	sec
t =	3012.51	sec

Fourth Iteration:

Ut =	109.43	sec
Ut* =	173.66	ft/sec
[(g*d)/(Ut*)^2] =	0.008	
Tp =	5.49	sec
t =	3012.77	sec

Fifth Iteration:

Ut =	109.43	sec
Ut* =	173.66	ft/sec
[(g*d)/(Ut*)^2] =	0.008	
Tp =	5.49	sec
t =	3012.77	sec

Sixth Iteration:

Ut =	109.43	sec
Ut* =	173.66	ft/sec
[(g*d)/(Ut*)^2] =	0.008	
Tp =	5.49	sec
t =	3012.77	sec

(the value of 't' has converged)

○ **Wave height and wavelength:**

Hs =	3.73	ft
Wave Length (λ) =	80.54	ft
Hmax =	6.71	ft

Maximum Wave Height: *(least of the following)*

Hmax =	6.71	ft
Hmax ≤ 0.65*ds =	4.47	ft
Hmax ≤ λ/7.0 =	11.51	ft

Therefore Hmax = 4.47 ft

○ **Resulting Storm Wave Properties:** *(see Fig)*

Tp = 5.49 sec

Hmax =	4.47	ft
Wave Length (λ) =	80.54	ft
η max =	3.13	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-max parameters:** *(eq 6.1.2.2.1-1)*

W_hat = 46.95 ft
W_hat / W = 0.73 > 0.15 Therefore W_hat = W_hat
W_hat = 46.95 ft

η max - Zc = 3.50 > db = 2.50 ft
β = 1.00

x = 0.055442

y = 0.5828575

For Girder Spans: *(eq 6.1.2.2.1 a)*

b0 =	-0.7713333	b4 =	-0.00041
b1 =	56.37	b5 =	-0.27067
b2 =	0.0505333	b6 =	5.41
b3 =	-193.1688		

Tapped Air Factor:

A_air = 0.0069199
B_air = 0.2927231

(η max - Zc) / dg = 1.9068273 > 1
%Air = 100.00

Assume 50% air pocket:
%Air = 50

TAF = 0.6387196 < 1 (O.K.)
TAF = 0.6387196

Quasi-Static Vertical Force: *(eq 6.1.2.2.1-1)*

Fv-max = 3.3863839 kip/ft

Length of Bridge = 27.00 ft
Fv-max Total = 91.43 kips 45.72 tons

● **Associated Vertical Slamming Force:** *(AASHTO Sec 6.1.2.2.2)*

B = -1.2473094

Zc / η max = -0.1183678 < 0
A = 0.0298363

Vertical Slamming Force: *(eq 6.1.2.2.2-1)*

Fs = 1.4043874 kip/ft

Length of Bridge = 27.00 ft
Fs 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: *(for AASHTO Florida Bulb - T72)*

a0 =	-0.2076	a5 =	-0.0167
a1 =	1.5772	a6 =	-0.0346
a2 =	-1.048	a7 =	0.5282
a3 =	0.0551	a8 =	-0.139
a4 =	0.093		

x = 0.5612067

y = 0.055442

Horizontal Quasi-Static Wave Force: *(eq 6.1.2.2.3)*

Fh-av = 0.2478735 kip/ft

Length of Bridge = 27.00 ft
Fh-av Total = 6.69 kips 3.35 tons

● **<u>Associated Moment about the Trailing Edge Due to the Quasi-static and Slamming Forces:</u>** *(AASHTO Sec 6.1.2.2.4)*

<u>For Girder Spans:</u>

a_m = 0.8481354 ft
b_m = -0.0513498 ft
c_m = -0.0045576 ft
W' = 3.50 ft
W* = 60.83 ft

<u>Associated Moment about Trailing Edge:</u> *(eq 6.1.2.2.4-1)*

Mt-av = 66.36 (kip/ft)-ft

Length of Bridge = 27.00 ft
Mt-av Total = 1791.59 kip-ft 895.79 tons-ft

● **<u>Resulting Quasi-Static Vertical Force and Associated Forces and Moments:</u>** *(AASHTO sec 6.1.2.2)*

