CORRIGENDUM No. 1/2025

This corrigendum contains updates to the Port Works Design Manual, 2002 Edition, and shall be read in conjunction with Corrigenda 1/2006, 1/2014, 1/2018, 1/2020 and 1/2022.

(A) Use of Fibre-Reinforced Polymer (FRP) in Marine Structures

PART 1 – General Design Considerations for Marine Works

(a) CONTENTS

Add the following sections:

Section 6.11 Fibre-Reinforced Polymer

Section 7.5 Strengthening of Existing Marine Structures with Fibre-Reinforced Polymer Systems

(b) Section 6.11 Fibre-Reinforced Polymer

Add the following section:

6.11 Fibre-Reinforced Polymer

6.11.1 General

Fibre-Reinforced Polymer (FRP) composites are a class of advanced materials engineered by embedding high-strength, high-stiffness fibres within a protective polymer matrix such as through the pultrusion process. The fibres serve as the primary load-bearing component, providing the material with its characteristic strength and rigidity, while the polymer matrix binds the fibres together, transfers loads between them, and protects them from environmental and physical damage. A range of fibre types can be utilised, each offering a unique profile of mechanical properties. The most common fibres are glass and carbon. The polymer matrix is typically a thermosetting resin, such as polyester, vinyl ester, or epoxy.

FRP materials can be manufactured into various forms to suit different applications such as reinforcing bars, and filament winding for hollow or cylindrical components. They are also available as dry fibre fabrics or pre-cured laminates (plates or strips) for on-site strengthening and repair applications. The combination of fibre, matrix, and manufacturing process allows FRP composites to be tailored to meet specific performance requirements, offering a versatile alternative to traditional construction materials like steel.

6.11.2 Use of FRP Materials in Marine Structures

In the context of marine structures, Glass Fibre-Reinforced Polymer (GFRP) is the most widely adopted type of FRP. This preference is driven by its compelling balance of high corrosion resistance, adequate structural strength, and a more favourable cost profile compared to higher modulus materials like Carbon Fibre-Reinforced Polymer (CFRP). The primary application of GFRP in marine environments is as a direct, non-corroding alternative to conventional steel reinforcement in new concrete structures. GFRP reinforcements are particularly advantageous in elements situated within the highly corrosive tidal and splash zones. By eliminating the risk of reinforcement corrosion, the use of GFRP reinforcement can reduce the overall life cycle costs.

Furthermore, GFRP and CFRP materials can serve as an option in the rehabilitation and strengthening of existing marine structures. Externally Bonded Reinforcement (EBR) systems, utilising either wet layup of fibre fabrics or adhesive bonding of pultruded laminates, are employed to restore or enhance the structural capacity of deteriorated components. Common applications include the flexural strengthening of beams and deck soffits, as well as shear strengthening of beams. For instance, wrapping corrosion-damaged concrete or steel piles with FRP jackets can arrest further deterioration, restore lost cross-sectional area, and enhance the overall durability and load-bearing capacity of the piles.

6.11.3 Considerations of Adopting Fibre-Reinforced Polymer Materials

The use of FRP materials for strengthening existing marine structures offers compelling advantages over traditional materials like steel, particularly in aggressive coastal environments with following advantages:

- Corrosion Resistance: FRP's exceptional corrosion resistance, which directly combats the primary deterioration mechanism in marine settings and significantly extends the service life of structures, reducing long-term maintenance burdens.

- High Strength-to-Weight Ratio: FRP exhibits a high strength-to-weight ratio, enabling substantial structural enhancement without adding significant mass. This also facilitates faster installation processes and minimizes the need for heavy lifting.

- Flexible Application: The inherent flexibility of fibre sheets or fabrics to form FRP onsite by applying resin allows them to be effectively bonded or wrapped around complex, irregular shapes and surfaces of existing structural members, a task often challenging with rigid materials like steel.

- Tailorable: The orientation of the fibres can be precisely aligned according to the specific structural demands, optimizing performance and enhancing cost-effectiveness for the intended strengthening application.

It is worth noting that the mechanical behaviour of FRP differs fundamentally from that of steel. The designers should understand and take into account the characteristics and limitations when adopting this material in the design such as the following:

- Material Cost: The material costs of FRP materials, especially CFRP materials, are higher than traditional steel or concrete materials although FRP-strengthened structures generally have lower life-cycle costs due to the reduced maintenance needs.

