

## **Technical Appendix D**

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**Balboa West Marina Expansion Project Coastal Engineering Study  
Everest International Consultants, Inc.  
July 2013**

# *Balboa West Marina Expansion Project*

## COASTAL ENGINEERING STUDY



Prepared for  
**URS Corporation**

Google Earth Pro  
Prepared by  
**Everest International Consultants, Inc.**

**July 2013**



**BALBOA WEST MARINA EXPANSION PROJECT  
COASTAL ENGINEERING STUDY**

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## **1. INTRODUCTION**

URS Corporation (URS) is assisting the Irvine Company in the development of the Balboa Marina West - an extension to an existing marina in the Lower Newport Bay, and has retained Everest International Consultants, Inc. (Everest) to provide coastal engineering support services for the proposed marina extension project. The coastal engineering study consists of two phases: Phase 1 for the evaluation of potential environmental concerns, and Phase 2 for the determination of design loadings for the selected marina extension alternative.

Phase 1 of the study was completed in March 2013. Based on the findings of the Phase 1 impact assessment, URS has selected the marina extension layout for the site. The selected marina layout and dock arrangement is shown in Figure 1.1. This report provides a summary of the approach and findings of the Phase 2 coastal engineering analyses conducted to evaluate wave and current loadings on the docks, boats and piles for the selected marina layout. The design wave conditions and the corresponding wave loadings are provided in Section 2 of this report; while the evaluation of the current conditions at the project site and the corresponding current loadings are described in Section 3.



## **2. WAVE LOADING**

### **2.1 Overview**

The purpose of the wave loading analyses is to estimate the horizontal wave induced forces and moments (where applicable) on the boats, docks, and piles of the proposed marina extension shown in Figure 1.1. A wave analysis was first conducted to estimate the wind wave and ship wake conditions at the proposed marina extension. The wind wave analyses utilized local wind data compiled from Balboa Pier to estimate the operational wind conditions while long term wind data compiled from John Wayne Airport were used to estimate the extreme wind conditions. Ship waves were estimated based on typical boats and operating conditions at the project location. Wave loadings on boats, docks and piles due to the larger of the wind waves and ship waves were then calculated using different methods and tide elevations.

### **2.2 Wave Analyses**

The project site is well sheltered by land and far away from the Newport Harbor entrance so no significant ocean swell is expected to penetrate to this location. Hence, the design wave conditions for the proposed marina are governed by local wind waves or ship wakes generated by passing ships.

#### **2.2.1 Wind Waves**

Analysis of wind waves starts with understanding the local wind patterns. Wind data are available from the nearby Balboa Pier for July, 2004 through January, 2013 (MesoWest 2013). However, a longer wind record would be better for the development of the extreme wind conditions for determining the design wave loadings. Nearby John Wayne Airport has 50 years of intermittent wind data from 1942 through 2012 (WeatherUnderground 2013). If the wind conditions at Balboa Pier and John Wayne Airport are similar, the longer record at John Wayne Airport can be used to establish the extreme wind conditions at the project site. Figure 2.1 shows the location of these two wind data sources relative to the project site.





Image: DeLorme Topo USA

**Figure 2.1 Balboa Pier and John Wayne Airport Wind Data Sources**

The wind roses developed based on the wind data from Balboa Pier and John Wayne Airport from 2004 through 2012 are shown in Figure 2.2. For both locations, the majority of the winds come from the southwest quadrant with speeds of less than 10 knots, with a small yet strong wind component from the north-northeast, representing the Santa Ana winds. Since the wind pattern at John Wayne Airport is similar to that at Balboa Pier, an extremal analysis was performed based on the 50-year wind data record collected at John Wayne Airport. The results are summarized in Table 2.1 below. The return period is defined as the average time interval between successive occurrences of an event being equaled or exceeded. For example, a wind speed with a 100-year return period can be expected to be exceeded, on average, once every 100 years.