Fv-max Total =	91.43	kips	*(Quasi-Static Vertical Force)*
Fs Total =	37.92	kips	*(Vertical Slamming Force)*
Fh-av Total =	6.69	kips	*(Quasi-Static Horizontal Force)*
Mt-av =	1791.59	kip-ft	*(Associated Moment about Trailing Edge)*

■ **<u>Maximum Horizontal Wave Force and Associated Forces and Moments:</u>** *(AASHTO Sec 6.1.2.3)*

● **<u>Maximum Horizontal Wave Force:</u>** *(AASHTO Sec 6.1.2.3.1)*

<u>ω check:</u> *(eq 6.1.2.3.1-3 or eq 6.1.2.3.1-4)*

check = 63.74 < W = 64.33 ft Use eq 6.1.2.3.1-3 for omega

ω = 63.74 ft

<u>Reference Horizontal Force:</u> *(eq 6.1.2.3.1-2)*

F*h-max = 4.58 kip/ft

<u>Horizontal Wave Force:</u> *(eq 6.1.2.3.1-1)*

Fh-max = 0.7626165 kip/ft

Length of Bridge = 27.00 ft
Fh-max Total = 20.59 kip 10.30 tons

● **<u>Associated Quasi-Static Vertical Force:</u>** *(AASHTO Sec 6.1.2.3.2)*

<u>α check:</u> *(eq 6.1.2.3.2-3 or eq 6.1.2.3.2-4)*

check = 18.06 < W = 64.33 ft Use eq 6.1.2.3.2-3 for alpha

α = 18.06 ft

<u>Reference Vertical Force:</u> *(eq 6.1.2.3.2-2)*

F*v-ah = 4.04 kip/ft

<u>Quasi-Static Vertical Wave Force:</u> *(eq 6.1.2.3.2-1)*

Fv-ah = 1.9231889 kip/ft

Length of Bridge = 27.00 ft
Fv-ah Total = 51.93 kip 25.96 tons

● **<u>Associated Vertical Slamming Forces:</u>** *(AASHTO Sec 6.1.2.3.3)*

*Note: Slamming force is calculated using the same method as AASHTO sec 6.1.2.2.2)

<u>Vertical Slamming Force:</u> *(eq 6.1.2.2.2-1)*

Fs = 1.4043874 kip/ft

Length of Bridge = 27.00 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*t-ah = 147.46652 (kip/ft)-ft

Associated Moment about Trailing Edge: *(eq 6.1.2.3.4-1)*
Mt-ah = 124.02284 (kip/ft)-ft

Length of Bridge =	27.00	ft		
Mt-ah Total =	3348.62	kip-ft	1674.31	ton-ft

● **Resulting Maximum Horizontal Wave Force and Associated Forces and Moments:** *(AASHTO sec 6.1.2.3)*

Fh-max Total =	20.59	kips	*(Maximum Horizontal Wave Force)*
Fv-ah Total =	51.93	kips	*(Quasi-Static Vertical Force)*
Fs Total =	37.92	kips	*(Vertical Slamming Force)*
Mt-ah Total =	3348.62	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

Kalihi Bridge: *(single span)*
Method For Estimating Wave Forces on Bridge Superstructures

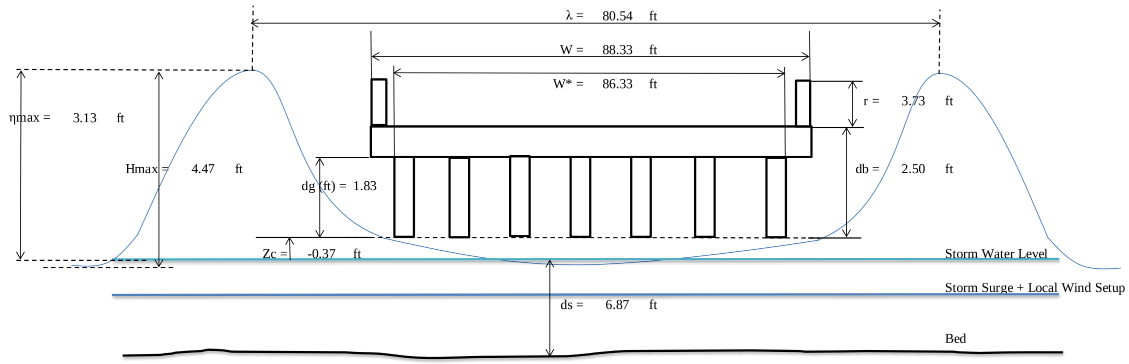
AASHTO (2008)
Constant Coefficients:

50 year wind speed =	105	mph	*(from AASHTO Fig 6.2.2.2-1 b)*
specific weight water =	0.064	kip/cubic ft	
g =	32.2	ft/sec^2	

♦ **Wave Calculations:**

● **Bridge Properties:**

Bridge Deck Width =	88.33	ft	Water Depth =	2.50	ft
Bridge Deck Length =	27.00	ft	Water surface to bot. of girder =	4.00	ft
Girder to Girder Width =	86.33	ft	Height of girder =	1.83	ft
Deck Thickness =	0.67	ft	Height of railing =	3.73	ft
Av =	2385.00	sq ft	Elevation to bot. of girder =	6.50	ft
N =	13	girders	Elevation to bot. of deck =	8.33	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 =	6.87	ft	*(from NFIP Flood Hazard Assessment Tool)*
d =	7.42	ft	*(average water depth over fetch length)*
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

U_1hr = 109.17 ft/sec To compute U_1hr use eq 6.2.2.4-4

Iteration Process: *(until 't' converges)*

First Iteration:
Ut = 110.17 ft/sec

Second Iteration:
Ut* = 175.12 ft/sec
[(g*d)/(Ut*)^2] = 0.008
Tp = 5.50 sec
t = 2994.53 sec

Third Iteration:
Ut = 109.44 sec
Ut* = 173.68 ft/sec
[(g*d)/(Ut*)^2] = 0.008
Tp = 5.49 sec
t = 3012.51 sec

Fourth Iteration:
Ut = 109.43 sec
Ut* = 173.66 ft/sec
[(g*d)/(Ut*)^2] = 0.008
Tp = 5.49 sec
t = 3012.77 sec

Fifth Iteration:
Ut = 109.43 sec
Ut* = 173.66 ft/sec
[(g*d)/(Ut*)^2] = 0.008
Tp = 5.49 sec
t = 3012.77 sec

Sixth Iteration:
Ut = 109.43 sec
Ut* = 173.66 ft/sec
[(g*d)/(Ut*)^2] = 0.008
Tp = 5.49 sec
t = 3012.77 sec *(the value of 't' has converged)*

○ **Wave height and wavelength:**
Hs = 3.73 ft
Wave Length (λ) = 80.54 ft
Hmax = 6.71 ft

Maximum Wave Height: *(least of the following)*
Hmax = 6.71 ft
Hmax ≤ 0.65*ds = 4.47 ft
Hmax ≤ λ/7.0 = 11.51 ft

Therefore Hmax = 4.47 ft

○ **Resulting Storm Wave Properties:** *(see Fig)*
Tp = 5.49 sec

Hmax =	4.47	ft
Wave Length (λ) =	80.54	ft
η max =	3.13	ft

■ **<u>Maximum Quasi-Static Vertical Force and Associated Forces and Moments:</u>** *(AASHTO Sec 6.1.2.2)*

● **<u>Maximum Quasi-Static Vertical Force:</u>** *(AASHTO Sec 6.1.2.2.1)*

○ **<u>Determination of Fv-max parameters:</u>** *(eq 6.1.2.2.1-1)*

W_hat = 46.95 ft
W_hat / W = 0.53 > 0.15 Therefore W_hat = W_hat
W_hat = 46.95 ft

η max - Zc = 3.50 > db = 2.50 ft
β = 1.00

x = 0.055442

y = 0.5828575

<u>For Girder Spans:</u> *(eq 6.1.2.2.1 a)*

b0 =	-0.7713333	b4 =	-0.00041
b1 =	56.37	b5 =	-0.27067
b2 =	0.0505333	b6 =	5.41
b3 =	-193.1688		