- Linear Elastic Behaviour: FRP materials have linear stress-strain behaviour, unequal tensile and compressive strength, which need to be properly considered in the structural design, especially for the serviceability limit state.

- Performance under High Temperature: FRP materials are susceptible to degradation at high temperatures, which should be properly considered in the design when the fire resistance of the structures is crucial such as superstructure of marine facilities which may subject to fire risks.

- Low Elastic Modulus of GFRP: The elastic modulus of GFRP materials is relatively low (i.e. around 50 GPa), which should be taken into account when structural strengthening.

While FRP products can serve as an option for construction of new marine structures or strengthening of existing marine structures, a balanced consideration of these advantages and limitations is essential for the selection and design of FRP strengthening for marine structures. For the details, the *Design Guidelines for the Use of Fibre-Reinforced Polymer (FRP) in Marine Structures* are available on the CEDD departmental website under CEO Publications for the project proponents' reference.

(c) Section 7.5 Strengthening of Existing Marine Structures with Fibre-Reinforced Polymer Systems

Add the following section:

7.5 Strengthening of Existing Marine Structures with Fibre-Reinforced Polymer Systems

(1) Introduction of Externally Bonded FRP Systems

Externally bonded FRP systems are a well-established technology for strengthening and rehabilitating existing infrastructures. These systems, which include FRP sheets, laminates, or strips applied to the surface of existing reinforced concrete members to enhance their structural performance. Common applications include bonding FRP laminates to beams or columns to increase flexural or shear capacity.

(2) Typical Applications

In Hong Kong, FRP has been used for strengthening existing structures since the 2000s with notable applications in strengthening of floor slabs, and the structural upgrading of bridges and heritage buildings.

The ability to provide substantial structural enhancement with minimal alterations to the original dimensions and self-weight of the structure, combined with its superior corrosion resistance, relative ease of installation, flexibility to conform to complex shapes, and overall cost-effectiveness over the structure's lifecycle, has made FRP strengthening an option for retrofitting and upgrading existing marine infrastructure. The designer should note and understand the characteristics and limitations before opting for using this material by balancing the factors such as those listed in Section 6.11.3.

(3) Key Design & Installation Considerations

The following considerations should be taken into account in FRP strengthening:

- Material Selection: FRP materials must meet specified mechanical properties such as tensile strength, modulus of elasticity, and bond strength which directly influence design parameters and safety margins.

- Environmental Durability: Appropriate environmental reduction factors should be applied to account for long-term degradation, with durability assessed over the structure's design life and in accordance with relevant exposure classes.

- Design Approach: Structural design must consider the linear elastic behavior of FRP systems and prioritize the avoidance of brittle failure modes.

- Bonding and Anchorage: Adequate bond strength and proper anchorage using lap joints, U-wraps, or mechanical fasteners are critical to prevent debonding failures.

- Surface Preparation: Substrates must be clean, sound, dry, and free of contaminants or obstructions, with all voids and irregularities repaired or smoothed to manufacturer tolerances.

- Application Technique: FRP systems, whether in situ lay-up or prefabricated, should be installed according to manufacturer instructions, ensuring proper curing and overlap lengths.

- Environmental and Mechanical Protection: Protective coatings and/or regular inspections are essential to maintain long-term performance in harsh marine conditions.

For the details, the *Design Guidelines for the Use of Fibre-Reinforced Polymer (FRP) in Marine Structures* are available on the CEDD departmental website under CEO Publications for the project proponents' reference.

(B) <u>Wave Forces on Pier Deck Structure</u>

PART 1 – General Design Considerations for Marine Works

(a) Section 5.10.6 Wave Uplift

Replace the Section with the following:

For a deck with its soffit just above the still water level, incoming waves may exert impulsive uplift forces as the rising water surface hits the deck's soffit. The impulsive uplift is characterized by relatively high magnitude but short duration. There have been instances of damage to open-type wharves with decks supported by vertical piles, in which the connections between the decks and piles were destroyed and the decks were uplifted while partially damaged. The access bridges between the decks and the earth retaining walls may be broughtdown by the action of impulsive wave uplift. The magnitude of uplift intensity is hard to evaluate.