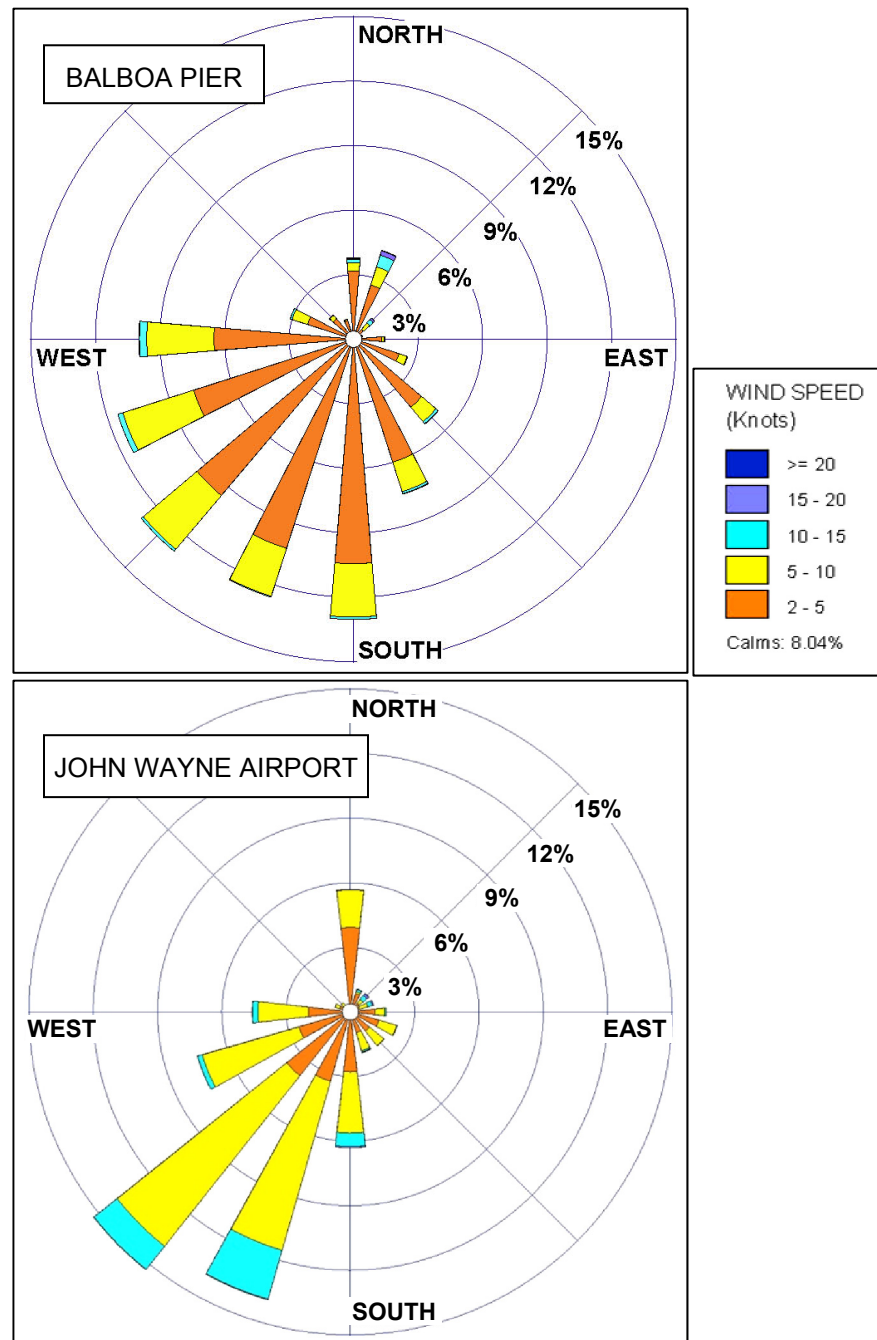


Figure 2.2 Wind Roses for Balboa Pier and John Wayne Airport

**Table 2.1 Extreme Wind Speeds at John Wayne Airport**

| RETURN PERIOD (YR) | 2-MINUTE WIND SPEED (KNOT) |
|--------------------|----------------------------|
| 2                  | 37.4                       |
| 5                  | 57.1                       |
| 10                 | 72.0                       |
| 25                 | 91.6                       |
| 50                 | 106.5                      |
| 100                | 121.4                      |

Extremely fast winds were found within the John Wayne wind record coming from all directions. Therefore, a conservative approach to estimate the extreme wind waves at the project site is to apply the fastest wind over the longest fetch leading to the project site, with the necessary adjustment of the wind duration appropriate for the fetch distance. This longest fetch is shown as a white arrow in Figure 2.3. The results are summarized in Table 2.2 below. In the table,  $H_{mo}$  is the energy based significant wave height,  $T_p$  is the peak wave period.