<u>Tapped Air Factor:</u>

A_air = 0.0073638
B_air = 0.2576135

(η max - Zc) / dg = 1.9068273 > 1
%Air = 100.00

Assume 50% air pocket:
%Air = 50

TAF = 0.6258027 < 1 (O.K.)
TAF = 0.6258027

<u>Quasi-Static Vertical Force:</u> *(eq 6.1.2.2.1-1)*

Fv-max = 3.3179007 kip/ft

Length of Bridge = 27.00 ft
Fv-max Total = 89.58 kips 44.79 tons

● **<u>Associated Vertical Slamming Force:</u>** *(AASHTO Sec 6.1.2.2.2)*

B = -1.2473094

Zc / η max = -0.1183678 < 0
A = 0.0298363

<u>Vertical Slamming Force:</u> *(eq 6.1.2.2.2-1)*

Fs = 1.4043874 kip/ft

Length of Bridge = 27.00 ft
Fs Total = 37.92 kips 18.96 tons

● **<u>Associated Horizontal Quasi-Static Wave Force:</u>** *(AASHTO Sec 6.1.2.2.3)*

*Note: Girders used on the Maipalaoa Bridge are similar to Florida Bulb - T 72s)

<u>From Table 6.1.2.2.3-1:</u> *(for AASHTO Florida Bulb - T72)*

a0 =	-0.2076	a5 =	-0.0167
a1 =	1.5772	a6 =	-0.0346
a2 =	-1.048	a7 =	0.5282
a3 =	0.0551	a8 =	-0.139
a4 =	0.093		

x = 0.5612067

y = 0.055442

<u>Horizontal Quasi-Static Wave Force:</u> *(eq 6.1.2.2.3)*

Fh-av = 0.2232638 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 ft
b_m = -0.0513498 ft
c_m = -0.0045576 ft
W' = 2.00 ft
W* = 86.33 ft

Associated Moment about Trailing Edge: *(eq 6.1.2.2.4-1)*

Mt-av = 86.60 (kip/ft)-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)*

ω check: *(eq 6.1.2.3.1-3 or eq 6.1.2.3.1-4)*

check = 63.74 < W = 88.33 ft Use eq 6.1.2.3.1-3 for omega

ω = 63.74 ft

Reference Horizontal Force: *(eq 6.1.2.3.1-2)*

F*h-max = 4.58 kip/ft

Horizontal Wave Force: *(eq 6.1.2.3.1-1)*

Fh-max = 0.7626165 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)*

α check: *(eq 6.1.2.3.2-3 or eq 6.1.2.3.2-4)*

check = 18.06 < W = 88.33 ft Use eq 6.1.2.3.2-3 for alpha

α = 18.06 ft

Reference Vertical Force: *(eq 6.1.2.3.2-2)*

F*v-ah = 4.04 kip/ft

Quasi-Static Vertical Wave Force: *(eq 6.1.2.3.2-1)*

Fv-ah = 1.8842961 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)*

Fs = 1.4043874 kip/ft

Length of Bridge = 27.00 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*t-ah = 198.41738 (kip/ft)-ft

Associated Moment about Trailing Edge: *(eq 6.1.2.3.4-1)*
Mt-ah = 166.87372 (kip/ft)-ft

Length of Bridge = 27.00 ft
Mt-ah Total = 4505.59 kip-ft 2252.80 ton-ft

● **Resulting Maximum Horizontal Wave Force and Associated Forces and Moments:** *(AASHTO sec 6.1.2.3)*

Fh-max Total =	20.59	kips	*(Maximum Horizontal Wave Force)*
Fv-ah Total =	50.88	kips	*(Quasi-Static Vertical Force)*
Fs Total =	37.92	kips	*(Vertical Slamming Force)*
Mt-ah Total =	4505.59	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 #2
Method For Estimating Wave Forces on Bridge Superstructures