Based on a study on wave-absorbing seawall, which is a closed structure, for the Victoria Harbour (HKU, 1998), the average uplift pressure on the deck soffit just above the still water level may be in the order of 1.3 to $1.7\gamma_w H_{max}$, where γ_w is the unit weight of seawater. However, the instantaneous uplift pressure may locally rise more than $10\gamma_w H_{max}$, but the equivalent static pressure for calculating stresses within the deck should be less than 4 times the hydrostatic head of the design wave height (OCDI, 2002). For piers supported by vertical piles, the guidance provided in BS6349-1-2:2016 with reference to research papers (McConnell et al., 2004 and Cuomo et al., 2007) is more relevant.

PART 2 – Guide to Design of Piers and Dolphins

(b) CONTENTS

Add the following appendix:
APPENDIX B ASSESSMENT OF WAVE FORCES ON PIER STRUCTURE

(c) Section 3.2.3 Design Wave Height and Pressure

Replace the Section with the following:

A piled deck pier may be considered as comprising a solid concrete deck edge and piles for the purpose of calculating the wave forces on the structure, as recommended in Section 5.10.5 of Part 1 of the Manual.

For closed structures, the wave uplift on the deck may be estimated with reference to the average uplift pressure measured in the study on wave absorbing seawalls (HKU, 1998), which is in the order of 1.3 to $1.7\gamma_w H_{max}$, where γ_w is the unit weight of seawater (see Section 5.10.6 of Part 1 of the Manual). However, it should be noted that the instantaneous uplift pressure may locally rise more than $10\gamma_w H_{max}$, but the equivalent static pressure for calculating stresses within the deck should be less than 4 times the hydrostatic head of the design wave height (OCDI, 2002).

For piers supported by vertical piles, the guidance provided in BS6349-1-2:2016 with reference to research papers (McConnell et al., 2004 and Cuomo et al., 2007) for estimation of lateral wave force and wave uplift force on pier deck structure is more relevant. More details of the estimation of wave loads for piers supported by vertical piles are given in Appendix B.

The impulsive uplift is characterized by the relatively high magnitude but short duration. The designer could estimate the equivalent static pressure by means of dynamic analysis if necessary. Under certain circumstances, the designer could consider to estimate the wave forces by means of physical model.

When the value of $\left(\frac{\eta_{max}-c_l}{d}\right)$ exceeds the limit between 0 to 0.4, the estimation method under BS6349-1-2:2016 is not applicable. In this connection, the lateral wave force may be determined by the method of Goda (BSI, 2000), given in Section 5.10.3 of Part 1 of the Manual. The effective depth of the deck depends on the layout of the deck, such as the depth of the beams and slabs, and must be assessed by the designer. As the method of Goda assumes no wave passing behind the structure but normal piled deck piers allow some wave passing, the method of Goda tends to overestimate the wave pressure exerted on the deck edge. The design wave height is the maximum wave height H_{max}. For design purpose, H_{max} may be taken as $1.8H_{1/3}$, where $H_{1/3}$ is the significant wave height.

The wave force on the piles may be assessed separately by Morison's equation (see Section 5.10.4 of Part 1 of the Manual). The design wave height is the maximum wave height H_{max}, and may be taken as $2H_{1/3}$ seaward of the surf zone, or if the structure is in the surf zone, the highest of the random breaking waves H_{max} at a distance equal to $5H_{1/3}$ seaward of the structure given by the equations in Appendix A of Part 1 of the Manual. A marine growth of 100 mm may be assumed on the piles below the mean sea level for the purpose of calculating the wave force if no other information or measurement is available. For piles closer than about four pile diameters, the loading on the front piles standing side by side in rows parallel to the wave crest should be increased by the factors in Section 5.10.4 of Part 1 of the Manual.

The maximum wave forces on the deck edge and the front piles may occur simultaneously but the maximum wave forces may not occur simultaneously at all piles in a pile bent.

