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TSUNAMI RESILIENCE OF PILE-SUPPORTED WHARVES: A FIRST PRINCIPLES APPROACH

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Mots-clés: Ports, Tsunami, Quais, Nouvelle-Zélande, Modélisation structurelle

1 INTRODUCTION

New Zealand’s position along the Pacific ‘Ring of Fire’ exposes it to a variety of destructive natural hazards including earthquakes, volcanoes, and tsunami. Tsunami have become a particular point of focus in recent years as a result of large and damaging events such as the 2004 Indian Ocean Tsunami, the 2011 Tohoku Tsunami, and the 2018 Palu Tsunami. While these large events have not borne great impacts on New Zealand’s coastline, New Zealand does have an extensive history of tsunami hazards including more than 40 documented events over the past 165 years. Though most of these have been insignificant in scale, damaging tsunami have occurred that would bear greater consequences today with the continuing development and growth of coastal settlements.

Ports are one category of infrastructure that presents an immediate risk from tsunami impacts as a result of their inherent position along the coast. In New Zealand, ports facilitate domestic and international shipping that most major industries rely on and have been declared a critical lifeline by the Civil Defense and Emergency Management Act of 2002 due to their role in receiving and processing aid shipments following disasters. Ports entail a wide variety of infrastructure components in terms of functionality and structural characteristics but one of the most fundamental components is wharves, damage to which would compromise the operability of the entire port. This research sought to provide a comprehensive evaluation of the structural impacts on New Zealand wharves from tsunami. As the dominant wharf typology in New Zealand is open, reinforced concrete, and pile-supported these characteristics were central in the evaluation. As no existing studies have provided a similarly comprehensive assessment, a first principles approach was adopted in which information about New Zealand wharves was compiled, tsunami load characteristics were derived, site-specific propagation models were developed in order to determine associated hazard parameters, and computational structural wharf models were analysed against the loading characteristics.

New Zealand’s international and domestic inter and intra-island shipping markets are dominated by 13 major commercial ports, the names and locations of which are shown in Figure 1. Figure 2 provides the value of cargo passing through each of New Zealand’s main ports for the 2015-2016 financial year (Statistics NZ 2016) which gives a comparative view of the size and criticality and Table 1 provides further details of those ports including the principle industries serviced by them and a broad summary of the facilities present in each.
Figure 1: Locations of the 13 major New Zealand ports on the North Island (left) and South Island (right)

Figure 2: Value of imports and exports passing through each New Zealand port in the 2015-2016 financial year
<table>
<thead>
<tr>
<th>Port Name</th>
<th>Description of Facilities and Services</th>
</tr>
</thead>
<tbody>
<tr>
<td>Northport</td>
<td>Deepwater port located on Marsden Point, near Whangarei. Most critical function is servicing shipping related to the Marsden Point Oil Refinery which is a critical New Zealand facility. Also facilitates forestry exports from a variety of plants and imports of gypsum, coal, and fertilizer, mostly for the Golden Bay Cement Plant nearby.</td>
</tr>
<tr>
<td>Ports of Auckland</td>
<td>Dominant import seaport in New Zealand featuring the largest container terminal in the country. Facilities large-scale vehicle and machinery imports among numerous other goods. Contains storage for a variety of dry and liquid goods and multiple deepwater berths. Rivalled in size by only Port of Tauranga.</td>
</tr>
<tr>
<td>Port of Tauranga</td>
<td>Only New Zealand port to rival Ports of Auckland in size. Primarily export facility servicing logs, petroleum, cement, agriculture, forestry, dairy, etc. Comprises large container terminal, bulk dry goods and liquid storage, petroleum storage facilities, a coal yard, cold storage and other facilities. Wharves are located on both the Mount Maunganui and Sulphur Point sides of Tauranga Harbour.</td>
</tr>
<tr>
<td>Port Taranaki</td>
<td>Contains a variety of bulk dry goods and liquids storage facilities. Major shipping facility for petrochemical industry with nearby Taranaki Gas Fields and Methane (methanol) Plant among others.</td>
</tr>
<tr>
<td>Eastland Port</td>
<td>Small port comparative to others servicing primarily export of forestry products.</td>
</tr>
<tr>
<td>Port of Napier</td>
<td>Medium-sized North Island port servicing mostly forestry exports but also agriculture and other products. Contains large-scale log storage facilities and a container terminal.</td>
</tr>
<tr>
<td>Centreport</td>
<td>North Island point of departure for interisland ferry services. Contains key fuel terminal, Holcim Cement facilities, and large container terminal. Services a variety imports and exports including forestry and agriculture.</td>
</tr>
<tr>
<td>Port of Marlborough</td>
<td>Comprises two main wharves: Waikawa in Shakespeare Bay which is a deepwater berth facilitating mostly bulk dry goods and forestry exports and Waiotahi in Picton Harbour. Port of Marlborough is the South Island berthing facility for the interisland ferries and as such a critical sea-based connection for the islands. Also facilitates a number of cruise ships throughout the year.</td>
</tr>
<tr>
<td>Port Nelson</td>
<td>Main goods produced in the region which are exported via Port Nelson include fish, forestry, and apples. Port contains a variety of storage facilities, a small container terminal, and several berths.</td>
</tr>
<tr>
<td>Lyttelton Port</td>
<td>Liquid bulk storage facilities that are employed by several large oil companies. Coal yard servicing most west coast coal mines. Largest South Island container terminal. Facilitates imports and exports including forestry products, gypsum, coal, cement, fish, and agriculture. Contains the South Island’s only dry dock.</td>
</tr>
<tr>
<td>Primexport</td>
<td>Comprises facilities to store bulk dry goods, bulk liquids, petroleum, and logs. Container terminal on-site which is run by Tamarak Container Terminal Limited, a subsidiary of Port of Tauranga. Services nearby Holcim Cement Plant. Primary exports include fish, dairy, forestry, and timber.</td>
</tr>
<tr>
<td>Port of Otago</td>
<td>Comprises facilities at two locations: Port Chalmers, and Dunedin. Key imports and exports include forestry, meat, and dairy. Storage facilities are present for bulk dry goods and liquors as well as a small container terminal.</td>
</tr>
<tr>
<td>Southport</td>
<td>Maintains wharf servicing Tiwai Point Aluminium Smelter and Town Wharf servicing mostly petroleum imports. Eight other berths lie on the island harbour. Hosts Stewart Island Ferry. Facilities include bulk dry goods and liquid storage and a small container terminal.</td>
</tr>
</tbody>
</table>

Table 1: Major New Zealand ports, their key industries, and facilities present

## 2 PROPAGATION MODELLING

Previous studies have investigated either generalized tsunami hazards along New Zealand coastlines [Power, 2013] or the specific tsunami hazards for a limited number of ports [Borrero & Goring, 2015a; Borrero et al., 2015b]. Power reported potential tsunami heights for all of New Zealand but based those findings on the results of propagation models over 20-kilometre lengths of the coastline. The maximum amplitude reported for a particular region is the maximum within that 20-kilometre span which likely occurs outside of the port boundaries for those segments in which they lie. The national tsunami hazard assessed in this report is shown in Figure 3.
Though informative, the aforementioned analyses were insufficient for providing a nationwide tsunami hazard assessment for the ports, hence the need for site specific propagation modelling. Given the intent of this research, each of the sites denoted in Figure 1 was modelled. Tsunami layers at these sites were developed using the Community Modelling Interface for Tsunami or ComMIT, a tool which produces water level and current speed estimates for tsunami based on a database of precomputed unit source models which can be augmented and combined to generate unique tsunami sources. Two types of information are required to generate tsunami models through ComMIT; detailed bathymetric maps and a seismic source model.

For this study, bathymetric grids were developed for each of the port locations evaluated based primarily on topographic and bathymetric data from the Land Information New Zealand (LINZ) Data Service. In some locations, additional LiDAR data was available on a one metre scale from local councils which was generally sufficient for producing the requisite regional topography. ComMIT utilises a system of three nested grids to balance propagation model results and runtime. For this study, each location consisted of a large bathymetric grid with a 500-metre spacing between nodes, a middle grid with 150-metre spacing, and a fine grid with ten metre spacing. Thus, for the finest grid immediately surrounding the port, water level and current speed data was calculated at a ten metre resolution.

Defining a seismic source model requires defining the location and dimensions of the slip plane and either the slip distance or moment magnitude of the earthquake. To provide a comprehensive overview of tsunami hazards, several moment magnitude earthquakes were chosen and applied to both local and distant subduction zones for each port. For local source tsunami, earthquake moment magnitudes ranged from $M_w 7.0-9.0$ at increments of 0.5 while for distant source scenarios the magnitudes ranged from $M_w 8.0-9.5$ at the same increments. In both cases, the lower end of these ranges represented the point below which tsunami were negligible within the ports while the upper end represented the upper limit of earthquakes that the respective subduction zones are expected to produce. The magnitude of the earthquake can be approximately connected to the size of the slip plane by applying an average stress drop across Pacific Ocean subduction zone segments and the equations derived by Abe (1975). In defining source locations, one local and one distant scenario location was chosen for each port based on Power's (2013) source analysis which represented the local and distant subduction zones that Power contends are most likely to produce tsunami at each location. Figures 4 summarise the source locations used for each port.
The result of each propagation model was a water level and current speed grid for each time step of each analysis. In order to determine the local maxima, the maximum water level and current speed at each node in the grid was isolated and plotted. Sample maximum water level and current speed plots for Northport are shown in Figure 5.

From these water level and current speed grids developed for each unique scenario, the ultimate maxima were extracted and are summarised via the plots in Figures 6 and 7, thus indicating the potential hazards associated with each port.
3 **TSUNAMI LOADING**

Tsunami loads are generally broken down into a number of fundamental components which can be categorised as either lateral or vertical loads. These two fundamental categories of tsunami loading both comprise a state of high-magnitude initial impact and subsequent, sustained hydrodynamic loading. Note that the primary hazard metric against which most equations evaluate loading is bore height though some also incorporate velocity components.

Quantification of lateral tsunami loading has received a greater research focus compared to vertical loading, as vertical loading is relevant only to certain types of infrastructure. Additionally, the load magnitudes associated with lateral loading are generally greater though the comparative potential for resulting damage depends on the nature of each component of the structure. Due to the relative lack of
information regarding vertical loading, particularly uplift, in current standards, the load magnitudes for horizontal components were derived from design standards whereas the uplift loading was derived from experimental studies.

### 3.1 Load Magnitudes

A number of design standards assign approximations to various tsunami loading components. Among those standards investigated include the Coastal Construction Manual [FEMA, 2011], City and County of Honolulu Building Code [CCH, 2000], and Guidelines for the Design of Structures for Vertical Evacuation for Tsunami [FEMA, 2012]. Each of these publications provides quantifications for lateral hydrodynamic loads and wave slam which are summarised in Table 2.

<table>
<thead>
<tr>
<th>Wave Slam ($F_s$)</th>
<th>CCH 2000</th>
<th>FEMA 2011</th>
<th>FEMA 2012</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$F_S = 4.5\rho gh^2w$</td>
<td>$F_S = \rho g C_d h w$</td>
<td>$F_S = 1.5 * F_D$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Hydrodynamic Load ($F_D$)</th>
<th>CCH 2000</th>
<th>FEMA 2011</th>
<th>FEMA 2012</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_D = \frac{\rho C_D A u^2}{2}$</td>
<td>$F_D = \frac{\rho C_D A u^2}{2}$</td>
<td>$F_D = \frac{1}{2} \rho C_D w (h u^2)_{max}$</td>
<td></td>
</tr>
</tbody>
</table>

*Table 2: Wave slam and hydrodynamic load equations presented in various design standards*

In the above equations, $\rho$ represents fluid density, $g$ is the acceleration of gravity, $h$ is the bore height, $w$ is the width of the inundated face, $C_s$ is the slam coefficient, $C_D$ is a drag coefficient, $A$ is the area of the submerged surface, and $u$ is the flow velocity.