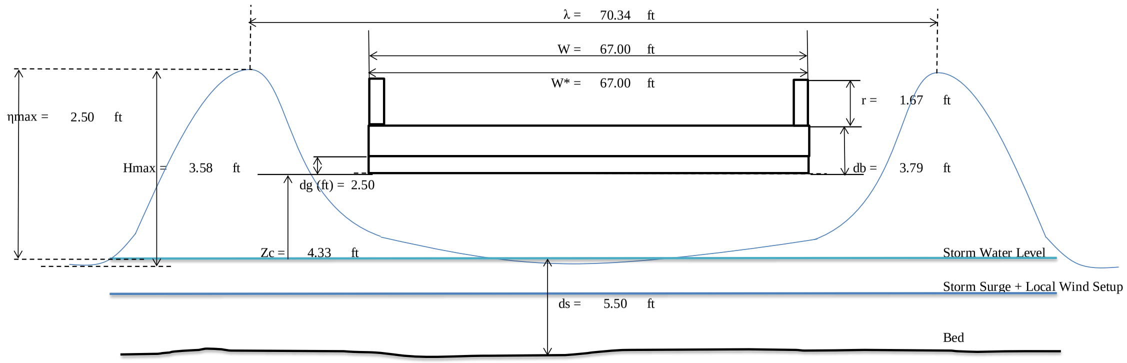
AASHTO (2008)
Constant Coefficients:

50 year wind speed =	105	mph	*(from AASHTO Fig 6.2.2.2-1 b)*
specific weight water =	0.064	kip/cubic ft	
g =	32.2	ft/sec^2	

♦ **Wave Calculations:**

• **Bridge Properties:**

Bridge Deck Width =	67.00	ft	Water Depth =	5.33	ft
Bridge Deck Length =	178.41	ft	Water surface to bot. of girder =	4.50	ft
Girder to Girder Width =	67.00	ft	Height of girder =	2.50	ft
Deck Thickness =	1.29	ft	Height of railing =	1.67	ft
Av =	11953.47	sq ft	Elevation to bot. of girder =	9.83	ft
N =	11	girders	Elevation to bot. of deck =	12.33	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)*
d =	6.88	ft	*(average water depth over fetch length)*
Fetch Length =	4767610.00	ft	
[(g*d)/(Ut*)^2] =	0.003		
Tp =	6.07	sec	

○ **Determination of time duration to develop fetch limited waves:**

t = 1947.76 sec < 3600 sec

U_1hr = 109.17 ft/sec To compute U_1hr use eq 6.2.2.4-4

Iteration Process: *(until 't' converges)*

First Iteration:
Ut = 110.32 ft/sec

Second Iteration:
Ut* = 175.42 ft/sec
[(g*d)/(Ut*)^2] = 0.007
Tp = 5.35 sec
t = 2800.42 sec

Third Iteration:
Ut = 109.56 sec
Ut* = 173.92 ft/sec
[(g*d)/(Ut*)^2] = 0.007
Tp = 5.33 sec
t = 2818.03 sec

Fourth Iteration:
Ut = 109.55 sec
Ut* = 173.90 ft/sec
[(g*d)/(Ut*)^2] = 0.007
Tp = 5.33 sec
t = 2818.29 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)*

○ **Wave height and wavelength:**
Hs = 3.53 ft
Wave Length (λ) = 70.34 ft
Hmax = 6.35 ft

Maximum Wave Height: *(least of the following)*
Hmax = 6.35 ft
Hmax ≤ 0.65*ds = 3.58 ft
Hmax ≤ λ/7.0 = 10.05 ft

Therefore Hmax = 3.58 ft

○ **Resulting Storm Wave Properties:** *(see Fig)*
Tp = 5.33 sec

Hmax =	3.58	ft
Wave Length (λ) =	70.34	ft
η max =	2.50	ft

■ **<u>Maximum Quasi-Static Vertical Force and Associated Forces and Moments:</u>** *(AASHTO Sec 6.1.2.2)*

● **<u>Maximum Quasi-Static Vertical Force:</u>** *(AASHTO Sec 6.1.2.2.1)*

○ **<u>Determination of Fv-max parameters:</u>** *(eq 6.1.2.2.1-1)*

W_hat =	-50.09	ft				
W_hat / W =	-0.75		<	0.15	Therefore W_hat = 0.15*W	
W_hat =	-50.09	ft				
η max - Zc =	-1.83		<	db =	3.79	ft
β =	1.00					

x = 0.0508238

y = -0.712121

<u>For Slab Spans:</u> *(eq 6.1.2.2.1 b)*

b0 =	-0.147175	b4 =	-0.00066
b1 =	40.357917	b5 =	0.613995
b2 =	0.0397417	b6 =	31.43333
b3 =	-93.38042		

<u>Tapped Air Factor:</u>

A_air = 0.0114354

B_air = 0.1053956

(η max - Zc) / dg = -0.732333 < 1

%Air = variable

However, the bridge is not a girder type bridge
therefore:

%Air = 0

TAF = 1 > 1 (O.K.)