(a) REFERENCES

Add the following references

- Cuomo, G., Tirindelli, M., & Allsop, W. (2007). Wave-in-deck loads on exposed jetties. Coastal Engineering, 54(9), 657 – 679.
- McConnell, K.J. and Allsop, W. and Cruickshank, I. (2004) Piers, jetties and related structures exposed to waves: guidelines for hydraulic loadings. Thomas Telford Publishing.

(b) APPENDIX B ASSESSMENT OF WAVE FORCES ON PIER STRUCTURE

Add Appendix B as shown in the enclosure.

(C) Eco-shorelines

PART 4 –Guide to Design of Seawalls and Breakwaters

(a) Section 3.4 Eco-shorelines	**Add the following after the last paragraph** A study on trial use of eco-shorelines on seawalls at Sai Kung, Lung Kwu Tan, and Ma Liu Shui, representing various water bodies of Hong Kong, was conducted between 2018 and 2021. The ecological monitoring results showed that the marine species richness and uniqueness in eco-shoreline features have consistently surpassed those observed in traditional artificial seawalls. Taking into account the promising and encouraging results, eco-shoreline design has been adopted in various projects in Hong Kong. A prime and representative example is the Tung Chung East reclamation project. Under this project, mangrove eco-shorelines, rocky eco-shorelines, and vertical eco-shorelines have been established across a total length of 3.8 kilometers, which fosters marine biodiversity and nurtures the habitat attracting more than 200 marine species, fishes and shore-birds. These installations have been strategically placed to maximize their ecological benefits while providing robust coastal protection. Eco-shoreline design has also been applied along the existing seawalls, e.g. Tsuen Wan Promenade, Causeway Bay Typhoon Shelter, West Kowloon Cultural District, New Yau Ma Tei Typhoon Shelter and Wan Chai Basin. The *Guidelines of Design, Installation and Maintenance of Eco-engineered Features on Artificial Shorelines* are available on the CEDD departmental website under CEO Publications. Project proponents should follow the Guidelines to adopt the eco-shoreline design into projects involving seawall construction or modification unless the impracticality can be demonstrated.

enclosure

APPENDIX B
ASSESSMENT OF WAVE FORCES ON PIER STRUCTURE
in PART 2 – Guide to Design of Piers and Dolphins

APPENDIX B

ASSESSMENT OF WAVE FORCES ON PIER STRUCTURE

Introduction

Wave-induced vertical forces on horizontal decks or platforms may be considered in three phases. At the instance of contact between the wave crest and the element, the slam or impulsive force, large in magnitude and short in duration, acts on the structure. This is followed by a longer duration (pulsating) positive force and then by a long-duration negative force (especially if the deck is frequently inundated). In some cases, wave momentum may be trapped beneath a deck, especially at the junctions of longitudinal and transverse beams. Local wave loads may be impulsive, with high intensity, but of short duration and spatial extent. An idealised wave load diagram is shown in Figure 1.

Horizontal loads on beam elements often exhibit different characteristics from vertical loads. The magnitude of the first impact load on an external beam (i.e. vertical element at the edge of the jetty) is generally lower than the corresponding vertical impact. For waves underneath any platform formed by beams and deck elements, interactions with the protruding elements are complex, and again wave crests and air may be trapped between beam and deck. This may result in high local horizontal impulsive loads on the seaward face of internal elements and noticeable horizontal forces acting seaward on the shoreward face of the vertical elements.

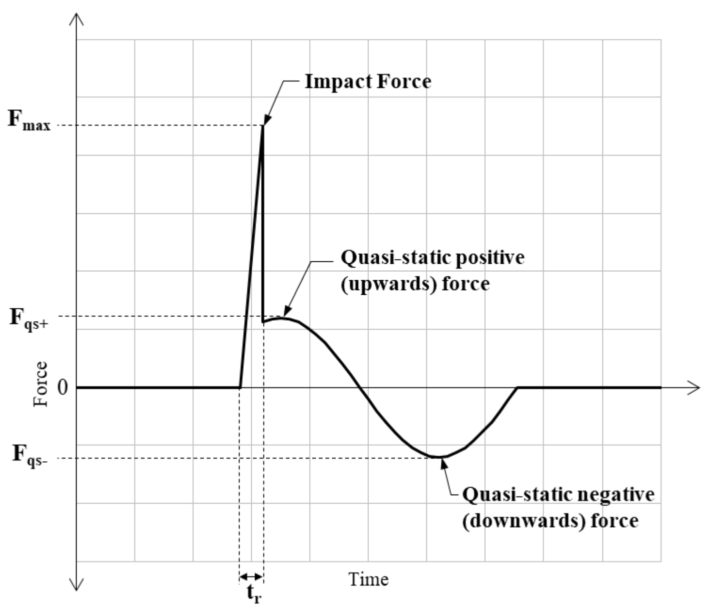


Figure 1 – Idealised force-time history superimposed on a typical force signal recorded by a horizontal element