Chen et al. (2016) provides the primary source for uplift load approximations. The experimental studies produced time series for uplift loading on a scale wharf model set on a variable artificial slope which clearly illustrated initial impact and steady state phases. A number of bore heights were tested relative to the height of the wharf deck which could be scaled to the heights applied in this research. Chen’s final equations for impact and steady state pressures normalised against equivalent hydrostatic loading are given in Equations 1 and 2:

$$\frac{\rho g}{\rho g h_b} = C_3 - \frac{h_d}{h_b}$$  \hspace{1cm} Eq. 1

$$\frac{\rho f}{\rho g h_b} = (\cot \theta + C_2) \ln \left[h_b \left(\frac{1}{2} F_{rb}^2 + 1 - \frac{1}{2} k_1 F_{rb}^2 \right)\right]$$  \hspace{1cm} Eq. 2

where $\rho g$ is the steady state loading, $\rho f$ is the impact loading, $h_b$ is the tsunami bore height, $h_d$ is the deck height, $\theta$ is the slope on which the wharf rests, $k_1$ is an energy loss coefficient, $C_2$ and $C_3$ are coefficients, and $F_{rb}$ is the Froude number.

### 3.2 Tsunami Bore Heights

As this was intended to be a representative study, the range of bore heights tested against the models is consistent with anticipated wave heights throughout New Zealand based on the propagation models. Thus the wave heights tested in this study range from a minimum of two metres to a maximum of ten metres. Tsunami below two metres resulted in insignificant damage to the wharf models and thus two metres was chosen as a minimum.

### 4 STRUCTURAL MODELLING

The structural modelling component of this research sought to apply the previously discussed tsunami loading characteristics to wharf models representative of existing New Zealand infrastructure in order to compute resulting damage states. Because ports in New Zealand are competitive, private entities, no attempt was made to recreate specific wharves from specific ports but rather a generalised approach was adopted. The primary source of information in developing wharf models was construction drawings and reports obtained from each of the respective ports and from Archives New Zealand. Once these sources
were obtained, geometric, material, geotechnical, and structural characteristics were collated and used to produce generalised models based on typical or recurring features.

Structural wharf models used in the damage state analysis were developed in OpenSees. A two-dimensional model was used in this analysis, as the form of an open-style wharf repeats itself (in bays) along the shoreline. By assuming propagation of the tsunami bore in line with each bay of the wharf, a single line of piles perpendicular to the shoreline was modelled. A two-dimensional model only allows for examination of stresses and application of loads horizontally and vertically, but not out of plane. Figure 8 depicts the generic wharf model employed in this research with some key details highlighted which will be discussed in more detail in the following sections.

4.1 Geometric Characteristics

The base structural model represents an open-style, pile supported wharf. While solid fill wharves are present in limited number, the overwhelming majority of New Zealand wharves are open-style. In addition to consistent use of open structures, many dimensions and geometric characteristics are relatively constant over time, hence the use of a single base model with variable properties. The geometry of each wharf model consisted of six reinforced concrete piles, a pile cap, and a deck. The piles are embedded into a shore slope at a 45-degree angle.

Figure 8: Details of representative wharf model employed in this research with dimensions and details of elements, connections, and foundation model

4.2 Material Models

Wharf structures in New Zealand consist primarily of reinforced concrete, particularly at largeports. Details of the existing concrete and steel material properties in New Zealand wharves are difficult to obtain and not available in much of the port documentation. Thus estimates were relied upon by examining historical design from the era of construction of various wharves when specific details were not available.

Minimum concrete compressive strength has increased periodically in New Zealand concrete structures design standards and in practice. Port documentation suggests that wharf structures from the earliest eras were designed with concrete based on compressive strengths of between 25.7 and 38.6 MPa. This compressive strength is consistent with the current minimum of 25.7MPa and exceeds those of even the most recent revisions to the concrete structure design standards [NZS, 2006]. As a conservative estimate, 25.7 MPa was generally applied to wharf models for older structures and 30 MPa was implemented for more recent structures. Additional considerations were made for the long-term aging trends of concrete which apply to most structures here. NZSEE (2006) utilised a series of studies [Priestley, 1995; Park 1996; Presland, 1999] on the 30-year strength of specimens taken from various reinforced concrete structures to recommend a conservative value of 1.5 times the 28-day compressive strength be adopted for the purposes of modelling aged concrete.
The New Zealand steel reinforcement design standard has been updated several times, including in 1963, 1975, and 1989. Infrastructure built prior to 1962 is more difficult to assess, though it has been suggested that grade 227 steel is generally a reasonable assumption [NZSEE, 2006]. In 1962, the specified minimum yield strength for steel reinforcement was 227 MPa [NZS, 1962] but this was increased to 275 MPa in an amendment to the same standard. A further modification was introduced in 1964 which introduced grade 414 steel for construction that would be required to withstand greater seismic loads [NZS, 1964]. Since that time, two grades of steel have been consistently specified in New Zealand steel reinforcement design standards. From 1975 until 1989, the two steels in use were grades 275 and 380 [NZS, 1975] and from 1989 until 2001 were 300 and 430 [NZS, 1989]. Since 2001, the New Zealand steel reinforcement design standard has specified two grades, 300 and 500 [AS/NZS, 2001]. In sitetests, recorded yield strengths have generally been higher than their specified values [Chapman, 1991]. NZSEE (2006) suggests that for the purposes of modelling, a 1.08 modification factor should be applied to lower characteristic values. Rather than modelling transverse reinforcement explicitly, a confined concrete model was adopted to represent the augmented concrete strength inside the hoops [Mander et al., 1988].

OpenSEES features several concrete and steel materials that may be applied to structural models. For the purposes of this research, the chosen concrete model was Popovics Concrete Model [Popovics, 1973] while steel reinforcement was represented by a Dodd-Restrepo material model [Dodd & Restrepo-Posada, 1995].

### 4.3 Section Properties

Pile section properties, which collectively consist of materials, rebar layout and dimensions, and shape and dimensions of the section, vary greatly throughout New Zealand presently and historically. All of these characteristics have changed overtime and while there has been great variation at any particular time, some designs have proven more representative of certain eras. Design variations have occurred for many reasons though one governing factor is advancement in seismic design and knowledge of New Zealand’s seismicity. Five pile section types were ultimately tested: one representative section for the period prior to 1963 and two sections each for the periods 1963 to 1989 and 1989 to 2001, one for low seismicity regions and one for high seismicity regions. Furthermore, both square and circular piles are common and so variations of both section shapes were developed, though pile caps were always represented as square elements.

### 4.4 Soil Model

Data from geotechnical site investigations was collated from nearly every major port in New Zealand to characterise typical soils encountered. Results suggested significant variation in soil properties between ports and within ports as well. To provide a simple representation of the effect of different soil profiles on the performance of wharves, four single-layered soil profiles were defined to represent loose sand, dense sand, soft clay, and stiff clay. Properties of each of the soil types were defined based on the site investigation data and are summarised in Table 3.

<table>
<thead>
<tr>
<th></th>
<th>Loose Sand</th>
<th>Dense Sand</th>
<th>Soft Clay</th>
<th>Stiff Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Saturated Unit Weight (kN/m³)</td>
<td>1.75</td>
<td>2.15</td>
<td>1.4</td>
<td>2.0</td>
</tr>
<tr>
<td>Angle of Internal Friction</td>
<td>30</td>
<td>45</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Undrained Shear Strength (kPa)</td>
<td>-</td>
<td>-</td>
<td>50</td>
<td>150</td>
</tr>
</tbody>
</table>

*Table 3: Summary of soil properties for simplified soil profiles*

Given the variation in soil types implemented in the models, methods for deriving the soil characteristics for both cohesive and cohesionless soils were adopted. The characteristics of the soil surrounding each pile were modelled using a spring bed approach. A P-y soil model approach was adopted to represent the lateral soil response, and was based either on the Matlock (1970) curve for clay or the API (2003) curve for sand. T-z and Q-z models were used to characterise the vertical soil response, representing side friction of the pile and soil bearing at the end of the pile respectively. Both are defined in a similar manner to that

4.5 Structural Representations

Piles, pile caps, and deck members were modelled using a force (nonlinear) beam-column. These types of beam-columns consider the spread of plasticity throughout the entire element via a predefined number of integration points along the element's length rather than concentrating deformations at the member's ends.

Many wharves contain extended strand moment resisting connections where reinforcing strands extend from the pile and into the pile cap or deck. This connection type was implemented as a series of zero length members with assigned material characteristics equivalent to the reinforcing steel. Since the steel is the material providing the connection and the majority of elevated tension would be carried by steel, this was deemed an appropriate representation.

4.6 Tsunami Load Application

The wharf structures comprise several elements connected by nodes. At each node, a tsunami load time series was applied that consisted of both an initial peak impact and a sustained steady-state load, seen in Figure 9 (a) for the lead pile and Figure 9 (b) for each subsequent pile, the timing of which are based on the load profiles from Chen (2016). The same time series was applied to the whole structure but was staggered to begin at variable time steps to account for the travel time to each node, based on the theoretical velocity of the tsunami and the distance between the piles. As both the lateral and vertical loads consist of impact and steady-state phases, similar procedures were used in their developments.

4.7 Structural Modelling Outputs

Damage in the wharf elements arose primarily as a result of a combination of uplift on the deck and large bending moments in the piles causing excessive axial tension. Resulting damage states were derived for each wharf model by collating and examining the stresses and strains at each integration point in each element and isolating the maximum values.

1. Elastic – An element falling within the elastic state has not surpassed the elastic phase on the stress-strain response at any point in its cross section.
2. Cracked – Applies to elements where the tensile stress in the concrete has exceeded the modulus of rupture, thus resulting in tension cracking.
3. Yield – Applies to elements where the internal tensile stress in the reinforcing bar exceeds the yield strength.
4. Fracture – Applies to elements where tensile stress in the reinforcement exceeds the ultimate tensile strength (UTS) and corresponding strain, causing the steel to fracture.
Each element in each unique wharf model was assigned the damage state corresponding with the maximum stress and strain in the section under the applied loading, as in Figure 10.

Figure 10: Sample plot showing element damage states for wharf

Following the derivation of damage states on an element level, comprehensive damage states were chosen to describe the damage to the whole structure. These damage states are intended to provide an illustration of the structural damage to each unique wharf typology under each tsunami loading case but do not necessarily indicate where that damage occurs. The damage states used to assess the structure were as follows:

1. Concrete Cracks – Similar to the cracking element damage state but indicates simply that cracking occurred somewhere in the structure.
2. Pile Yielding – Indicates that reinforcing steel has yielded somewhere in the structure.
3. Pile Fracture – At the associated wave height, steel has fractured somewhere in the structure.
4. Severe Damage, Partial Fracture in Half of Piles – Indicates at least three piles have experienced partial fracture. Partial fracture was defined as fracture of some reinforcing strands but not the entire pile.
5. Severe Damage, Complete Fracture in Half of Piles – Indicates at least three piles have completely fractured, thus greatly reducing the bearing capacity of the structure.