TAF = 1

<u>Quasi-Static Vertical Force:</u> *(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

● **<u>Associated Vertical Slamming Force:</u>** *(AASHTO Sec 6.1.2.2.2)*

B = 1.7118991

Zc / η max = 1.7316017 > 0

A = 0.0574009

<u>Vertical Slamming Force:</u> *(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

● **<u>Associated Horizontal Quasi-Static Wave Force:</u>** *(AASHTO Sec 6.1.2.2.3)*

*Note: Girders used on the New Makaha #3A Bridge are similar to 36 in Adjacent Box Girders)

<u>From Table 6.1.2.2.3-1:</u> *(for Box Girders)*

a0 =	-0.0304	a5 =	0.0025
a1 =	1.4247	a6 =	0.0403
a2 =	-1.1168	a7 =	0.5503
a3 =	0.3455	a8 =	-0.3612
a4 =	-0.048		

x = -0.33542

y = 0.0508238

Horizontal Quasi-Static Wave Force: *(eq 6.1.2.2.3)*

Fh-av = -0.129509 kip/ft < 0 *therefore set equal to zero*

Length of Bridge = 178.41 ft
Fh-av Total = 0.00 kips 0.00 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	ft
b_m =	-0.042158	ft
c_m =	-0.0049	ft
W' =	0.00	ft
W* =	67.00	ft

Associated Moment about Trailing Edge: *(eq 6.1.2.2.4-1)*

Mt-av = -0.48 (kip/ft)-ft

Length of Bridge = 178.41 ft
Mt-av Total = -84.88 kip-ft -42.44 tons-ft

● **Resulting Quasi-Static Vertical Force and Associated Forces and Moments:** *(AASHTO sec 6.1.2.2)*

Fv-max Total =	-864.23	kips	*(Quasi-Static Vertical Force)*
Fs Total =	0.05	kips	*(Vertical Slamming Force)*
Fh-av Total =	0.00	kips	*(Quasi-Static Horizontal Force)*
Mt-av =	-84.88	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)*

check = 10.12 < W = 67.00 ft Use eq 6.1.2.3.1-3 for omega

ω = 10.12 ft

Reference Horizontal Force: *(eq 6.1.2.3.1-2)*

F*h-max = 0.66 kip/ft

Horizontal Wave Force: *(eq 6.1.2.3.1-1)*

Fh-max = #NUM! kip/ft

Length of Bridge = 178.41 ft
Fh-max Total = #NUM! kip #NUM! 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 = -19.27 < W = 67.00 ft Use eq 6.1.2.3.2-3 for alpha

α = -19.27 ft

Reference Vertical Force: *(eq 6.1.2.3.2-2)*

F*v-ah = 2.26 kip/ft

Quasi-Static Vertical Wave Force: *(eq 6.1.2.3.2-1)*

Fv-ah = #NUM! kip/ft

Length of Bridge = 178.41 ft
Fv-ah Total = #NUM! kip #NUM! 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)*

Fs = 0.0002861 kip/ft

Length of Bridge =	178.41	ft		
Fs Total =	0.05	kips	0.03	tons

● **<u>Associated Moment About Trailing Edge:</u>** *(AASHTO Sec 6.1.2.3.4)*

<u>Reference Moment:</u> *(eq 6.1.2.3.4-2)*

M*t-ah = #NUM! (kip/ft)-ft

<u>Associated Moment about Trailing Edge:</u> *(eq 6.1.2.3.4-1)*

Mt-ah = #NUM! (kip/ft)-ft

Length of Bridge =	178.41	ft		
Mt-ah Total =	#NUM!	kip-ft	#NUM!	ton-ft

● **<u>Resulting Maximum Horizontal Wave Force and Associated Forces and Moments:</u>** *(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)*

■ **<u>Current Loads on Superstructure:</u>** *(AASHTO Sec 6.1.2.4)*

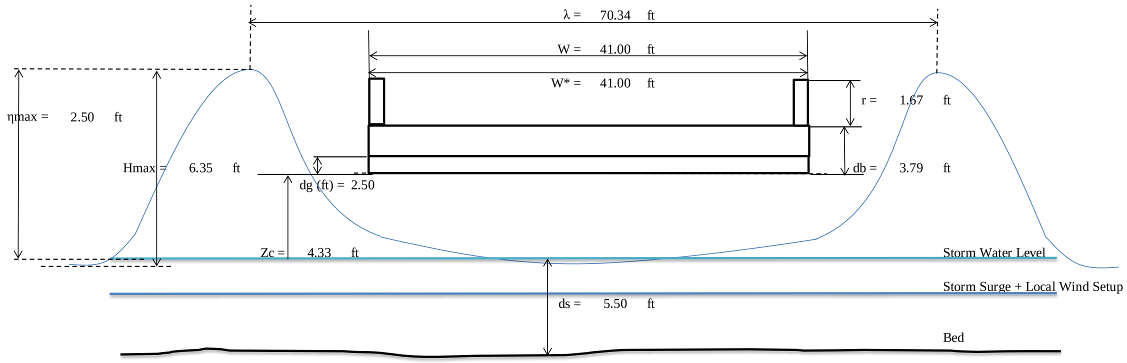
*Note: Current loads are not considered in this study.

Fhc = 0 kips

Nimitz Highway at Aloha Tower: Slip Cover #3
Method For Estimating Wave Forces on Bridge Superstructures

AASHTO (2008)
Constant Coefficients:

50 year wind speed =	105	mph	*(from AASHTO Fig 6.2.2.2-1 b)*
specific weight water =	0.064	kip/cubic ft	
g =	32.2	ft/sec^2	



♦ **Wave Calculations:**

• **Bridge Properties:**

Bridge Deck Width =	41.00	ft	Water Depth =	5.33	ft
Bridge Deck Length =	240.00	ft	Water surface to bot. of girder =	4.50	ft
Girder to Girder Width =	41.00	ft	Height of girder =	2.50	ft
Deck Thickness =	1.29	ft	Height of railing =	1.67	ft
Av =	9840.00	sq ft	Elevation to bot. of girder =	9.83	ft
N =	13	girders	Elevation to bot. of deck =	12.33	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)*
d =	6.88	ft	*(average water depth over fetch length)*
Fetch Length =	4767610.00	ft	
[(g*d)/(Ut*)^2] =	0.003		
Tp =	6.07	sec	

○ **Determination of time duration to develop fetch limited waves:**

t = 1947.76 sec < 3600 sec

U_1hr = 109.17 ft/sec To compute U_1hr use eq 6.2.2.4-4

Iteration Process: *(until 't' converges)*

First Iteration:

Ut = 110.32 ft/sec

Second Iteration:

Ut* =	175.42	ft/sec
[(g*d)/(Ut*)^2] =	0.007	
Tp =	5.35	sec
t =	2800.42	sec

Third Iteration:

Ut =	109.56	sec
Ut* =	173.92	ft/sec
[(g*d)/(Ut*)^2] =	0.007	
Tp =	5.33	sec
t =	2818.03	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

Fourth Iteration:

Ut =	109.55	sec
Ut* =	173.90	ft/sec
[(g*d)/(Ut*)^2] =	0.007	
Tp =	5.33	sec
t =	2818.29	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)

○ **Wave height and wavelength:**

Hs =	3.53	ft
Wave Length (λ) =	70.34	ft
Hmax =	6.35	ft

Maximum Wave Height: *(least of the following)*

Hmax =	6.35	ft
Hmax ≤ 0.65*ds =	3.58	ft
Hmax ≤ λ/7.0 =	10.05	ft
Therefore Hmax =	3.58	ft

○ **Resulting Storm Wave Properties:** *(see Fig)*

Tp = 5.33 sec

Hmax =	3.58	ft
Wave Length (λ) =	70.34	ft
η max =	2.50	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-max parameters:** *(eq 6.1.2.2.1-1)*

W_hat = -50.09 ft
W_hat / W = -1.22 < 0.15 Therefore W_hat = 0.15*W
W_hat = -50.09 ft

η max - Zc = -1.83 < db = 3.79 ft
β = 1.00

x = 0.0508238

y = -0.712121

For Slab Spans: *(eq 6.1.2.2.1 b)*

b0 =	-0.147175	b4 =	-0.00066
b1 =	40.357917	b5 =	0.613995
b2 =	0.0397417	b6 =	31.43333
b3 =	-93.38042		

Tapped Air Factor:

A_air = 0.0107478
B_air = 0.1284632

(η max - Zc) / dg = -0.732333 < 1
%Air = variable

However, the bridge is not a girder type bridge
therefore:
%Air = 0

TAF = 1 > 1 (O.K.)
TAF = 1

Quasi-Static Vertical Force: *(eq 6.1.2.2.1-1)*

Fv-max = -4.844062 kip/ft

Length of Bridge = 240.00 ft
Fv-max Total = -1162.57 kips -581.29 tons

● **Associated Vertical Slamming Force:** *(AASHTO Sec 6.1.2.2.2)*

B = 1.7118991

Zc / η max = 1.7316017 > 0
A = 0.0574009

Vertical Slamming Force: *(eq 6.1.2.2.2-1)*

Fs = 0.0002861 kip/ft

Length of Bridge = 240.00 ft
Fs Total = 0.07 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)

From Table 6.1.2.2.3-1: *(for Box Girders)*

a0 =	-0.0304	a5 =	0.0025
a1 =	1.4247	a6 =	0.0403
a2 =	-1.1168	a7 =	0.5503
a3 =	0.3455	a8 =	-0.3612
a4 =	-0.048		

x = -0.33542

y = 0.0508238

Horizontal Quasi-Static Wave Force: *(eq 6.1.2.2.3)*

Fh-av = -0.21334 kip/ft

Length of Bridge = 240.00 ft
Fh-av Total = -51.20 kips -25.60 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 ft
b_m = -0.042158 ft
c_m = -0.0049 ft
W' = 0.00 ft
W* = 41.00 ft

Associated Moment about Trailing Edge: *(eq 6.1.2.2.4-1)*

Mt-av = -0.76 (kip/ft)-ft

Length of Bridge = 240.00 ft
Mt-av Total = -183.42 kip-ft -91.71 tons-ft

● **Resulting Quasi-Static Vertical Force and Associated Forces and Moments:** *(AASHTO sec 6.1.2.2)*

Fv-max Total =	-1162.57	kips	*(Quasi-Static Vertical Force)*
Fs Total =	0.07	kips	*(Vertical Slamming Force)*
Fh-av Total =	-51.20	kips	*(Quasi-Static Horizontal Force)*
Mt-av =	-183.42	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)*

check = 28.74 < W = 41.00 ft Use eq 6.1.2.3.1-3 for omega

ω = 28.74 ft

Reference Horizontal Force: *(eq 6.1.2.3.1-2)*

F*h-max = 3.16 kip/ft

Horizontal Wave Force: *(eq 6.1.2.3.1-1)*

Fh-max = #NUM! kip/ft

Length of Bridge = 240.00 ft
Fh-max Total = #NUM! kip #NUM! 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 = -4.95 < W = 41.00 ft Use eq 6.1.2.3.2-3 for alpha

α = -4.95 ft

Reference Vertical Force: *(eq 6.1.2.3.2-2)*

F*v-ah = 0.58 kip/ft

Quasi-Static Vertical Wave Force: *(eq 6.1.2.3.2-1)*

Fv-ah = #NUM! kip/ft

Length of Bridge = 240.00 ft
Fv-ah Total = #NUM! kip #NUM! 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)*

Fs = 0.0002861 kip/ft

Length of Bridge = 240.00 ft
Fs 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*t-ah = #NUM! (kip/ft)-ft

Associated Moment about Trailing Edge: *(eq 6.1.2.3.4-1)*
Mt-ah = #NUM! (kip/ft)-ft

Length of Bridge = 240.00 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 =	0.07	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