To measure wave forces on jetty deck and beam elements, physical model studies were carried out at HR Wallingford by McConnell et al. (2004). The model was designed based on consultation with an industrial steering group to ensure that the configuration and dimensions of the elements were typical of real structures. The model was constructed at a scale equivalent to 1:25 based on dimensions of a 'typical' jetty. The model structure and wave conditions were also approximately equivalent to some offshore installations at 1:50. As most of the results are presented in dimensionless form, any particular scale ratio is irrelevant. Three configurations were set up in the physical model studies, a "no-panels (NP)" configuration, a "panels (P)" configuration and a "flat-deck (FP)" configuration.

Based on the results of the physical model studies by McConnell et al. (2003 & 2004), Cuomo et al. (2007) improved the prediction formulae by filtering out the noise / corruption from dynamic response of the model instrument using a wavelet transform method. Three separate sets of coefficients were provided for the three configurations respectively. The prediction formulae and the coefficients for the NP configuration, which is applicable to the typical piers in the form of pile-supported deck structures in Hong Kong, are presented in the following section.

To achieve the most economical design, the pier head should be oriented to the incoming direction wave as far as possible. Designers should also consider other wave directions as appropriate. However, waves from directions other than the dominant wind direction is likely to be much smaller. For wave incoming at an angle, horizontal loads shall be resolved into x and y components.

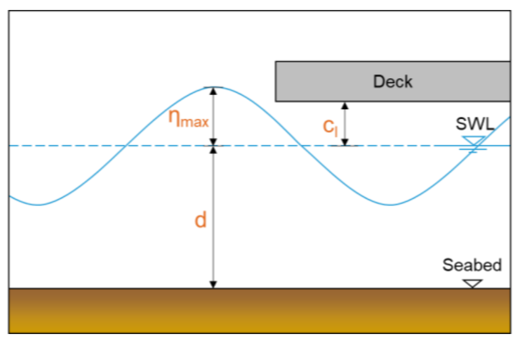
Prediction formulae

For open deck structures, the prediction formulae below are provided by Cuomo et al. (2007). The input parameter for Eq. (1), i.e. $\left(\frac{\eta_{max}-c_l}{d}\right)$, ranges from 0 to 0.4, and has a strong linear correlation with the wave pressure.

The coefficients in Table 1 and 2 below correspond to the "no-panels (NP)" configuration in the physical model study, which is considered suitable for the typical piers in the form of pile-supported deck structures in Hong Kong. For other configurations, further guidance can be found in *Wave-in-deck loads on exposed jetties* by Cuomo et al. (2007).

$$P^*_{1/250} = \frac{P_{qs\,1/250}}{\rho_w \cdot g \cdot H_s} = a \cdot \left(\frac{\eta_{max} - c_l}{d}\right) + b \quad \text{Eq. (1)}$$

$$P_{max\,1/250} = a' \cdot P_{qs+\,1/250} \quad \text{Eq. (2)}$$



where

$P^*_{1/250}$	Dimensionless pressure
$P_{qs\,1/250}$	Quasi-static pressure at 1/250 significance level
$P_{max\,1/250}$	Impulsive pressure
ρ_w	Unit weight of seawater (1,025 kg/m^3)
g	Gravitational acceleration (9.81 m/s)
H_s	Significant wave height
η_{max}	Maximum wave crest elevation
c_l	Clearance between soffit and water level

d	Water depth
a, b & a'	Coefficients (refer to Table 1 and 2 below)

Table 1 – Coefficients *a* and *b* for Eq. (1)