These comprehensive damage states were assigned to each unique modelling scenario and the results are presented in Figures 11-14.
Figure 11: Comprehensive wharf damage states for loose sand soil model scenarios

Figure 12: Comprehensive wharf damage states for dense sand soil model scenarios
Most inputs to the models had great impact on the performance of the structures. The era tested and corresponding material characteristics had the greatest isolated impact on performance, as indicated by the great variation in resulting damage states when all other inputs are constant. High strength steel performs much better than low strength and given the augmented strength of steel in later eras, newer wharves are expected to perform better. Soil types also have a large impact with the cohesionless soils generally performing better than the cohesive soils. Finally, square piles performed superior to circular piles in all equivalent cases.
Based on observations of the element damage states, significant damage was spread throughout much of the wharf structures, particularly under larger wave heights. However, the most significant damage was consistently present in two locations where there were high bending moment concentrations: immediately beneath the pile-pile cap connections and along the ground surface, as shown in Figure 15. The concentrations near the top of the pile are generally larger and more prone to result in fracture.

![Figure 15: Locations of consistent high stresses in wharf models](image)

5 CONCLUSIONS

This research has presented a first principles approach to assessing tsunami impacts on pile-supported wharves. Given the lack of information on each component necessary to produce these impact models, three distinct phases of work were pursued: propagation modelling within the major New Zealand port environments, determination of tsunami loading characteristics, and development of computational structural models to represent generalised New Zealand wharves. The outputs of such efforts were a set of maximum water level and current speed data for each major New Zealand port under a variety of tsunami inundation scenarios and damage state information for various wharf typologies under tsunami ranging from two to ten metres in bore height. In general, New Zealand’s east coast is far more susceptible to tsunami inundation and the damage that would result. Though no existing wharves were modelled explicitly due to commercial sensitivity, the generalised models give an indication regarding the damage that may arise under those tsunami inundation scenarios. The wharf constructions, material characteristics, and geotechnical characteristics play a large role in the performance of the wharves as evidenced by large variations in that performance under identical loading scenarios. The primary consistency in the structural modelling is that damage occurs frequently at the top of the pile near the connection to the pile cap and where the pile meets the ground level.

6 ACKNOWLEDGEMENTS

This research comprises the bulk of my PhD work at the University of Auckland. I am grateful to my supervisor Liam Wotherspoon, co-supervisor Asaad Shamseldin, and effective co-supervisor Bruce Melville for their hard work and guidance. I am thankful for the funders of this work which included the University of Auckland and the New Zealand natural Hazards Research Platform. My current employer, the National Institute of Water and Atmospheric Research (NIWA) has been extremely kind in providing me with time to write and support in composing this paper. Finally, a great thank you to Ron Cox from the University of New South Wales and Michael de Vos from the Port of Napier for making me aware of and inviting me to participate in this opportunity.
7 REFERENCES

- NZS (1964): “NZS 1879 – Hot Rolled Steel Bars of HY60 Grade (60,000 psi) for Reinforced Concrete”, Standards Council of New Zealand.
New Zealand’s is a Pacific Island nation which experiences impacts from a wide variety of natural hazards including tsunami. While large-scale, destructive tsunami have not recently been generated in the vicinity of New Zealand, there is a history of large local and distant source tsunami occurring and resulting in significant runup heights along various coastlines. Although tsunami have the potential to penetrate inland, the first and most severe damage will occur in coastal cities and communities. Among that infrastructure that is most exposed to damage from tsunami is ports. Given the criticality of ports to the economic and social well-being of New Zealand, this research sought to assess potential tsunami impacts on existing New Zealand port infrastructure via a first-principles approach consisting of three steps. First, tsunami propagation models were developed for each of New Zealand’s 13 major ports. Second, tsunami loading characteristics were derived from a variety of design standards and experimental modelling. Finally, structural wharf models were developed to represent New Zealand infrastructure and tested against the tsunami loads to calculate predictive damage states. The results of this work included both detailed tsunami hazard information for the respective ports and the generalised damage states that could be linked back to specific ports.

RESUME

La Nouvelle-Zélande est une nation insulaire du Pacifique qui subit les effets d'une grande variété de risques naturels, dont les tsunamis. Bien que des tsunamis destructeurs à grande échelle n’aient pas été générés récemment à proximité de la Nouvelle-Zélande, des tsunamis d’origine locale ou lointaine se sont produits par le passé et ont entraîné des hauteurs de run-up importantes le long de diverses côtes. Bien que les tsunamis puissent pénétrer à l’intérieur des terres, les premiers dégâts, et les plus graves, se produisent dans les villes et les communautés côtières. Les ports font partie des infrastructures les plus exposées aux dommages causés par les tsunamis. Compte tenu de la criticité des ports pour le bien-être économique et social de la Nouvelle-Zélande, cette recherche a cherché à évaluer les impacts potentiels des tsunamis sur les infrastructures portuaires néo-zélandaises existantes par le biais d’une approche de principe en trois étapes. Premièrement, des modèles de propagation des tsunamis ont été développés pour chacun des 13 principaux ports de Nouvelle-Zélande. Ensuite, les caractéristiques de charge des tsunamis ont été dérivées d’une variété de normes de conception et de modélisation expérimentale. Enfin, des modèles de quais structurels ont été développés pour représenter l’infrastructure néo-zélandaise et testés par rapport aux charges du tsunami pour calculer les états de dommages prédictifs. Les résultats de ce travail comprenaient à la fois des informations détaillées sur les risques de tsunami pour les différents ports et des états de dommages généralisés pouvant être reliés à des ports spécifiques.
ZUSAMMENFASSUNG


RESUMEN

Nueva Zelanda es una nación insular del Pacífico que experimenta el impacto de una gran variedad de riesgos naturales, entre ellos los tsunamis. Aunque recientemente no se han generado tsunamis destructivos a gran escala en las proximidades de Nueva Zelanda, existen antecedentes de grandes tsunamis de origen local y lejano que han provocado alturas de escorrentía significativas a lo largo de varias costas. Aunque los tsunamis tienen el potencial de penetrar tierra adentro, los primeros y más graves daños se producirán en las ciudades y comunidades costeras. Entre las infraestructuras más expuestas a los daños de los tsunamis se encuentran los puertos. Dada la importancia de los puertos para el bienestar económico y social de Nueva Zelanda, esta investigación trató de evaluar los posibles impactos de los tsunamis en la infraestructura portuaria existente de Nueva Zelanda mediante un enfoque de primeros principios que consta de tres pasos. En primer lugar, se desarrollaron modelos de propagación de tsunamis para cada uno de los 13 puertos principales de Nueva Zelanda. En segundo lugar, se derivaron las características de carga de los tsunamis a partir de una serie de normas de diseño y modelos experimentales. Por último, se desarrollaron modelos estructurales de muelles para representar la infraestructura neozelandesa y se probaron con las cargas de los tsunamis para calcular los estados de daños predictivos. Los resultados de este trabajo incluyeron tanto información detallada sobre el peligro de tsunami para los respectivos puertos como los estados de daños generalizados que podían vincularse a puertos específicos.
INTRODUCTION

Coastal waters experience growing pressures that will get aggravated under climate change and further economic development. This applies particularly to areas near commercial harbours and large coastal cities, where the number of conflicting uses for the emerged and submerged coastal domains require urgent decisions under present weather and planning under future climate scenarios. The effective management of these coastal waters and beaches requires quantitative criteria for decision making, such as can be provided by field observations combined with high-resolution numerical simulations. Such combination allows an objective estimation of risk levels for management and decision making in coastal activities such as commercial navigation, entrance routes, small craft navigation, water sports, beach tourism and aquaculture deployment. These activities would be better managed within a quantitative risk framework, where decisions are based on advanced hydro-morphodynamic predictions with explicit error intervals. Available models, however, do not incorporate bathymetric evolution to provide an evolving and accurate forecasting of the state of coastal waters. Such hydro-morphodynamic calculations, also applicable to estimate breakwater overtopping or dredging hazards, are not usually accompanied by error intervals derived from field data, which hampers the introduction of such predictions into the working routines of coastal and harbour authorities, nor into the criteria for a variety of end users already mentioned. The proposed modelling suite, originally prepared for this work as part of a thesis, combined with field in-situ and remote observations should improve coastal decision making based on objective criteria and with explicit error intervals that evolve with weather and coastal morphodynamic state.
Figure 1: Location and detailed view of ‘La Arena’ beach in Muskiz village, on the Gulf of Biscay, North coast of Spain. Red arrow points north.

3 OBJECTIVES AND STATE OF THE ART

Most coastal management decisions are taken reactively without benefiting from present meteo-oceanographic operational predictions and field data. This results in hard to rank uncertainties (from wind fields, turbulence closures, bathymetric data, etc.) and large (often implicit) hydro-morphodynamic errors. Moreover, there is a lack of robust indexes to characterise coastal environments under present and projected climates. Numerical modelling results do not feature the accuracy and reliability required by end-users, which limits the use of advanced meteo-oceanographic information, despite the buoy networks and video monitoring techniques progressively deployed in many countries. As a consequence, present models characterise hydro-morphodynamic patterns with significant levels of uncertainty due to the variety of co-existing spatial and temporal scales (e.g. from wind waves of a few seconds to meso-scale circulation of a few weeks). To solve this complex scenario, simplifications have been required to set up a robust modelling suite both applicable to micro and macro tidal scenarios, where the latter present a stronger and easier to model signal but also the complexity of an active beach profile evolving with tidal level, which further complicates the numerical discretisation requirements.

This work aims to demonstrate how present numerical tools, strategically combined and calibrated with field data can support proactive decisions for harbour and coastal engineering and management. The proposed tool can be introduced into an early warning system, using the original indicators derived in this work for risk based decision making in coastal applications. The developed numerical suite has been applied and validated at a coastal embayment subject to macro tidal ranges and energetic wave storms typical of the Bay of Biscay (Cantabrian Sea). The combined numerical-observational development consists of the following blocks:

i) Coupled hydro-morphodynamic modelling suite, incorporating a regular bathymetric updating
ii) Explicit error intervals based on innovative error metrics after a thorough validation with field data
iii) Original user-oriented indicators to support engineering and management decisions
iv) Preparation of the model-data combination for a pre-operational Early Warning System (EWS)

4 COUPLED MODELLING AND VALIDATION

The developed modelling suite is based on the SWAN wave model for deep to intermediate water depths in a large domain with a regular mesh size of 100 m resolution. This outer model feeds a high resolution coupled hydro-morphodynamic model (XBEACH code) where the computational domain encompasses the harbour and adjacent beaches, with a curvilinear variable mesh size of 20 m in the offshore zone and 5 m in the nearshore zone. The wave model SWAN considers refraction, numerical diffraction and wind-current interactions. The hydro-morphodynamic XBEACH model calculates coupled wave-current fields, sediment fluxes and resulting topo-bathymetry. All codes have been successfully calibrated in stand-alone mode before preparing the coupled version. The field data for such validation comprises met-ocean data from the coastal buoy network of Puertos del Estado, video recordings of the surf zone and emerged beach from the KostaSystem (Azti-Tecnalia) and regular topo-bathymetric surveys for the studied area. Once the reliability of single models was ensured, an efficient iterative calculus scheme has been developed, applying regular buoy data and video recorded images for an explicit error estimation (Figure 2). The so obtained hydro-morphodynamics have allowed deriving robust indexes to characterise waves, currents or morphodynamic response according to the requirements of final end users (harbour or beach authorities, risk management or tourist operators).