**Table 2.2 Extreme Wind Waves at Project Site**

| RETURN PERIOD (YR) | $H_{mo}$ (FT) | $T_p$ (SEC) |
|--------------------|---------------|-------------|
| 50                 | 3.4           | 3.2         |
| 100                | 4.0           | 3.5         |



Aerial Photo: Google Earth Pro

**Figure 2.3 Fetches for Wind Wave Analyses**

### 2.2.2 Ship Wakes

Ship wakes are vessel generated waves which propagate away from the sailing line of the vessel. Figure 2.4 shows a picture of ship wakes generated by a recreational boat. The probable ship wake at the project site would be governed by the types and dimensions of ships likely to pass by the docks; as well as how fast they are traveling and their distance from the docks. Hence, a range of vessel types and sizes were used in estimating ship wakes at the project site. The dimensions of the vessels were estimated from aerial photographs of Newport Harbor, with the maximum length assumed to be equal to the maximum vessel length shown in the proposed dock layout shown in Figure 1.1. The speed limit in Newport Harbor is 5 miles per hour (4.4 knot), but for this analysis it was assumed that, at times, some vessels may exceed the legal limit.

Table 2.3 summarizes the vessel properties and conditions, calculation methods, and resulting wave conditions at the project site. The table shows the maximum wave height,  $H_{max}$ , and the associated wave period,  $T$ , at the project site. As shown in the table, with the largest boats in Newport Harbor exceeding the posted speed limit, the resulting wave heights and periods at the project site are still smaller than those of the extreme wind waves shown in

Table 2.2. Hence, wind waves are the controlling short period wave conditions for calculating the wave loading at the docks, boats, and piles for the proposed marina extension.

**Table 2.3 Ship Wake Input, Calculation Method, and Resulted Wave Conditions**

| SCENARIO DESCRIPTION            | BOAT LENGTH (FT) | SPEED (KNOT) | METHOD        | H <sub>MAX</sub> (FT) | T (SEC) |
|---------------------------------|------------------|--------------|---------------|-----------------------|---------|
| Sportboat - planning            | 20               | 20           | Bhowmik       | 0.3                   | n/a     |
| Sportboat - subcritical         | 20               | 5            | Kriebel       | 0.2                   | 1.4     |
| Sportboat - critical            | 20               | 9            | Kriebel       | 3.5                   | 2.4     |
| Superyacht - posted speed limit | 100              | 4.4          | Kriebel       | 0.7                   | 1.2     |
| Superyacht - speeding           | 100              | 8            | Kriebel       | 0.6                   | 2.2     |
| Superyacht - posted speed limit | 100              | 4.4          | Gates Herbich | 0.5                   | 1.2     |
| Superyacht - speeding           | 100              | 9            | Gates Herbich | 1.9                   | 2.4     |
| Superyacht - speeding           | 100              | 10           | PIANC 1987    | 2.9                   | 2.7     |



**Figure 2.4 Ship Waves from a Recreational Boat**

## 2.3 Wave Loading

The purpose of the wave loading analyses was to estimate the horizontal wave induced forces and moments (where applicable) on the docks, boats and piles at the project site. Each structure (i.e., dock, boat, or pile) requires a different calculation method and hence is discussed separately below.

As mentioned earlier, extreme wind waves are higher than ship wakes and hence were used in calculating the wave loadings. For rigid structures it is common to use a design wave height equal to the highest 1% of the waves, which is calculated as 1.67 times the significant wave height. The marina basin depth was assumed to be -10 feet, MLLW, and wave loadings were evaluated for two tide elevations - Mean Higher High Water (MHHW) and the Mean Lower Low Water (MLLW). It was assumed that waves approach the dock, boat, or pile normally, a condition that would produce the highest loading for the same wave.

### 2.3.1 Docks

For calculation of wave forces on docks, the dock face was assumed to be 32 inches high. Wave forces on the vertical side of a dock were estimated using multiple methods. The resulting wave forces based on each of the three methods are of the same order but slightly different. As expected, all the three methods show that the wave forces increase with the design wave heights (i.e. wave force is higher for the 100-year condition compared to the 50-year condition). However, the methods are not consistent in the effect of tide elevations on the wave forces. One method shows that the wave force is slightly higher for MHHW tide compared with MLLW tide, while another method shows the opposite. Instead of simply picking the largest wave force for each return period as a recommended conservative design wave force for each tide elevation, one recommended design wave force is estimated as the combined average of the top results irrespective of tide elevation, i.e. there is only one recommended design wave force for each return period. The recommended wave force per unit length of dock for the 50 and 100-year wind wave is shown in Table 2.4.

**Table 2.4 Wave Forces on Docks**

| <b>RETURN PERIOD (YR)</b> | <b>WAVE FORCE<br/>PER UNIT LENGTH OF DOCK (LB/FT)</b> |
|---------------------------|---|
| 50                        | 520   |
| 100                       | 617   |

An example application of this force is provided. For a 40-ft long section of dock, the 100-year, maximum horizontal wave force would be 40 ft x 617 lb/ft = 24,680 lbs. This force should be applied at the elevation of the dock connection to the pile for calculating the moment on the pile.