Wave Pressure	Element	Position	a	b
Upward Pressure	Deck	External	1.57	0.52
		Internal	1.57	0.73
	Beam	External	1.10	0.46
		Internal	1.36	0.46
Downward Pressure	Deck	External	-0.66	-0.36
		Internal	-1.35	-0.29
	Beam	External	-0.04	-0.48
		Internal	-0.23	-0.29
Horizontal Pressure	Beam	External	1.19	0.43

Table 2 – Coefficients *a'* for Eq. (2)

Wave Pressure	Element	Position	a'
Upward Impulsive Load	Deck	External	2.22
		Internal	2.29
	Beam	External	2.28
		Internal	2.59
Horizontal Impulsive Pressure	Beam	External	2.45

B.1 Example 1

Calculation of Wave-in-deck Loads Using Cuomo's Method

<u>Given</u>

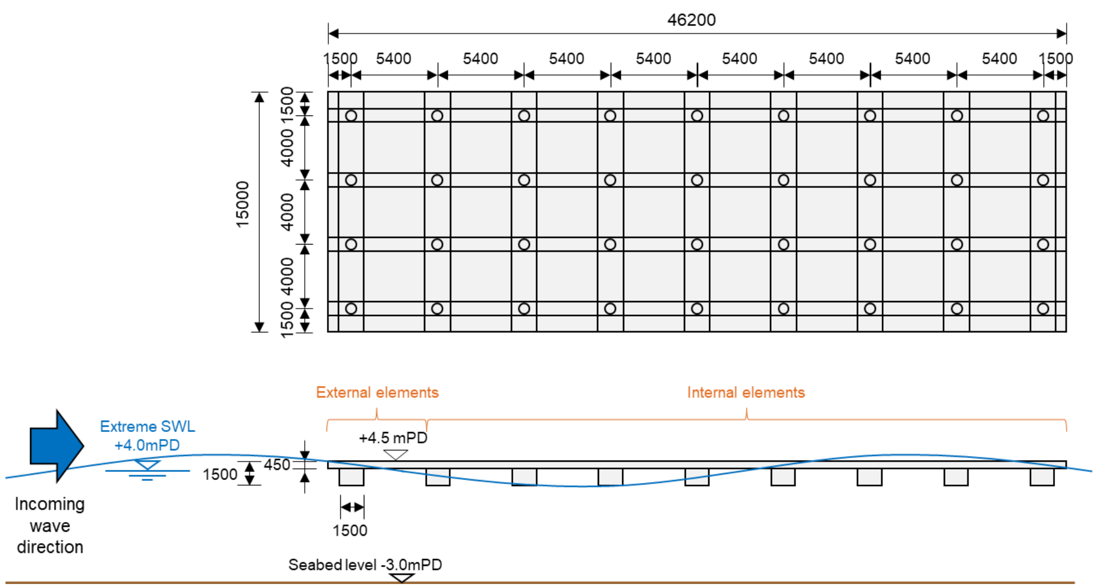
Details for a proposed pier in the form of a pile-supported deck are given as follows:

<u>Deck structure:</u>

Deck Level	=	+4.50 mPD
Slab Thickness	=	450 mm
Pile cap depth	=	1500 mm

<u>Design metocean conditions:</u>

Water Level	=	+4.0 mPD
Seabed Level	=	-3.0 mPD
Significant wave height, H_s	=	$H_{1/3}$ = 2.0 m
Peak wave period, T_p	=	6.0 s



<u>Find</u>

The design wave uplift, downdrag and horizontal pressures acting on the deck

<u>Solution</u>

First, determine c_l , d and η_{max}.

$$c_l = soffit\ level - water\ level = +4.05\ mPD - \ +4.0\ mPD = 0.05\ m$$

$$d = water\ level - seabed\ level = \ +4.0mPD - \ -3.0\ mPD = 7\ m$$

In order to calculate η_{max}, first determine the number of waves during the storm/tide peak, N_z , maximum wave height, H_{max} , and deep water wave length, L_m. The following calculation of H_{max} is based on the most probable value given as a modal value of H_{max} / $H_{1/3}$ by Goda (2000) based on Longuet-Higgins (1952).