![Figure 2: Proposed methodological scheme for the developed modelling suite and applications](image)

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Figure 3: Validation scheme for the hydro-morphodynamic modelling suite and derived application indexes

The error assessment has been conducted sequentially, beginning with the wave data and ending with the bathymetric evolution (Figure 3). Simulations start from a baseline seabed geometry and after conducting an initial sensitivity analysis it has been decided to update morphodynamics during storm growth, peak and decay. The higher the resolution the more frequent will be the morphodynamic updating, increasing computational costs and opening the possibility of numerical error accumulation. The methodology can be easily exported and implemented in harbours and beaches without excessive instrumentation, providing results directly applicable to analyse problems that have actually occurred (e.g., impact of a past storm) or to support decisions that avoid future problems (e.g., when implementing the proposed models within an operational forecasting system). The developed tool can also be used to optimise field measurements and to control harbour environmental impacts, with error levels well below those in state-of-art models that do not consider hydro-morphodynamic coupling.

5  WAVE MODELLING

5.1  Equations and Boundary Conditions

The selected wave model, SWAN, is particularly well suited for intermediate water depths. The main governing equation is given by the wave action (N) conservation law:

$$ \frac{\partial N}{\partial t} + \Delta_x \left[ (c_g \cdot \vec{u}) \cdot N \right] + \frac{\partial c_g N}{\partial \sigma} + \frac{\partial c_g N}{\partial \theta} = S_{tot} $$  

Where N represents the spectral energy divided by wave frequency (σ). N evolves with time/space, with the wave energy propagation velocity (c_g) the current velocity field (\vec{u}), frequency and propagation direction. To facilitate practical applications a time and depth averaged current field has been considered. With the same target, the implemented boundary conditions to minimise poor behaviour near the domain borders are:

i)  Offshore boundary conditions [Dirichlet type], from field observations to prescribe wave conditions at deep water contours

ii) Lateral radiation boundary conditions [Neumann type], allowing incoming waves and tides and enabling outward going inner reflections
iii) Shore boundary conditions [Neumann type], absorbing sponge layers for waves arriving to the shoreline

The selected numerical discretisation looks for

i) A smooth deep to shallow water transition for a robust model nesting
ii) Limited computational costs to prepare the code for an operational early warning system
iii) Numerical accuracy in coastal regions with sharp gradients (bathymetry or shoreline)
iv) Numerical stability for longer computations based on enhanced physical-numerical consistency

These conditions are all required for the selected application case where the shore irregularity particularly near the beginning of the main breakwater presents a tough challenge for numerical simulations. Rocky shoals with an irregular lateral boundary and the discharge from the ‘Barbadún’ river would not be robustly simulated otherwise. The numerical grid for the offshore domain (Figure 4) covers enough area to guarantee a stable wind wave generation and propagation. The offshore wave boundary condition comes from the Bilbao Vizcaya wave buoy, kindly provided by Puertos del Estado.

Figure 4: Computational domain and mesh for the large (deep water) domain to simulate wave evolution. The colouresscale represents depth ranging from 0 (emerged) to -70 m (submerged).

The selected wave storm is a modal event, typical of this coastal area. It stated on 12 September 2017, with an average wave direction of 300° (waves coming approximately from the NW) and an average wind velocity about 10 m/s. Tidal range was between 2.5 and 4.5 m generating tidal currents above 0.5 m/s which must be considered for coastal navigation and sediment transport. Tidal evolution has been introduced through a mean sea level variation and consistent boundary velocities, using the long wave approximation.

5.2 Simulations and Error Metrics

The wave field has been simulated for periods longer than one week to provide statistically stable indexes for harbour and coastal decision making. The outer boundary condition for the hydrodynamic domain (SWAN) has been defined by a Jonswap spectrum suited for deep to intermediate depths. The nested hydrodynamic module, fed by SWAN results, considers wave-current combined bed friction using the Madsen’s formulation\(^\text{10}\). Wave simulations have been compared to buoy registered data for various parameter combinations (Table 1).

Table 1: Summary of the parameter settings used in the wave simulations

<table>
<thead>
<tr>
<th>Simulation</th>
<th>Jonswap γ [-]</th>
<th>Period [s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>γ = 3.3</td>
<td>Peak period</td>
</tr>
<tr>
<td>B</td>
<td>γ = 3.3</td>
<td>Mean period</td>
</tr>
<tr>
<td>C</td>
<td>γ = 2.0</td>
<td>Mean period</td>
</tr>
<tr>
<td>D</td>
<td>γ = 1.5</td>
<td>Mean period</td>
</tr>
<tr>
<td>E</td>
<td>γ = 1.0 (Pierson-Moskowitz)</td>
<td>Mean period</td>
</tr>
</tbody>
</table>

The results show an overprediction when selecting the peak period and a medium to narrow width spectrum (γ=2 or γ=3.3). The best fit and smaller error intervals correspond to a wider spectrum with γ=1.5 and the average zero crossing period as input period. With such a parameter selection the significant wave height error is reduced from 0.46 m (case A) down to 0.21 m (case D). Such 45%-error reduction proves the need to analyse the more suitable parameter combination for achieving error levels acceptable by harbour or coastal users. The visual inspection of the resulting wave fields for cases A and D (Figure 5) does not allow such a discrimination which again underlines the need for quantitative error assessments as shown in Figure 6 in terms of wave height and period. The error interval is in general within 10% for wave height and period while for directions the overall differences were within a 20º sector, acceptable for most coastal and harbour applications.

Figure 5: Simulated wave fields for case A (left) and case D (right) of Table 1. The colour scale represents the significant wave height field (m).

Figure 6: Measured and simulated significant wave height and average period time series displaying error levels.
Error metrics have included bias and normalised root mean square for storm, calm and the full set of wave conditions. A sample of these metrics for the five cases in Table 1 appears in Figure 7, which shows an optimal bias for wave height and period for the parameter settings of case D. These error intervals, comparable to other analyses from the state of the art\textsuperscript{11} result in point-wise uncertainties smaller than 0.20 m for the significant wave height, 1 s for the averaged period and 5° for the peak wave direction, within the commonly accepted accuracy range for harbour and coastal engineering and management.

Figure 7: Evolution error (Bias) for cases A to D (see Table 1)

6 HYDRO-MORPHODYNAMIC COUPLING

6.1 Equations and Boundary Conditions

The inner domain equations are based on the Xbeach\textsuperscript{12} code, simulating coupled wave-current fields and resulting morphodynamic evolution. The wave module now considers the dissipation due to the breaking wave roller in the energy balance equation:

\[
\frac{\partial N}{\partial t} + \Delta_x \left[ (c_w u + \vec{u}) \cdot \nabla N \right] + \frac{\partial c_w N}{\partial x} + \frac{\partial c_w N}{\partial \theta} = \frac{S_{tot}}{\sigma}
\]  

(1)

The wave energy density for breaking waves with a roller ($S_r$) is a function of spatial coordinates, time and wave direction. $c_w$, $c_e$ and $c_d$ represent the wave energy propagation velocity along the coordinate axes ($x$, $y$) and the directional axis ($\theta$). The energy dissipation within the roller\textsuperscript{13} has been estimated as a product of the phase speed and the roller stress per unit area ($r$) and can be written as:

\[
D_r = c \cdot r = \frac{\rho_w g A_r \sin(\beta_{roller})}{L_T}
\]  

(3)

The circulation governing equations are the mass and momentum conservations laws, including as drivers the gradients of radiation stresses\textsuperscript{14} and with a turbulence closure where the eddy viscosity coefficient, $v_h$, depends on the roller energy dissipation ($D_r$):

\[
v_h = h \left( \frac{D_r}{\rho_w} \right)^{1/3} + v_{h,back}
\]  

(4)

Where $h$ is the water depth and $v_{h,back}$ is the background turbulence level, leading to values around 0.15 m$^2$/s for spilling or plunging breakers.

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\textsuperscript{13} Roelvink et al. (2009).

\textsuperscript{14} Deigaard (1993).

\textsuperscript{14} S. -Arcilla and Lemos (1991).
Sediment transport has been modelled using a number of formulations where the one by Van Rijn has been selected since it provides a better fit to available observations. The equilibrium sediment concentration \(C_{eq}\) has been obtained as:

\[
C_{eq} = \frac{A_{sb}}{h} \left( \left( u^{2} + 0.64 u_{rms}^{2} - u_{cr}^{2} \right)^{1.5} + \frac{A_{sz}}{h} \left( \left( u^{2} + 0.64 u_{rms}^{2} - u_{cr}^{2} \right)^{2.4} \right) \right)
\]

Where \(A_{sb}\) and \(A_{sz}\) are the suspension and bed coefficients and \(u_{rms}\) represents the near orbital velocity including turbulence effects. To improve morphodynamic predictions a threshold for sediment motion has been included to avoid the ‘noise’ of low energy hydrodynamic conditions. The selected threshold is a function of the critical Shields parameter \(\theta_{cr}\) which depends on the sediment diameter \(D_{s}\) as:

\[
\theta_{cr} = \frac{0.3}{1 + 1.2D_{s}} + 0.55 \left[ 1 - \exp(-0.02D_{s}) \right]
\]

When \(\theta < \theta_{cr}\), there is no sediment transport while \(\theta_{cr} < \theta < 0.8\) corresponds to sediment transport associated to bedform migration. For \(\theta < 0.8\) the sediment flux corresponds to sheet flow.

The resulting bathymetry is obtained from sediment mass conservation, expressed as:

\[
\frac{\partial z_{b}}{\partial t} + \frac{f_{mor}}{1 - p} \left( \frac{\partial q_{x}}{\partial x} + \frac{\partial q_{y}}{\partial y} \right) = 0
\]

Where \(z_{b}\) is the sand level, \(f_{mor}\) is a morphodynamic acceleration factor that depends on the morphodynamic updating strategy and \((q_{x}, q_{y})\) are the total sand transport fluxes, calculated as:

\[
q_{x}(x, y, t) = h C \, U^{e} - D_{h} h \frac{\partial C}{\partial x} - f_{slope} |U^{e}| h \frac{\partial z_{b}}{\partial x}
\]

Where \(f_{mor}\) is a correction factor to improve the fit to observations. The resulting set of fit parameters requires some expert knowledge and only by combining simulations and observations it is possible to select the best parameter combination and to make explicit the error intervals and their propagation from deep to shallow water from the hydrodynamic model to the morphodynamic model.

The sediment mass conservation law has been discretised with an upwind scheme and including and avalanche correction, as a function of the equilibrium concentration \(C_{eq}\). This can be expressed as:

\[
\frac{\partial hc}{\partial t} + \Delta \tilde{S}_{b} = \frac{hc_{eq} - hc}{T_{y}}
\]

Where \(c\) is the depth averaged concentration, \(C_{eq}\) is the equilibrium concentration, \(T_{y}\) is a reference time scale depending on water depth and sediment fall velocity and \(\Delta \tilde{S}_{b}\) are sedimentary fluxes.

### 6.2 Simulations and Error Metrics

The morphodynamic domain includes the submerged and emerged coastal sectors, to support an integral management of the coastal domain, discretized with a variable curvilinear mesh with a resolution of 20 m for the deep area and 5 m for the shallow and emerged area, as shown in Figure 8.

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16 Reniers et al. (2004).
17 Soulsby, R. et al. (1997).
Figure 8: High-resolution shallow water domain for hydro-morphodynamic coupled simulations. The colour scale represents depth ranging from +20 (emerged) to -40 m (submerged).

The threshold of movement for sand has been imposed considering the actual sediment granulometry, provided by Azti-Tecnalia and featuring relatively homogeneous diameters with $D_{50}=0.2$ mm. The initial set of simulated cases did not consider bathymetric evolution, leading to larger errors in the sea bed predictions. The runs without calibration showed unrealistic erosion and sedimentation patterns (Figure 9, left) when using default values for all parameters. This led to sediment accumulations reaching +4.0 m during the studied storm, which were significantly reduced by a local fit of hydro-morphodynamic parameters (Figure 9, right).

Figure 9: Erosion – accretion patterns during the storm peak for the run with default parameters (left) and after calibration using local data (right), showing the maximum differences during the calibrations process. The colour scale represents erosion (negative values) and accretion (positive values) in metres.
The overestimation in erosion/sedimentation patterns was observed in areas with significant gradients, more frequent in the eastern part of the beach. The transport formulation\(^\text{19}\) that provided the best fit to observations was selected for all subsequent runs (Figure 9, right).

\[ BSS = 1 - \frac{(Z_{b,c} - Z_{b,m})^2}{(Z_{b,0} - Z_{b,m})^2} \] (10)

Where \(Z_{b,c}\) represents the modeled bathymetry, \(Z_{b,m}\) the measured bathymetry and \(Z_{b,0}\) the initial bathymetry. A perfect fit (measurements fully agree with simulations) is given by \(BSS = 1\) while it good fit (high quality simulation) is given by a \(BSS\) between 0.35 and 0.4\(^\text{21}\). The area with \(BSS \geq 0.35\) (Figure 10 in yellow) is limited by -6 m depth and +2 m elevation. Outside this area, the adjustment remains reasonable, although less accurate. The quality of morphodynamic simulations has been checked with topo-bathymetric images provided by Azti-Tecnalia. The average \(BSS\) in the yellow shaded area for the simulation with calibrated parameters was 0.7178, remarking the high-quality results of the developed modelling suite.

7 HARBOUR AND COASTAL ENGINEERING AND MANAGEMENT

7.1 Applications for Beach-Harbour Interactions

The developed modelling suite predicts hydro-morphodynamic fields for the emerged and submerged coastal domains, with explicit error intervals which tend to grow above the end users’ preferred thresholds when using abathymetry that is not regularly updated. Here bathymetric updating is based on the calculated bed evolution, validated with surveys and images from video cameras. The bathymetric evolution derived from the Timex images from Azti-Tecnalia and the numerical simulations appear in Figures 11 and 12. The sea-bed evolution is typical from a dissipative morphodynamic state, featuring shoreline erosion accompanied by sediment deposition in deeper waters within the active profile.


The complex breaker bar deposition is clearly appreciable from the white foam pattern in the Timex images and is qualitatively similar to the one obtained from the numerical simulations, with morphodynamic complexity decreasing for increasing depth and distance to the rock shoals. Erosive areas are predicted on the Eastern side of the beach due to the wave concentration over the submerged shoals while deposition occurs on the submerged beach, starting around the -2.0 m isobath and going down to the -6.0 m depth.

Figure 11: Video recorded Timex images (KostaSYSTEM from Azti-Tecnalia) before (10a) and after (10b) the wave storm

Regular bathymetric updating (about every 1 to 2 days) reduces bar formation and the intensity of erosion and deposition, improving the qualitative agreement with observations. The difference in predicted morphodynamic evolution, with and without bathymetric updating, can reach 0.5 m for this beach and under the analysed typical storm. Based on the performed simulations the best strategy for bathymetric updating would be in the middle of the storm growth interval, right after the peak of the storm and in the middle of the decay stage of the storm.

Figure 12: Accretion/erosion patterns for the studied embayment (in meters) without (left panel) and with (right panel) bathymetric updating. The colour scale represents erosion (negative values) and accretion (positive values) in metres.

7.2 Applications for Navigation and Dredging

The calculated wave and current fields can be directly applied in support of dredging and navigation decisions, providing an objective estimation of hazard levels. The proposed index for dredging (D) is a
function of the significant wave height \((H_D)\), the ship draft \((d)\) and the water depth under the ship keel \((D_{keel})\), written as:

\[
D(x, y) = f \left( \frac{H_s}{d \cdot D_{keel}}, x, y \right) = \gamma_1 \frac{H_s}{d}(x, y) + \gamma_2 \frac{H_s}{D_{keel}}(x, y)
\]  

(11)

Where the \(D\) index has been obtained as a linear combination of 2 wave- and ship-based dimensionless variables, related by two parameters, \(\gamma_W\) of order 1 and \(\gamma_s\) of order 5, for bringing the two terms to the same order of magnitude. Squat effects have not been considered assuming the dredger is stationary. The resulting hazard levels appear in Figure 13, where red indicates an important risk level and green a safe dredging or navigation. The figure shows growing hazards for deeper waters, indicating that the first term of equation 11 dominates. This is because the water depth was in all cases large enough and because the index does not incorporate the risk areas near the shore at both sides of the estuary. The wave limit in deep water has been set as 1.5 m, corresponding to a TSHD2000 dredger.

![Figure 13: Plot of D index characterizing dredging hazards for the studied coastal sea and estuary. The D index has not been estimated for the nearshore subdomain, representing only the deep water and intermediate zone hazard levels.](image)

The index proposed for navigation, \(N\), is a function of the wave and current energy levels and wave steepness. The proposed index can be written as a linear combination of these 3 terms:

\[
N = f(H_s, v_c, T, x, y) = \beta_1 H_s^2 + \beta_2 |v|^2 + \beta_3 \frac{v_c}{H_s/T}(x, y)
\]

(12)

Where \(T\) represents the peak wave period and \(\beta_W, \beta_s, \beta_Y\) are parameters for bringing the three terms to the same order of magnitude. \(\beta_W\) is of order 0.1, \(\beta_s\) of order 10 and \(\beta_Y\) of order 1. The resulting hazard levels appear in figure 14, showing an alternating pattern of higher and lower hazard levels essentially reflecting the wave energy pattern due to refraction/diffraction. That is because in this case the first term in equation 12 dominates and because the index does not consider navigation hazards associated to wind or the dangerous areas near harbour structures or rocky shores.
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7.3 Applications for Erosion and Flooding

The combined predictions of submerged and emerged beach evolution have served to derive two morphodynamic indexes for the submerged and emerged subdomains, using the BSS value as a guide for the practical reliability of the results. The seabed evolution under the considered storm appears in Figure 15 which shows the erosion/accretion pattern under the growth stage of the storm (left) and the simulated morphodynamic evolution assessed by means of the BSS index (right). The purple area corresponds to BSS > 0.35, indicating that the morphodynamic prediction is reliable. The index for the submerged beach characterises erosion ($E$) and is defined as:

$$E(x, y) = \Delta z(x, y)$$

(13)

Where $\Delta z$ is the bed level erosion (accretion has not been included in the assessment).

For the emerged beach the index characterises flooding ($F$) and is defined as:

$$F(x, y) = \Delta h(x, y)$$

(14)

Figure 14: Plot of the $N$ index characterizing navigation hazards for the studied coastal sea and estuary domains. The $N$ index has not been estimated for the rocky shores or areas near harbour outer breakwaters.

Figure 15: Erosion/accretion pattern under the growth stage of the storm (left) and the reliability of the morphodynamic prediction in terms of the BSS index (right), the color scale represents erosion (negative values) and accretion (positive values) in metres. The areas with BSS > 0.35 indicate a more reliable prediction for the hazard level.
Where $\Delta h$ is the flooding thickness. The predicted flooding levels under low (left panel) and high (right panel) astronomical tidal levels, superposed to a storm surge of 0.5m, appear in Figure 16, showing more extended flooding under high tide, particularly at the western side of the beach and embayment. The increase in water level also clearly affects the breakwater on the east side, indicating higher overtopping rates.

![Figure 16: Flooding pattern due to a storm surge of +0.5 m under low (left) and high (right) astronomical tide conditions. The colour scale represents flooding ranging from 0 (no flooding) to 2 m.](image1)

### 7.4 Applications for Water Sports

The index for water sport activities ($WS$) can be written as a function of wave and current energy (eq. 15), proportional to the squared wave height and current velocity, respectively:

$$WS(x, y) = \alpha_w H^2(x, y) + \alpha_n v^2(x, y)$$  \hspace{1cm} (15)

Where $H$ and $v$ represent wave height and modulus of current velocity, function of the spatial coordinates, while $\alpha_w$ and $\alpha_n$ represent parameters to bring both variables to the same order of magnitude. For the presented application $\alpha_w$ has been set to 1.0 and $\alpha_n$ has been set to 50. The resulting hazard map for water sports such as surfing or swimming, appears in Figure 17, displaying hazard areas before a typical storm event (left panel) and during the peak of that storm (right panel), with low hazard levels in green and high levels in red. The index shows an alternating pattern, green to yellow in calm conditions and yellow to red under storm conditions. This essentially reflects the wave field, indicating a dominance of the first term in equation 15.

![Figure 17: Hazard index maps for water sports in calm (left) and storm (right) conditions for the studied embayment. Hazard level is indicated by colour-coding where green stands for low, yellow for medium and red for high.](image2)
8 CONCLUSIONS AND FUTURE WORK

Applying the increasing number of available field data and operational predictions to harbour and beach decisions requires explicit error intervals and tailored indexes, suited to users’ requirements. The developed models reduce error levels in hydro-morphodynamic predictions by regular bathymetric updating, while assessing uncertainty by means of indicators such as the BSS index. This provides an objective criterion to determine the reliability of simulations, from which a number of indexes for common applications in coastal zones are proposed to characterize hazard levels for navigation, dredging, beach erosion, berm flooding or water sports. Error intervals have been determined by comparing nested simulations with in situ and video data for the studied coastal domain, providing a basis for further applications, where the accuracy has been enhanced through a regular updating of the domain geometry. This geometry, numerically obtained after each morphodynamic time step, can be validated by means of surveys, aerial images or averaged video recordings. The morphodynamic time step (1 to 2 days) should capture the main features of seabed evolution while avoiding the accumulation of numerical errors. Bathymetric updating and a smart selection of hydro-morphodynamic parameters (e.g. average instead of peak period) allows a significant reduction of errors, bounding the uncertainty in error propagation from deep to shallow water. Morphodynamic errors exceed hydrodynamic ones by up to one order of magnitude, getting smaller under storm events, which are the ones more critical for managing hazard levels.

The nested and coupled modelling suite, combined with permanent monitoring can provide explicit error intervals to enhance the uptake of the proposed indexes in the working routines of harbour and beach users/managers. An efficient post-processing module will support the use of these indexes for coastal activities and risk management. The computational efficiency of the developed models suggests their introduction into an operational service, where hydro-morphodynamic fields and indexes can be predicted a few days in advance (about 5 to 7 nowadays in most operational services). By combining numerical simulations with field observations it will be possible to provide advanced information on hazard levels for harbour and coastal engineering decisions and from that optimise risks under storm events or associated to typical operations in coastal seas such as dredging or water sports.

SUMMARY

Increasing pressures and co-existing activities on coastal seas require reliable information on risk levels and, if possible, with sufficient advance time to enable information-based decisions for harbour and coastal engineering. This work suggests a coupled set of hydro-morphodynamic models that reduce error levels with respect to the state of the art by considering the evolving bathymetry in shallow coastal areas and how that affects wave and current predictions. These models are accompanied by a post-processing routine that, based on available buoy data and coastal video recordings, leads to explicit error intervals for the calculated hydro-morphodynamic variables. The resulting wave, circulation and morphodynamic fields, with associated error levels, have served to derive user oriented indicators to facilitate the uptake of these modelled results into the working protocols of harbour and coastal Authorities, as well as in the decisions of any other interested end-user. The developed models jointly simulate the emerged and submerged coastal zones, promoting a more integrated management and decision making while suggesting their introduction into an operational meteo-oceanographic system. This operationalisation would provide advanced information (with a horizon of about 5 to 7 days) on hydro-morphodynamic variables and the resulting hazards for activities such as dredging, coastal navigation, water sports or beach management under erosion and flooding.

The developed modelling suite has been adapted and tested for a typical storm acting on a macro-tidal bay and beach with multiple co-existing uses. This study area is located at the beginning of the main breakwater of the Bilbao Harbour (Bay of Biscay coast of Spain), illustrating the need of such modelling tools for the complex harbour-beach interactions so common in developed or developing coasts. The proposed indicators characterise risk levels for a set of illustrative activities in coastal seas, supporting information-based decisions for harbour and coastal engineering and thus bounding and making explicit
their associated hazard levels. Such a combination of models-observations-indicators could be introduced into operational systems, like the one currently run by Puertos del Estado, to provide advanced information of hydro-morphodynamics and user oriented indexes. This would facilitate further calibrations of the proposed models and the derivation of additional indicators to face present and future weather or climatic impacts in such heavily pressured coastal seas, resulting in improved risk levels and overall sustainability of conflicting uses.

RESUME

Les pressions croissantes et les activités coexistantes sur les mers côtières nécessitent des informations fiables sur les niveaux de risque et, si possible, avec un temps d'avance suffisant pour permettre des décisions basées sur l'information pour l'ingénierie portuaire et côtière. Ce travail propose un ensemble couplé de modèles hydro-morphodynamiques qui réduisent les niveaux d'erreur par rapport à l'état de l'art en prenant en compte l'évolution de la bathymétrie dans les zones côtières peu profondes et la façon dont cela affecte les prédictions de vagues et de courants. Ces modèles sont accompagnés d'une routine de post-traitement qui, sur la base des données de bouées et des enregistrements vidéo côtiers disponibles, conduit à des intervalles d'erreur explicites pour les variables hydro-morphodynamiques calculées. Les champs de vagues, de circulation et de morphodynamique qui en résultent, avec les niveaux d'erreur associés, ont servi à dériver des indicateurs orientés vers l'utilisateur pour faciliter l'intégration de ces résultats modélisés dans les protocoles de travail des autorités portuaires et côtières, ainsi que dans les décisions de tout autre utilisateur final intéressé. Les modèles développés simulent conjointement les zones côtières émergées et submergées, favorisant une gestion et une prise de décision plus intégrées tout en suggérant leur introduction dans un système météo-océanographique opérationnel. Cette opérationnalisation fournirait des informations avancées (avec un horizon d'environ 5 à 7 jours) sur les variables hydro-morphodynamiques et les risques qui en résultent pour des activités telles que le dragage, la navigation côtière, les sports nautiques ou la gestion des plages soumises à l'érosion et aux inondations.

La suite de modélisation développée a été adaptée et testée pour une tempête typique agissant sur une baie macro-tidale et une plage avec de multiples usages coexistantes. Cette zone d'étude est située au début de la digue principale du port de Bilbao (côte du Golfe de Gascogne en Espagne), illustrant le besoin de tels outils de modélisation pour les interactions complexes entre le port et la plage, si courantes sur les côtes développées ou en développement. Les indicateurs proposés caractérisent les niveaux de risque pour un ensemble d'activités illustratives dans les mers côtières, soutenant des décisions basées sur l'information pour l'ingénierie portuaire et côtière et limitant et rendant explicite les niveaux de danger associés. Une telle combinaison de modèles, d'observations et d'indicateurs pourrait être introduite dans des systèmes opérationnels, comme celui actuellement géré par Puertos del Estado, pour fournir des informations avancées sur l'hydro-morphodynamique et des indices orientés vers l'utilisateur. Cela faciliterait les calibrations ultérieures des modèles proposés et la dérivation d'indicateurs supplémentaires pour faire face aux impacts météorologiques ou climatiques actuels et futurs dans ces mers côtières soumises à de fortes pressions, ce qui permettrait d'améliorer les niveaux de risque et la durabilité globale des utilisations conflictuelles.

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Las presiones crecientes y las actividades coexistentes en los mares costeros exigen una información fiable sobre los niveles de riesgo y, a ser posible, con suficiente antelación para poder tomar decisiones basadas en la información para la ingeniería portuaria y costera. Este trabajo propone un conjunto acoplado de modelos hidromorfodinámicos que reducen los niveles de error con respecto al estado del arte al considerar la evolución de la topografía en las zonas costeras poco profundas y cómo ésta afecta a las predicciones de oleaje y corrientes. Estos modelos van acompañados de una rutina de posprocesamiento que, basándose en los datos de las boyas y las grabaciones de vídeo costeras disponibles, conduce a intervalos de error explícitos para las variables hidromorfodinámicas calculadas. Los campos de oleaje, circulación y morfodinámica resultantes, con los niveles de error asociados, han servido para derivar indicadores orientados al usuario para facilitar la incorporación de estos resultados modelizados en los protocolos de trabajo de las Autoridades portuarias y costeras, así como en las decisiones de cualquier otro usuario final interesado. Los modelos desarrollados simulan conjuntamente las zonas costeras emergidas y sumergidas, promoviendo una gestión y una toma de decisiones más integradas, al tiempo que sugieren su introducción en un sistema meteo-oceanográfico operativo. Esta operacionalización proporcionaría información avanzada (con un horizonte de unos 5 a 7 días) sobre las variables hidro-morfodinámicas y los riesgos resultantes para actividades como el dragado, la navegación costera, los deportes náuticos o la gestión de las playas bajo erosión e inundación.

El conjunto de modelos desarrollado se ha adaptado y probado para una tormenta típica que actúa sobre una bahía y una playa macromareal con múltiples usos coexistentes. Esta zona de estudio está situada al principio del espigón principal del puerto de Bilbao (costa del Golfo de Vizcaya en España), lo que ilustra la necesidad de este tipo de herramientas de modelización para las complejas interacciones puerto-playa tan comunes en las costas desarrolladas o en desarrollo. Los indicadores propuestos caracterizan los niveles de riesgo para un conjunto de actividades ilustrativas en los mares costeros, apoyando las decisiones basadas en la información para la ingeniería portuaria y costera y, por lo tanto, delimitando y haciendo explícitos sus niveles de peligro asociados. Esta combinación de modelos-observaciones-indicadores podría introducirse en los sistemas operativos, como el que actualmente gestiona Puertos del Estado, para proporcionar información avanzada de la hidromorfodinámica e índices orientados al usuario. Esto facilitaría nuevas calibraciones de los modelos propuestos y la derivación de indicadores adicionales para hacer frente a los impactos meteorológicos o climáticos presentes y futuros en estos mares costeros tan presionados, lo que redundaría en una mejora de los niveles de riesgo y de la sostenibilidad general de los usos en conflicto.

RESUMEN

MOORING LOADS: STATE OF PRACTICE IN THE USA AND NEW DESIGN CONSIDERATIONS FOR DOLPHINS

by

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Keywords: mooring loads, dolphin design, monopile analysis, soil-structure interaction, allowable stress design

Mots clés: charges d’amarrage, conception du dauphin, analyse du monopieu, interaction sol-structure, conception des contraintes admissibles

1 INTRODUCTION

Marine terminals for oil, petrochemicals, and liquefied natural gas (LNG) typically have pile-supported loading platforms which align with the vessel manifold for cargo transfer. The piping and marine loading arms require a small deck compared to the area needed for other cargotypes, such as containers. The cost savings from reducing the amount of structure over water lead to designing loading platforms that are often very much shorter than the vessels. This means that a vessel’s head, stern, and breast lines must be secured to isolated mooring dolphins constructed on either side of the platform; see Figure 1. When designing mooring dolphins there are three key considerations:

- Defining mooring line loads on the structure
- Sizing the capacity of mooring hardware and anchorage to the structure
- Determining the type and size of foundation

Figure 1: Tanker Berth with Mooring Dolphins (Courtesy of Moffatt & Nichol)
Structural engineers who design mooring dolphins, and in particular young professionals, quickly encounter a challenging truth: no one set of codes or standards governs marine structural design. There is a wide-body of knowledge on the subject and guidelines for calculating mooring loads, sizing mooring hardware, and designing foundations. It is often difficult, however, for the engineer to synthesise the available guidance and apply it to detailed design. Certain documents may cover one topic in detail but not others or may be based upon different assumptions. In the USA, it is often at the discretion of the owner and engineer to agree upon a set of standards for a project in a pick-and-choose manner. The selection and use of different codes are of particular value and interest to the industry and can lead to inconsistent or incorrect use of design loads, load factors, and factors of safety. There is a current effort by the American Society of Civil Engineers (ASCE) to develop a unified standard for the analysis and design of marine structures; challenges include a lack of consensus on load combinations and factors of safety. ASCE is scheduled to publish this new standard by 2025.

This paper summarises the state of practice in calculating mooring loads for dolphin design. It then presents a case study which illustrates the design process for new monopile dolphins at an existing terminal in the Southeast USA. It defines the project’s codes, standards and guidelines and then calculates mooring loads using both mooring analysis and analytical equations. Monopile analysis and design are illustrated, highlighting the conflicting guidance on loads and load factors in the codes. New methods are proposed for engineers to avoid overly conservative designs which could otherwise lead to excessive construction material and installation costs. The final section provides lessons learned and recommendations for best practice design methodology and improved guidance for structural engineers.

2 MOORING DOLPHIN DESIGN GUIDANCE

In 2016, PIANC Working Group 153 published ‘Recommendations for The Design and Assessment of Marine Oil and Petrochemical Terminals’ to provide a uniform set of recommendations for designers, owners, operators, and manufacturers. WG 153 drew from worldwide guidance, including the following:

- PIANC
- Chapter 31F of the California State Building Code ‘Marine Oil Terminals’
- British Standards Institution
- Oil Companies Marine International Forum (OCIMF)
- Society of International Gas Tanker and Terminal Operators (SIGTTO)
- American Petroleum Institute (API)
- internal documents by industry operators

WG 153 describes the aim of dolphin design, which is to avoid damage to infrastructure, especially brittle failure modes involving breakout of mooring hardware under load; see Figure 2. This may result in a projectile which could injure personnel, damage the vessel, provide an ignition source, create a spill, or interrupt operations. The aim is a fail-safe system in the event of an overload. Each link in the system must be stronger than the preceding link. The failure sequence should be as follows. When a mooring line is tensioned beyond its Working Load Limit (WLL) the ship’s winch brakes will render and start to release the line in a controlled manner, redistributing load to other lines. Should the winch brake not render and instead lock, the mooring line will part (break) when tension exceeds its Minimum Breaking Load (MBL). The mooring hardware is designed to be stronger than the MBL of the line(s) they support. For example, if a line has an MBL of 138 metric tonnes (t), the Safe Working Load (SWL) of a single hook may be 150 t. The supporting dolphin anchorage and foundation should be designed to resist the SWL of the mooring hardware. Figure 3 illustrates this fail-safe system. This approach is also incorporated into BS 6349-2 ‘Code of Practice for the Design of Quay Walls, Jetties and Dolphins’ (2019).
Figure 2: Brittle Bollard Failure (Courtesy of John Gaythwaite)

Figure 3: Mooring Dolphin Fail-Safe System. Adapted from PIANC WG 153 (2016).

*Winch brakes may be set to render at 60 % to 80 % MBL.*
Mooring loads may be calculated using static or dynamic mooring analysis in a numerical model or via physical model tests. These models calculate allowable tensions in each mooring line which may be summed to give the total load on a dolphin. Alternatively, analytical equations may be used to calculate the total load on a dolphin based on the rated capacity of the mooring hardware. WG 153 Equation 7-1, which is intended for practice in the USA, is presented below.

\[ F_{ZA} = SWL \cdot [1 + 0.6 (n - 1)] \]

- \( F_{ZA} \) = Total horizontal force transferred to the hook assembly and anchorage
- SWL = Safe working load of one hook
- \( n \) = Number of hooks
- The factor of 0.6 is based on ship winch brake rendering per OCIMF MEG4

This equation represents an extreme condition where one quick release hook is loaded to 100% of its SWL with all other hooks at 60% SWL. This exceeds the typical limits of safe mooring practice, where allowable line tensions are limited to 55% MBL. BS 6349-1-2 (2017) provides similar equations for three conditions which are compatible with Eurocode design. The first is an extreme operating condition with loads obtained from mooring analysis. The second is an extreme operating condition with loads calculated based on the ship’s winch brake holding load of 60% MBL (similar to WG 153 Equation 7-1). The third is an accidental condition with loads calculated based on ship’s winch brake holding load of 80% MBL. These are presented together in Figure 4 below in terms of total force on a dolphin.

![Figure 4: Forces on Dolphin. Adapted from MEG4 [Oil Companies International Marine Forum, 2018]. BS 1, 2, and 3 are the three conditions described above from BS 6349-1-2 (2017).](image)

3  CASE STUDY: STEEL MONOPILE MOORING DOLPHINS

A mooring upgrade project in the Southeast USA involves the partial demolition of an existing timber wharf to install steel monopiles with a single mooring bollard mounted on each. The owner wants to accommodate Panamax-class tankers, but the existing wharf and cleats lack sufficient structural capacity to moor these vessels. Figure 5 shows the wharf. The project is currently under construction and the primary challenge in
design was to determine the required capacity and size of the steel monopiles. This case study is organised into the following sections:

- Codes, Standards, and Guidelines  
- Mooring Analysis  
- Mooring Load Calculation Using PIANC WG 153  
- Steel Monopile Analysis and Design

![Figure 5: Existing timber wharf with cleats for barges (Courtesy of Moffatt & Nichol)](image)

### 3.1 Codes, Standards and Guidelines

The state building code adopts the 2015 International Building Code [International Code Council, 2015] for structural design, which in turn adopts the following documents for use in design. Some of these documents have more recent versions that were not yet adopted by the state during the project.

- American Society of Civil Engineer’s (2010): “ASCE / SEI 7-10 Minimum Design Loads for Buildings and Other Structures”. This document includes loads and load combinations for structural design, but its coverage of marine structures is limited (e.g. mooring and berthing forces are not included).
- American Institute of Steel Construction’s (2010): “ANSI / AISC 360-10 Specification for Structural Steel Buildings”. This code governs steel design for buildings but may be applied to marine structures using interpretation and judgement.
- American Concrete Institute’s (2014): “ACI 318-14 Building Code Requirements of Structural Concrete”. This code governs the concrete anchorage design for the bollards.

The following are best practice guidelines in the USA, but not required by code.

- PIANC WG 153 (2016): “Recommendations for The Design and Assessment of Marine Oil and Petrochemical Terminals”. This document provides guidance on mooring loads and dolphin design.
3.2 Mooring Analysis

The design vessel for mooring analysis was defined by analysing characteristics of 121 Panamax tankers as reported by the INTERTANKO Q88 database [Q88 LLC, 2018]. The characteristics are bracketed by confidence intervals and the 95% non-exceedance values are used for design; see Table 1.

<table>
<thead>
<tr>
<th>Classification</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead Weight Tonnage</td>
<td>t</td>
<td>76,000</td>
</tr>
<tr>
<td>Length Overall</td>
<td>m</td>
<td>249</td>
</tr>
<tr>
<td>Length Between Perpendiculars</td>
<td>m</td>
<td>234</td>
</tr>
<tr>
<td>Beam</td>
<td>m</td>
<td>38.4</td>
</tr>
<tr>
<td>Depth</td>
<td>m</td>
<td>21.0</td>
</tr>
<tr>
<td>Loaded Draft</td>
<td>m</td>
<td>14.5</td>
</tr>
<tr>
<td>Ballast Draft</td>
<td>m</td>
<td>8.1</td>
</tr>
<tr>
<td>Loaded Displacement</td>
<td>t</td>
<td>94,859</td>
</tr>
<tr>
<td>Ballast Displacement</td>
<td>t</td>
<td>50,988</td>
</tr>
<tr>
<td>Cargo Manifold Offset from Midships</td>
<td>m</td>
<td>0.9 Forward</td>
</tr>
<tr>
<td>Lateral Windage Area: Ballast Draft</td>
<td>m²</td>
<td>3,685</td>
</tr>
<tr>
<td>Frontal Windage Area: Ballast Draft</td>
<td>m²</td>
<td>883</td>
</tr>
<tr>
<td>Parallel Body: Ballast Draft/Forward/Aft of Manifold</td>
<td>m</td>
<td>70.9/73.5</td>
</tr>
<tr>
<td>No. Mooring Lines Deployed from Winches</td>
<td>No.</td>
<td>12</td>
</tr>
<tr>
<td>Type of Mooring Line</td>
<td>-</td>
<td>Steel Wire</td>
</tr>
<tr>
<td>Mooring Line Diameter</td>
<td>mm</td>
<td>36</td>
</tr>
<tr>
<td>Minimum Breaking Load</td>
<td>t</td>
<td>93</td>
</tr>
<tr>
<td>Mooring Line Tail</td>
<td>-</td>
<td>Nylon (80 mm)</td>
</tr>
<tr>
<td>MBL of Nylon Tail</td>
<td>t</td>
<td>125</td>
</tr>
</tbody>
</table>

Table 1: Design Vessel Characteristics [Q88 LLC, 2018]
OPTIMOOR, developed by Tension Technology International, is a static mooring analysis program which uses vessel, pier, and mooring arrangement data along with wind and current applied at various speeds from any direction. The program provides resultant forces and vessel motion. The mooring arrangement for the design vessel is shown in Figure 6. Each mooring dolphin is labelled MP-1 through MP-4. Four lines are deployed to each of the forward and aft dolphins and 2 lines to each of the inner dolphins. The analysis concluded that the limiting wind speed for safe mooring was 46 knots from any direction, which is based on maintaining mooring line tension below 51 tonnes (55 % MBL). This wind speed is greater than the 1 % annual exceedance value for the site of 43 knots. For comparison, UFC 4-152-01 – ‘Design: Piers & Wharves’ (2017) and UFC 4-159-03 – ‘Design: Moorings’ (2016) recommends a 2-% annual exceedance wind speed for berths where vessels cannot depart in advance of severe weather. Chapter 31F of the California State Building Code ‘Marine Oil Terminals’ (MOTEMS) recommends a 4-% annual exceedance wind speed. The 46 knot wind from the mooring analysis is therefore conservative.

![Figure 6: Plan View of Mooring Arrangements.](image)

<table>
<thead>
<tr>
<th>Mooring Dolphin</th>
<th>No. Mooring Lines per Dolphin</th>
<th>Max. Line Tensions [% MBL]</th>
<th>Force on Dolphin [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>MP 1 &amp; 2</td>
<td>4</td>
<td>20 %, 26 %, 48 %, 54 %</td>
<td>1,350</td>
</tr>
<tr>
<td>MP 3 &amp; 4</td>
<td>2</td>
<td>28 %, 28 %</td>
<td>510</td>
</tr>
</tbody>
</table>

*Table 2: Mooring analysis results*

### 3.3 Mooring Load Calculation Using PIANC WG 153

The WG 153 Equation 7-1 is intended for quick-release hooks but can be applied to bollards by substituting the hook SWL for the mooring line MBL; see MEG4 (2018).

For MP 1 & 4:

\[
F_{ZA} = 93 \left[ 1 + 0.6 \left( 4 - 1 \right) \right] = 260 \text{ t (2,550 kN)}
\]

For MP 2 & 3:

\[
F_{ZA} = 93 \left[ 1 + 0.6 \left( 2 - 1 \right) \right] = 149 \text{ t (1,460 kN)}
\]
WG 153 states that Equation 7-1 represents an ultimate load which should be compared to mooring analysis results times a load factor (e.g. 1.60). This 1.60 value is recommended for Load and Resistance Factor Design (LRFD) load combinations in UFC 4-152-01 – *Design: Piers & Wharves* (2017) and *MOTEMS* [California Code of Regulations, 2016]. These loads are compared in Table 3.

<table>
<thead>
<tr>
<th>MooringDolphin</th>
<th>No. Mooring Lines</th>
<th>Force on Dolphin [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Mooring Analysis</td>
</tr>
<tr>
<td>MP 1 &amp; 2</td>
<td>4</td>
<td>1,350</td>
</tr>
<tr>
<td>MP 3 &amp; 4</td>
<td>2</td>
<td>510</td>
</tr>
</tbody>
</table>

*Table 3: Summary of mooring loads*

### 3.4 Steel Monopile Analysis and Design

Each monopile needed to fit into a pre-cut hole in the wharf and be sufficiently rigid so that it did not impact the adjacent deck when loaded. The lateral capacity of the dolphin depends both on the steel section and soil resistance. The soil beneath the wharf is clay with undrained shear strength between 50 and 100 kPa. The soil-structure interaction of large diameter piles under lateral loads has long been the subject of research of the offshore construction industry. Much of this research is based on soil testing and empirical data. Tomlinson & Woodward (2015) review the state of the practice for laterally loaded piles, from hand calculations to numerical models to pile testing.

Clays are nonlinear under lateral loading and it is necessary to relate the pile-soil deformation to soil resistance. The American Petroleum Institute (API) provide guidance on modelling this behaviour and recommends the use of lateral soil resistance versus deflection curves, commonly called p-y curves [American Petroleum Institute, 2014a ; 2014b]. These are derived empirically from stress-strain data from both laboratory tests and full scale pile testing; see Matlock (1970). Different curves are provided at different soil depths and can be applied in conjunction with beam-column structural analysis methods to calculate bending moments, shears, soil reaction, and compatible pile-soil deformation. The software LPile, developed by Ensoft Inc. (2019), is used in the USA to analyse single laterally loaded piles using p-y curves. The pile is modelled as a beam-column and p-y curves are applied as nonlinear Winkler–spring mechanisms. The program allows the user to input soil parameters, pile geometry, material properties, and loads; see Figure 7.

![Figure 7: Typical monopile loading conditions and p-y Curves](Ensoft, Inc., 2019)
LPile is well-suited to modelling monopiles under mooring loads. The engineer should, however, be cautious in the application of loads calculated using WG 153 Equation 7-1, which are ultimate loads. The long-established research of API, including the derivation of p-y curves from test data and its provisions for pile structural design, are based upon unfactored service loads. APIRP 2A-WSD (2014a) states that platforms and piles shall be designed using allowable stress (working stress) methods, commonly referred to as Allowable Stress Design (ASD). This means that modelling soil-structure interaction using p-y curves for pile design in accordance with API is intended for use with ASD. This requires the use of unfactored service loads. Also, the LPile default settings are based on ASD with unfactored service loads.

Applying the ultimate load from WG 153 Equation 7-1 in LPile using ASD methods results in an overly conservative design. For example, consider MP 1 and 4. The steel monopiles have a yield stress of 345 MPa. Deflections at the top of the pile must be below 600 mm to avoid the dolphin deck contacting the existing wharf. The ultimate load results in significantly higher bending moments and requires a larger pile compared to the service load obtained from mooring analysis; Figure 8 and Table 4 compare results.

![Figure 8: Bending moments in monopiles for MP 1 & 4. Left side shows service load from mooring analysis. Right side shows ultimate load from WG 153 Eqn. 7-1.](image)

<table>
<thead>
<tr>
<th>Description</th>
<th>Load [kN]</th>
<th>Max. Moment [kNm]</th>
<th>OD x WT [mm]</th>
<th>Pile Length [m]</th>
<th>Pile Weight [Mg]</th>
<th>Deflection [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>WG 153 Eqn. 7-1</td>
<td>2,550</td>
<td>50,500</td>
<td>2286 x 64</td>
<td>50</td>
<td>175</td>
<td>482</td>
</tr>
<tr>
<td>Mooring Analysis</td>
<td>1,350</td>
<td>24,800</td>
<td>1829 x 51</td>
<td>45</td>
<td>101</td>
<td>466</td>
</tr>
</tbody>
</table>

OD = outside diameter. WT = maximum wall thickness.

Table 4: Pile design requirements for MP 1 & 4 under different loads
There is an option to run the LPile analysis using ultimate loads (LRFD). However, the research of the application of p-y curves using ultimate loads is less established within the offshore industry. Using ultimate loads in an LRFD analysis may push soil resistance closer to the highly non-linear peak of the p-y curves. The foreword of API RP 2A-WSD warns against using LRFD methods for pile design. It states that the code is based on the 1989 edition of the AISC – ‘Specification for Structural Steel Buildings – Allowable Stress Design and Plastic Design’ and that later versions of AISC specifications are specifically not recommended for design of offshore platforms. The load and resistance factors in the new codes are derived from and calibrated for building design. These factors are based in part on the proportion of live loads to dead loads subject to different loads. Two new analytical methods are proposed for handling this conflict. These methods are not available in the published literature and are described in detail below.

3.4.1 Method 1: Reduce WG 153 Equation 7-1 Mooring Load to an Equivalent ASD Load

The WG 153 Equation 7-1 mooring load is an ultimate load which is not compatible with analysis using LPile and allowable stress design (ASD). This load may be reduced to a compatible service load for ASD design. UFC 4-152-01 – ‘Design: Piers & Wharves’ (2017) Table 3-7 defines a load factor of 1.60 to be applied to mooring loads for use in LRFD, or ultimate, design. The ultimate load from WG 153 Equation 7-1 may be reduced to an equivalent service load by dividing by 1.60. For this case, dividing 2,550 kN by 1.60 gives 1,590 kN. This load is still 18% greater than the 1,350 kN load obtained from mooring analysis.

3.4.2 Method 2: Apply full WG 153 Mooring Load but Permit Increase in Allowable Stress

API RP 2A-WSD (2014a) Section 6.1.2 states that when stresses are caused by lateral forces from ‘design environmental conditions’ that allowable stresses may be increased by one-third. The API RP 2A-WSD equation for allowable bending stress for cylindrical elements with a diameter to wall thickness (D/t) ratio between 10,340/F_y and 20,680/F_y (in SI units) is given below. A similar approach to allowable bending moments is provided in AISC/ANSI 360-10.

\[
F_b = 0.84 - 1.74 \left( \frac{F_y D}{Et} \right) F_y
\]

- D = outside diameter of pile
- E = Young’s modulus
- t = wall thickness
- F_y = specified minimum yield stress

Permitting a one-third increase in allowable stress was part of older editions of ASCE/SEI and AISC codes but has since been replaced by load reduction factors (e.g. Method 1 above). Mueller & Carter (2003) provide a useful history. ASCE/SEI 7-10 (and the newer 7-16) still permits increases in allowable stress as long as they are not used in conjunction with load reduction factors. That is, Methods 1 and 2 may not be applied together. Table 5 compares results.

<table>
<thead>
<tr>
<th>Method</th>
<th>Load [kN]</th>
<th>Max. Moment [kNm]</th>
<th>Allowable Bending Stress [MPa]</th>
<th>Calculated Bending Stress [MPa]</th>
<th>ODxWT [mm]</th>
<th>Pile Length [m]</th>
<th>Pile Weight [Mg]</th>
<th>Deflection [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>MooringAnalysis</td>
<td>1,350</td>
<td>24,800</td>
<td>253</td>
<td>201</td>
<td>1829x51</td>
<td>45</td>
<td>101</td>
<td>466</td>
</tr>
<tr>
<td>Method 1</td>
<td>1,600</td>
<td>28,300</td>
<td>253</td>
<td>230</td>
<td>1829x51</td>
<td>45</td>
<td>101</td>
<td>526</td>
</tr>
<tr>
<td>Method 2</td>
<td>2,550</td>
<td>49,900</td>
<td>336</td>
<td>405</td>
<td>1829x51</td>
<td>55</td>
<td>123</td>
<td>1,160*</td>
</tr>
</tbody>
</table>

*Deflection under ultimate load is not valid for comparison to other service loads.

Table 5: Comparison of methods for monopile analysis
Method 1 results in marginally higher bending moment and deflection than the mooring analysis case but the same pile size still works. This is because the Method 1 load represents a service load which is compatible with the service load analysis and design methodology (ASD). Method 2, however, results in unacceptably high bending stresses for the same pile size, even with the increase in allowable stress of one-third. This is because although higher stress is permitted, the ultimate load induces significantly higher bending moments in the pile. The load is an ultimate load and is not compatible with ASD methods. The resulting deflections are not valid for comparison with the other two service loads.

Method 1 is recommended for use instead of Method 2 because it represents a service load which is compatible with ASD design using p-y curves as prescribed by API RP 2A-WSD (2014a), whereas Method 2 does not.

The final design for MP-1 and MP-4 is shown below in Figure 9. This case study assumes that the wall thickness is continuous along the pile length; however, in reality it varies with depth. The bollard is sized to have a SWL slightly larger than the design mooring load. The bollard casting is designed using LRFD (ultimate design) by using the full WG 153 Equation 7-1 load. The concrete pile plug provides sufficient frictional resistance to avoid pullout. The reinforced concrete is designed to resist loads up to the tensile strength of the anchor bolts.

Figure 9: Elevation View of Monopile Dolphin for MP 1 & 4. Dimensions are in mm.

4 CONCLUSIONS

Wind, waves, and current acting on vessels generate line tensions which can exert large lateral forces on dolphins and typically govern their structural design. Mooring loads may be calculated using static and dynamic mooring analysis or physical model tests, which normally limit the allowable line tensions to 55% MBL. These tensions are summed to define the total load on a dolphin, which represents a service load. Alternatively, analytical equations such as PIANC WG 153 Equation 7-1 may be used to calculate the load on a dolphin based on the rated capacity of the mooring hardware. This represents an ultimate load and is significantly higher than the load from mooring analysis.
A case study for the design of steel monopile mooring dolphins in the Southeast USA is presented. Mooring loads are calculated via mooring analysis and compared to WG 153 Equation7-1. These loads are applied to the monopile in the software LPile [Ensoft, Inc., 2019], which models soil-structure interaction using soil springs. The soil-springs are defined using lateral soil resistance versus deflection curves, commonly called p-y curves, which are derived empirically from stress-strain data from laboratory tests and full scale pile testing. The application of p-y curves to laterally loaded pile design was developed largely by the American Petroleum Institute:see API RP 2A-WSD (2014a) and API RP 2GEO/ISO 19901-4:2003 (2014b). The methods of piledesign prescribed by API and the derivation of p-y curves, however, are based on allowable stressdesign (ASD) and unfactored service loads. This means that the ultimate load calculated using WG153 Equation 7-1 is not compatible with this method and results in overly conservative pile sizes. Applying the service load calculated from mooring analysis results in a 45-m long pile with a diameter of 1829 mm, wall thickness of 51 mm, and mass of 101 Mg. Applying the ultimate load from WG 153 results in a 50-m long pile with a diameter of 2286 mm, wall thickness of 64 mm, and mass of 175 Mg.

Two new methods are proposed to apply the ultimate load from WG 153 Equation 7-1 in piledesign. Method 1 reduces the ultimate (LRFD) load to an equivalent service (ASD) load by dividing by a load factor of 1.60. The 1.60 factor is defined in UFC 4-152-01 – ‘Design: Piers & Wharves’ (2017) and ‘MOTEMS’ [California Code of Regulations, 2016] for LRFD load combinations. Dividing by 1.60 brings the ultimate load down to a service load which is compatible with LPile analysis and ASD pile design. Applying this method results in a 45-m long pile with a diameter of 1829 mm, wall thickness of 58 mm, and mass of 101 Mg (same pile size as for mooring analysis load).

Method 2 does not reduce the load but instead permits a one-third increase in allowable bending stress in the steel pile. This increase is permitted by API RP 2A-WSD (2014a). Applying this method with the same pile size as Method 1 results in excessive bending stresses, even with the one-third increase. This is because the ultimate load induces significantly higher bending moments in the pile. This load is not compatible with ASD analysis and design and the resulting deflections are not valid for comparison against the other methods.

Method 1 is recommended for use because it represents a service load which is compatible with ASD design using p-y curves as prescribed by API RP 2A-WSD (2014a), whereas Method 2 does not. In addition to monopiles, this guidance may be applied to the ASD design of multi-pile dolphins which are analysed using 3-D finite element models that employ soil-springs derived from LPile analyses.

This case study helps clarify existing challenges and advances the state of practice for mooring dolphin design using codes and standards from the USA. The application of the PIANC WG 153 guidance on mooring loads to the design of a monopile dolphin is described in a level of detail not available in the published literature. The two proposed methods may be applied by structural engineers designing monopile or multi-pile dolphins both in the USA and internationally. This research is based on limited project experience and will require further validation to confirm its applicability to other projects.

5 ACKNOWLEDGEMENTS

I thank my colleagues at Moffatt & Nichol for their support and guidance. I extend special thanksto Martin Eskijian, Matthew Trowbridge, Omar Jaradat, Brian Shaw, and Joshua Martinez.

6 REFERENCES

- American Concrete Institute (2014): “ACI-318-14 Building Code Requirements for Structural Concrete”, Farmington Hills, MI: ACI.
Riverside walkways and promenades are very common in towns and cities with navigable waterways. Some of these locations are exposed to overtopping of vessel-generated waves. The widely used guidance to assess overtopping is EurOtop, which is applicable to random wave trains associated with wind seas. Currently there is no methodology for the assessment of overtopping due to vessel wake. This paper presents a procedure for estimation of overtopping due to vessel wake waves at vertical walls (with no influence of the foreshore and non-impulsive conditions) by modifying the equations given in EurOtop II (pre-released 2016). The alterations made address statistical differences between wind-generated and vessel-generated waves, by changing the Weibull scale and shape parameters that describe the distribution of overtopping events. Physical modelling was carried out to validate the approach. The modelling results indicate that mean overtopping discharge from both irregular and ship waves are comparable for the case studied. It was concluded that the modified EurOtop II formulae are appropriate for estimating the mean discharge and maximum volume of overtopping due to wake waves.
Les promenades et les promenades au bord de l’eau sont très fréquentes dans les villes où il y a des voies navigables. Certains de ces endroits sont exposés au débordement des vagues générées par les bateaux. Le guide largement utilisé pour évaluer le débordement est EurOtop, qui s’applique aux trains de vagues aléatoires associés aux mers de vent. Actuellement, il n’existe pas de méthodologie pour l’évaluation du débordement dû au lavage des navires. Ce document présente une procédure d’estimation du débordement dû aux vagues de sillage des navires sur les parois verticales (sans influence de l’estran et des conditions non impulsives) en modifiant les équations données dans EurOtop II (pré-publié en 2016). Les modifications apportées portent sur les différences statistiques entre les vagues générées par le vent et celles générées par les navires, en changeant l’échelle de Weibull et les paramètres de forme qui décrivent la distribution des événements de débordement. Une modélisation physique a été réalisée pour valider l’approche. Les résultats de la modélisation indiquent que le déversement moyen des vagues irrégulières et des vagues de navire est comparable pour le cas étudié. Il a été conclu que les formules EurOtop II modifiées sont appropriées pour estimer la décharge moyenne et le volume maximum de débordement dû aux vagues de sillage.