### 2.3.2 Boats

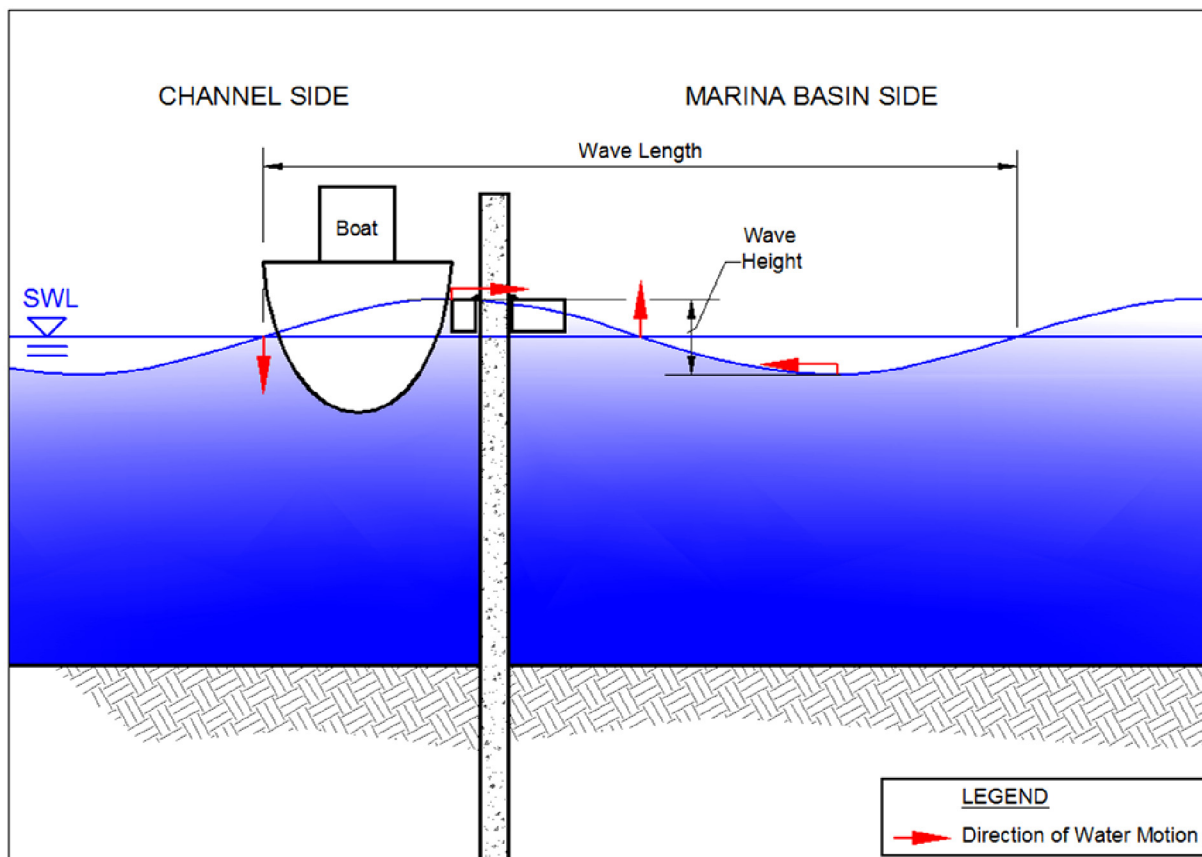
Horizontal wave forces on vertical sides of the boats were calculated using the same methods for calculating wave loadings on the docks described above for both MLLW and MHHW conditions and the 50-year and 100-year wind waves. Wave loadings were calculated for three different categories of vessel sizes: 20 ft, 60 ft and 120 ft in length. Maximum boat drafts were assumed to be limited to one foot less than the water depth during MLLW condition and boat freeboard was assumed to be 5 feet. Table 2.5 summarizes the resulting wave force per unit length of boat in pounds per foot (lb/ft).

**Table 2.5 Wave Forces on Boats**

| WIND WAVE<br>RETURN PERIOD | BOAT LENGTH (FT)        | 20                                     | 60   | 120  |
|----------------------------|-------------------------|--|------|------|
|                            | DESIGN DEPTH (FT, MLLW) | -6                                     | -10  | -12  |
|                            | DRAFT (FT)              | 5                                      | 9    | 11   |
|                            | WATER LEVEL             | WAVE FORCE/UNIT LENGTH OF BOAT (LB/FT) |      |      |
| 50-YEAR                    | MLLW                    | 1200                                   | 1620 | 1853 |
|                            | MHHW                    | 1135                                   | 1335 | 1480 |
| 100-YEAR                   | MLLW                    | 1523                                   | 2123 | 2445 |
|                            | MHHW                    | 1350                                   | 1740 | 1900 |

An example application of this force is provided. For a 60-ft long boat, the 100-Year, maximum horizontal wave force during a MHHW tide would be 104,400 lbs (60 ft x 1,740 lb/ft).

The wave loads on boats are de-coupled from wave loads on docks due to phase shifts between the two. This concept about the wave phase differences is illustrated in Figure 2.5. As the wave passes the boat and dock, while the wave is at its crest at the dock (exerting maximum horizontal force), the wave motion is down on the channel side of the boat (exerting negligible horizontal force).

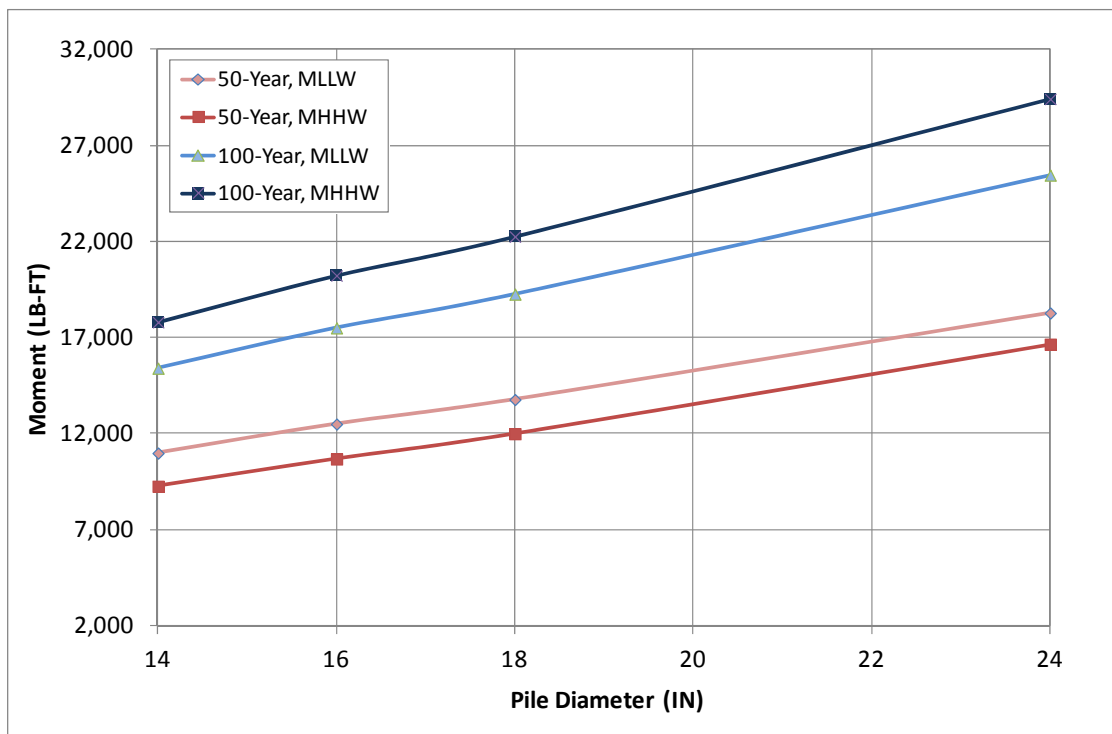
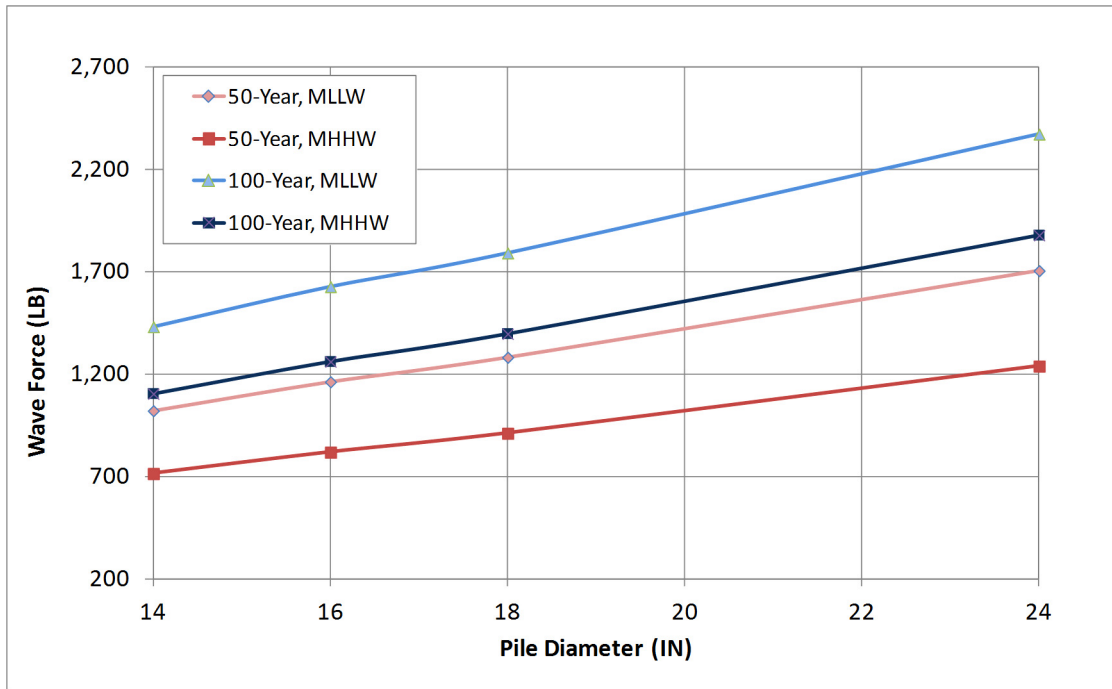


**Figure 2.5 Phase Lag in a Cross Section of Wave Passing a Docked Boat**

### 2.3.3 Piles

A nonlinear wave theory described in the *Coastal Engineering Manual* (USACE 2011) was used to calculate wave forces and moments per exposed pile above the un-scoured mud line. As provided in design drawings, the mud line is to be dredged to -10 feet, MLLW. Figure 2.6 shows the wave forces (pound) and moments (pound-feet) calculated for concrete piles with diameters ranging from 14 to 24 inches. A range of pile diameters were provided so that the marina engineer can choose the most appropriate results for their purposes. The forces shown in the figure represent the maximum possible combination of inertial and drag forces for a single vertical pile. The forces and moments presented here do not include forces on docks or boats which should be added to the pile forces and moments as needed.





**Figure 2.6 Wave-Induced Forces and Moments on Piles**

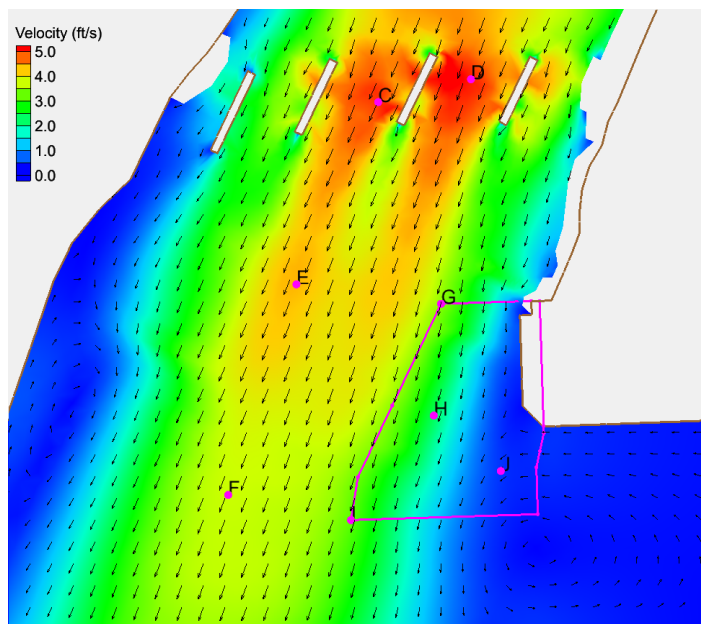
### 3. CURRENT LOADING

#### 3.1 Overview

The purpose of the current loading analyses is to estimate the horizontal current induced forces on the boats, docks, and piles of the proposed marina extension. A two-dimensional (2D) hydrodynamic model of Newport Bay was used to simulate currents in the project vicinity for the following two conditions: (i) during a 100-year flood event, and (ii) during a Tsunami event. The higher current of the two events was then used as the design current for calculating the current induced forces on boats, docks and piles. Potential change in the current induced forces during the 100-year flood event due to the presence of debris was also considered.

#### 3.2 100-Year Flood Currents

In evaluating potential impacts of the proposed marina extension during Phase 1 of the coastal engineering analyses (Everest, 2013), a two-dimensional (2D) hydrodynamic model of Newport Bay was used to simulate the currents at the project vicinity during a 100-year flood event that captures all major stormwater discharges into Upper Newport Bay. The same model was used here to determine the possible maximum flood currents at the project site by running the model with various scenarios regarding the timing of the arrival of the peak of the 100-year storm flow at different phases of the tides. It was found that maximum currents at the project location occurs if the peak of the flood flow arrives when the tide is ebbing and the water elevation is at approximately 2.8 ft (MLLW). The velocity distribution under this condition is shown in Figure 3.1. The currents at the project site vary with location with the maximum currents of approximately 4.6 ft/sec occurring along the outer edge of the project boundary.



**Figure 3.1 Velocity Distributions Near the Project Location during 100-Year Flood**

### 3.3 Tsunami Current

The State of California Seismic Safety Commission report (2005) *The Tsunami Threat of California* states that the Cascadia subduction zone will produce the State's largest tsunami. Moffatt & Nichol (2007) estimated that a Cascadia tsunami of magnitude 9.2 would have a dominant period of 50 minutes and resulted in maximum wave amplitudes of 0.9 ft and 1.1 ft at the Angel's Gate of Port of Los Angeles and Queen's Gate of Port of Long Beach, respectively. The Cascadia tsunami conditions offshore of Newport Harbor are expected to be different than at the Ports of Los Angeles and Long Beach than. However, without site specific information for Newport Harbor, the conditions at Port of Los Angeles and Port of Long Beach are used for estimating the likely Tsunami current at the project site. The same 2-D hydrodynamic model that was used for the impact analyses (Everest 2013) was used to simulate Tsunami current at the project location with a 1 ft wave amplitude and 50 minute period at the offshore boundary of the model. The model simulated currents at the project location is approximately 2.9 ft/sec.

### 3.4 Current Loadings

Since the current at the project location during a 100-year flood event (4.6 ft/sec) is higher than the tsunami event (2.9 ft/sec), it is used as the design current for calculating the current loadings on the dock, boat and piles.

#### Current Loads on Docks

Horizontal pressure from water currents is represented by the following equation (PIANC, 1994; Gaythwaite, 1990):

$$P = C_d \frac{\gamma}{2g} u^2$$

Where,

$C_d$  is the drag coefficient, ranging from 0.6 to 1.0 (Tobiasson & Kollmeyer, 1991), and 1.0 was used to be conservative;

$\gamma$  is the specific weight of water;

$g$  is the acceleration of gravity; and

$u$  is the surface current speed, a factor 1.14 was applied to convert the depth averaged current speed from the model to a surface speed (Gupta, 1989).

Based on the design current of 4.6 ft/sec, the pressure on the dock will be 26 lb/ft<sup>2</sup>. For the 32 inch dock planned to be used for the marina extension, the current force per linear feet of the dock face will be 42 lb/ft, assuming 60% of the dock face is submerged.

### Current Loads on Boats

For estimating current loads on boats, the same method for docks was used but the depth averaged water velocity was used instead of the surface velocity since the boat has normally has a deeper draft subjected more to the average velocity. For the design current of 4.6 ft/sec during ebb tide with water elevation at 2.8 ft MLLW, the pressure on the boat will be 20.2 lb/ft<sup>2</sup>.

### Current Loads on Piles

Current loads on submerged portions of piles were calculated similar to those calculated for boats using the depth averaged current speeds, but with a drag coefficient  $C_d$  of 0.6 for smooth round or octagonal piles. With the design current of 4.6 ft/sec during ebb tide with water elevation at 2.8 ft MLLW, the pressure on the pile will be 12.1 lb/ft<sup>2</sup>. Current loads on 16-inch, 18-inch and 20-inch diameter piles at different areas with dredged depths at -6, -10, and -12 ft MLLW were calculated and summarized in Table 3.1 below.

**Table 3.1 Current Loads on Piles**

| DESIGN DEPTH (FT MLLW) | FORCE (LB)    |               |               |
|------------------------|---------------|---------------|---------------|
|                        | 16" DIA. PILE | 18" DIA. PILE | 20" DIA. PILE |
| -6                     | 142           | 160           | 178           |
| -10                    | 207           | 232           | 258           |
| -12                    | 239           | 269           | 299           |

### **3.5 Impacts of Debris to Docks**

During a flood event, stormwater runoff from San Diego Creek and other channels upstream of the project location will carry debris with the freshwater flow. Based on the photographs provided by the City of Newport Beach, the debris mainly consists of green waste of small tree branches and sticks. The City of Newport Beach regularly deploys debris bloom just upstream of the project location to prevent the debris from entering the lower Newport Bay. Figure 3.2 shows a picture of the debris trapped behind a debris bloom during past deployment.



Photo courtesy: Chris Miller, City of Newport Beach Harbor Resources Division

**Figure 3.2 Illustration of a Debris Mat Trapped behind the Proposed Dock**

The docks for the proposed marina extension will act like a debris bloom collecting debris behind it during a rain event if there is no deployment of the debris bloom upstream of the project location. The debris collecting behind the dock will result in greater current loadings on the dock. This increased current loading during a 100-year flood event was estimated by the shear stress on a debris mat based on a method originally developed for ice flows (Gaythwaite, 1990):

$$F = C_d U^2 A_d$$

Where

$F$  is the drag force on the debris mat;

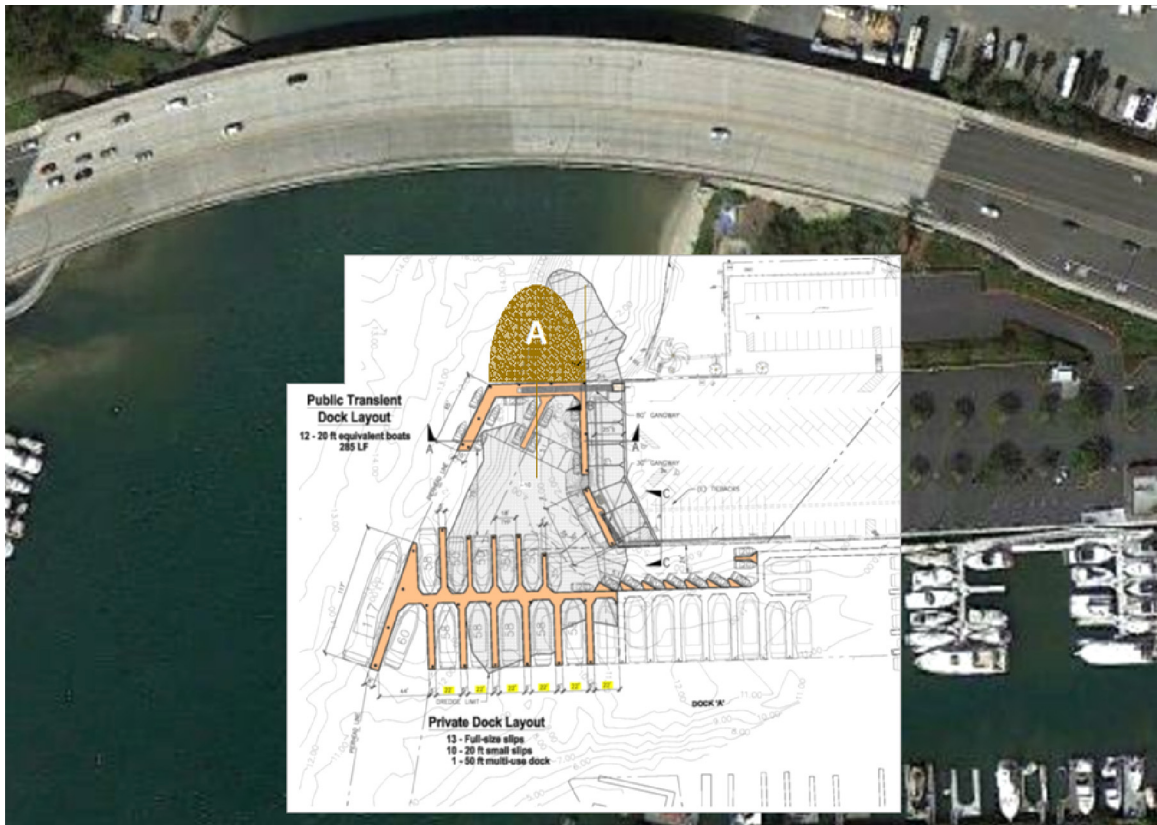
$C_d$  is current driven debris shear stress drag coefficient;

$U$  is near surface current speed; and

$A_d$  is the surface area of debris mat.

Assuming the debris mat extends upstream a distance equal to the dock length in the shape of half an ellipse as illustrated in Figure 3.3, the debris area  $A_d$  is approximately 5,700 ft<sup>2</sup>.

Assuming a drag coefficient of 0.1, under the 100-year flood conditions, the force of the debris mat on the 32-in dock will be approximately 180 pound per linear feet (lb/ft).



Aerial Photo: Google Earth Pro; Drawing: URS

**Figure 3.3 Illustration of a Debris Mat Trapped behind the Proposed Dock**

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