Assuming a storm duration of 3 hours,

$$Number\ of\ waves, N_z = \frac{Storm\ duration\ (in\ seconds)}{T_p} = \frac{3 \times\ 3600}{6} = 1800$$

$$(H_{max}/H_{1/3})\ _{mode} \cong 0.706\ \sqrt{\ln N_z} = 0.706\sqrt{\ln 1800} = 1.933$$

$$H_{max} = 1.933 \times 2\ m = 3.87\ m$$

$$L_m = \frac{gT^2}{2\pi} = \frac{9.81 \times 6^2}{2\pi} = 56.21\ m$$

Calculate η_{max} based on the approximation by Stansberg (1991)[1]:

$$\eta_{max} = \frac{H_{max}}{2} \times exp\left(\frac{2\pi}{L_m} \times \frac{H_{max}}{2}\right) = \frac{3.87}{2} \times exp\left(\frac{2\pi}{56.21} \times \frac{3.87}{2}\right) = 2.40\ m$$

Check that $\left(\frac{\eta_{max}-c_l}{d}\right)$ is between 0 and 0.4:

$$\left(\frac{\eta_{max} - c_l}{d}\right) = \left(\frac{2.40 - 0.05}{7}\right) = 0.336$$

Therefore, the prediction formulae by Cuomo et al. (2007) are applicable.

For the impulsive uplift pressure on external deck elements, the coefficients *a* = 1.57, *b* = 0.52 and *a'* =2.22.

[1] McConnell, K. and Allsop, W. and Cruickshank, I. (2004) *Piers, jetties and related structures exposed to waves: guidelines for hydraulic loadings.* Technical Report. Thomas Telford.

Dimensionless pressure:

$$P^*_{1/250} = a \cdot \left(\frac{\eta_{max} - c_l}{d}\right) + b = 1.57 \cdot \left(\frac{2.40 - 0.05}{7}\right) + 0.52 = 1.05$$

Quasi-static upward pressure:

$$P_{qs+\,1/250} = P^*_{1/250} \cdot \ \rho_w \ \cdot g \cdot H_s = 1.05 \times 1.025 \times 9.81 \times 2.0 = 21.1\ kPa$$

Impulsive upward pressure:

$$P_{max\ 1/250} = a' \cdot P_{qs+\,1/250} \ = 2.22 \times 21.1 = 46.8\ kPa$$

For the quasi-static downdrag pressure on external deck elements, a = -0.66 and b = -0.36.

Dimensionless pressure:

$$P^*_{1/250} = a \cdot \left(\frac{\eta_{max} - c_l}{d}\right) + b = -0.66 \cdot \left(\frac{2.40 - 0.05}{7}\right) - 0.36 = -0.58$$

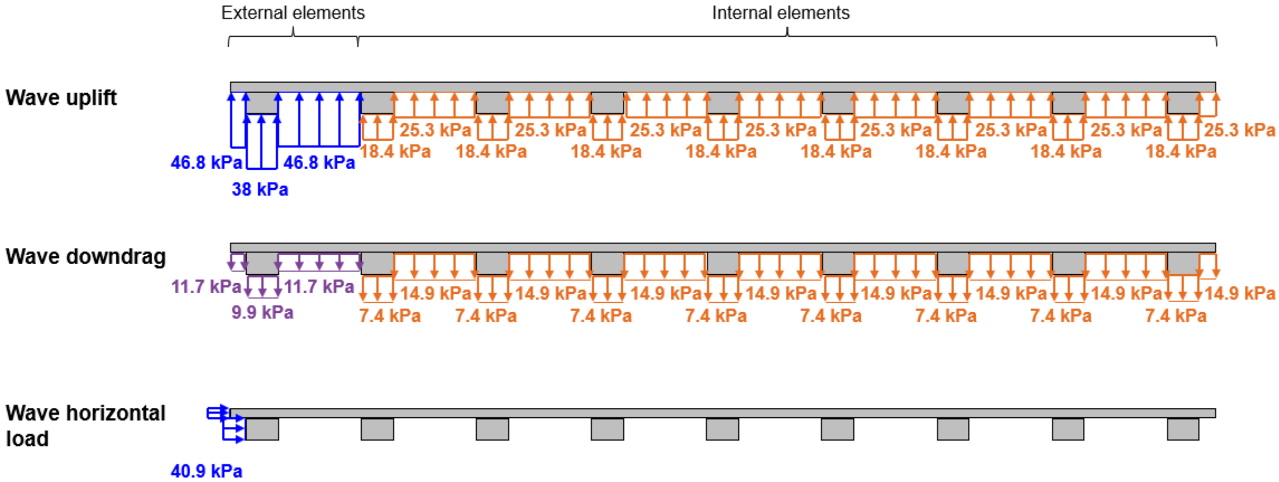
Quasi-static upward pressure:

$$P_{qs+\,1/250} = P^*_{1/250} \cdot \ \rho_w \ \cdot g \cdot H_s = -0.58 \times 1.025 \times 9.81 \times 2.0 = -11.7\ kPa$$

Similarly, referring to the coefficients for other elements, the wave-in-deck pressures are as follows:

	a	***b***	***a'***	**Pressure**
Quasi-static uplift pressure on internal deck elements	1.57	0.73	-	25.3 kPa
Quasi-static downdrag pressure on internal deck elements	-1.35	-0.29	-	-14.9 kPa
Impulsive uplift pressure on external beam element	1.10	0.46	2.28	38.0 kPa
Quasi-static downdrag pressure on external beam element	-0.04	-0.48	-	-9.9 kPa
Quasi-static uplift pressure on internal beam elements	1.36	0.46	-	18.4 kPa
Quasi-static downdrag pressure on internal beam elements	-0.23	-0.29	-	-7.4 kPa
Impulsive horizontal pressure on external beam element	1.19	0.43	2.45	40.9 kPa

The design wave-in-deck loads are as follows:



B.2 Example

Incoming Waves at Angle

Given

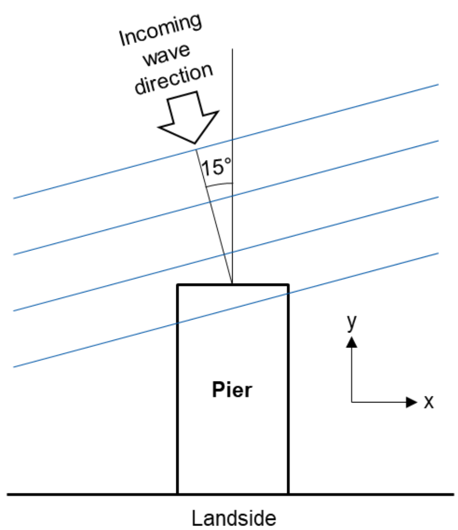
A proposed pier in the form of a pile-supported deck is to be constructed at an angle of 15° to the dominant wave direction.

Deck structure:

Deck Level	=	+4.50 mPD
Slab Thickness	=	450 mm
Pile cap depth	=	1500 mm

Design metocean conditions:

Water Level	=	+4.0 mPD
Seabed Level	=	-3.0 mPD
Significant wave height, H_s	=	$H_{1/3}$ = 2.0 m
Peak wave period, T_p	=	6.0 s



Find

The design horizontal pressures acting on the deck

Solution

First, determine the horizontal wave load.

Similar to Example 1 above, for the impulsive horizontal pressure on external deck and beam elements, the coefficients a = 1.19, b = 0.43 and a' =2.45.

Dimensionless pressure:

$$P^*_{1/250} = a \cdot \left(\frac{\eta_{max} - c_l}{d}\right) + b = 1.19 \cdot \left(\frac{2.40 - 0.05}{7}\right) + 0.43 = 0.83$$

Quasi-static horizontal pressure:

$$P_{qs+\,1/250} = P^*_{1/250} \cdot\ \rho_w\ \cdot g \cdot H_s = 0.83 \times 1.025 \times 9.81 \times 2.0 = 16.7\ kPa$$

Impulsive horizontal pressure:

$$P_{max\ 1/250} = a' \cdot P_{qs+\frac{1}{250}} = 2.45 \times 16.7 = 40.9\ kPa$$

Next, resolve the horizontal wave load into x and y component.

$$P_{x,max\ 1/250,} = 40.9 \times \sin 15° = 10.6\ kPa$$

$$P_{y,max\ 1/250,} = 40.9 \times \cos 15° = 39.5\ kPa$$

The design horizontal wave loads are as follows: